UNIVERSITY OF NOTTINGHAM

SCHOOL OF CIVIL ENGINEERING

CHARACTERISATION OF DRY PROCESS CRUMB RUBBER MODIFIED ASPHALT MIXTURES

by

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To my wife

ABSTRACT

As new European legislation will prohibit the use of shredded scrap tyres for land filling by 2006, governments in the European countries are increasingly under pressure to find alternatives to landfill disposal. The lack of proper controlled use of scrap tyres would lead to added disposal costs, increase illegal dumping or inadequate storage and could increase the risk of fire and environmental damage.

Within the expanding recycling market, only two, to date, have shown the potential to use a significant number of scrap tyres, (i) fuel for combustion and (ii) crumb rubber modified (CRM) material for asphalt paving. Although combustion can consume millions of tyres, it is not an ideal environmental solution. The only remaining potential market for using crumb rubber is CRM material for asphalt paving for road construction either as a binder modifier (known as the wet process) or as an aggregate (known as the dry process). Compared to the wet process, the dry process has been a far less popular method due to inconsistent field performance. However, mixtures produced using the dry process do consume larger quantities of recycled crumb rubber and the dry process is logistically easier than the wet process and therefore potentially available to a larger market.

To characterise the dry process CRM material, the work carried out in this research project has been divided into two parts. In part one, two constituent materials, crumb rubber and bitumen, were investigated in terms of their interaction as a function of bitumen crude source and penetration grades. In the second part, based on the results obtained from rubber-bitumen interaction study, one type of bitumen was selected to be used in the CRM mixture where up to 5% rubber by mass of total aggregate contents was incorporated into a conventional Dense Bitumen Mixture (DBM) designed to use as a binder course layer. The mechanical properties in terms of stiffness, fatigue and resistance to permanent deformation were investigated and compared with the conventional primary aggregate mixtures. In addition, durability

studies were also performed to investigate the moisture susceptibility and long-term ageing characteristics of the CRM materials.

The results from extensive laboratory studies have indicated that the rate of absorption is directly related to the penetration grade (viscosity) as well as to the chemical composition of the bitumen (crude source) but the total amount of absorption is controlled by the nature of the crumb rubber rather than bitumen type and grades. In terms of the rheological properties of the residual bitumen, all the binders showed an increase in viscosity, stiffness (complex modulus) as well as elastic response with these changes being consistent for two crude sources and a range of bitumen grades.

Elemental mechanical testing on the dry process CRM asphalt mixtures have demonstrated that although there is a large reduction in asphalt mixture stiffness, the fatigue performance of the CRM mixtures are generally superior to that of the conventional DBM mixtures, while the permanent deformation performance was found to be only marginally worse particularly at high void contents. The durability studies have indicated that CRM mixtures are more susceptible to moisture induced damage where stiffness, fatigue life and resistance to permanent deformation were adversely effected and showed a general reduction in performance compared to similar mixtures tested in their unconditioned state. The long-term ageing studies also indicated that although the stiffness moduli of the mixtures were increased due to the combined effect of rubber-bitumen interaction and bitumen oxidation, the excessive age hardening of bitumen leads to an increase in brittleness of the mixtures resulting in a reduction in long-term fatigue and resistance to permanent deformation performance.

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DECLARATION

The work described in this thesis was conducted at the University of Nottingham, School of Civil Engineering between October 2000 and September 2003. I declare that the work is my own and has not been submitted for a degree of another university

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CHAPTER 1

Introduction

1.1 BACKGROUND

Scrap tyres form a major part of the world's solid waste management problem. Each year the UK alone produces around 30 million waste tyres with 1 billion being produced globally. Almost half of them are landfilled or stockpiled with the rest being recycled, exported and disposed of illegally. In Europe, governments are attempting to find alternative uses of scrap tyres as new European Union Landfill Directives have already prohibited the disposal of whole tyres to landfill from 2003 and will prohibit land filling of shredded tyre by 2006 (Hird *et al*, 2002). If alternatives to landfill disposal are not found, disposal costs will increase and illegal dumping or inadequate storage will continue to worsen. The fire risk associated with illegal dumps has the potential to cause significant environmental harm. In addition, road traffic is predicted to increase by 17% in the UK alone between 2000 and 2010 (DETR 2000) and, consequently, the number of post-consumer tyres arising in UK is likely to increase.

Within the expanding recycling market, only two applications, to date, have shown the potential to use a significant number of scrap tyres, (i) fuel for combustion and (ii) crumb rubber modified (CRM) material for asphalt paving. Although combustion can consume millions of tyres, it is not an ideal environmental solution. The only remaining potential market for using crumb rubber is CRM material for asphalt paving. In the last two decades, utilisation of scrap tyres as a road construction material has become a popular means to minimise this environmental pressure. Considerable work has been done in various countries in terms of the utilisation of scrap tyres and there is a long list of published literature dealing with different aspects of this challenging material.

The use of scrap tyres in asphalt mixture applications is not a recent development with reclaimed tyre crumb being used in the asphalt industry for over 30 years. Documentation is extensive but disjointed, making a summary of its history difficult (Epps, 1994). However, in recent years, the waste tyre problem has become so acute (Shulman, 2000) that there is an urgent need to find an optimum and effective way to use scrap tyres in asphalt mixtures.

The use of crumb rubber in asphalt paving is gaining more attention in many parts of the world as this material gives better mechanical and functional performance of the mixture as well as being a proficient way of dealing with this waste product (Epps, 1994). Crumb rubber modified (CRM) asphalt is a general type of modified asphalt that contains scrap tyre rubber. Modified asphalt paving products can be made with crumb rubber by several techniques, including a wet process and a dry process. In the wet process CRM binders are produced when finely ground crumb rubber (0.075 mm to 1.2 mm) is mixed with bitumen at elevated temperatures prior to mixing with the aggregate. Binder modification of this type is due to physical and compositional changes in an interaction process where the rubber particles swell in the bitumen by absorbing a percentage of the lighter fraction of the bitumen, to form a viscous gel. In the dry process, granulated or ground rubber and/or crumb rubber (0.4 to 10 mm) is used as a substitute for a small portion of the fine aggregate (typically 1-3 percent by mass of the total aggregate in the mixture). The rubber particles are blended with the aggregate prior to the addition of the bitumen.

1.2 PROBLEM STATEMENT

Considerable research into the wet process and the production of CRM binders have been undertaken in North America over the last ten years. The wet process has the advantage that the binder properties are better controlled although it needs special equipment to blend bitumen and rubber. On other hand, the dry process is a far less popular method due to increased costs for specially graded aggregate to incorporate the reclaimed tyre crumb, construction difficulties and most importantly poor reproducibility and premature failures in terms of cracking and ravelling of the asphalt road surfacing (Amirkhanian, 2000, Fager, 2001, Hunt, 2002). However, the dry process has the potential to consume larger quantities of recycled crumb rubber compared to the wet process resulting in greater environmental benefits. In addition, the production of CRM asphalt mixture by means of the dry process is logistically easier than the wet process and, therefore, the dry process is potentially available to a much larger market. However, research into the dry process is limited. The main assumption with the dry process is that rubber crumb is solely part of the aggregate and the reaction between bitumen and crumb rubber is negligible. Recent research at the University of Nottingham has showed that in the dry process, rubber crumb swells and reacts with bitumen at elevated temperatures and has an effect on the performance of the bitumen and, therefore, the asphalt mixture (Singleton, 2000). In addition, field trials have shown the performance of dry process CRM material used as a surface layer to be inconsistent and service life varies from two to twenty years. There are several reasons for this, uncontrollable crumb rubber sources, poor workmanship, flexible nature of the rubber particles, the adhesion with bitumen and the reaction described above. Tyre properties change with age and vary from manufacturer to manufacturer and this variability of scrap tyre source makes it even more difficult to control the consistency of the properties of the crumb rubber and consequently the properties of the mixture.

Some of the above-mentioned problems could be overcome by using CRM material as a flexible binder course layer where the direct impact of load is less compared to a surfacing. In this study, CRM mixtures are designed to be used as a flexible binder course layer. Extensive laboratory studies will be carried out to understand the constituent material and mixture properties. If the results show an improvement in the properties of the bituminous mixture, utilisation of such a material would be very rewarding in the UK. This would help work towards the reduction of waste scrap tyres and with the struggle to save the environment.

1.3 RESEARCH OBJECTIVES

The aim of this research project is to develop an understanding of the way in which recycled particulate rubber modifies the mechanical performance of the bituminous material and CRM asphalt mixture following the dry process. The programme has been divided in two main areas; firstly, an investigation on the constituent materials where the fundamental chemical and mechanical properties of the crumb rubber, binders, rubber-bitumen composites are assessed, and how the material behaves at elevated temperature is determined; secondly, an extensive laboratory investigation on CRM mixtures, designed by modifying a conventional UK Dense Bitumen Macadam (DBM), to study the mechanical properties in terms of stiffness modulus, fatigue and resistance to permanent deformation.

The overall aims of the project are to:

- Investigate the interaction between crumb rubber and bitumen using the dry process crumb rubber modified (CRM) technology as a function of crude source and penetration grade of bitumen.

- Investigate the chemical and rheological changes of residual bitumen following the rubber-bitumen interaction.

- Evaluate the mechanical properties of crumb rubber-bitumen composites containing different bitumens and subjected to varying degrees of short-term ageing using a repeated load triaxial apparatus at different temperatures and frequencies.

- Determine the mechanical properties of the dry process CRM asphalt mixtures in terms of stiffness, fatigue and rutting resistance using the Nottingham Asphalt Tester (NAT).

- Investigate the long-term performance and moisture sensitivity of dry process CRM asphalt mixtures using the NAT suite of tests following long-term oven ageing and moisture conditioning.

1.4 RESEARCH METHODOLOGY

To achieve the core research objectives listed above, the research methodology is divided into six tasks described below:

Task 1: Literature Review

Perform a comprehensive literature review on crumb rubber materials, rubber-bitumen interaction including literature on the design and application of crumb rubber modified mixtures in different parts of the world.

Task 2: Chemical Analysis of Rubber-Bitumen Interaction in the Dry Process

Perform a laboratory study to investigate the chemical processes that occur within the crumb rubber particles and the bitumen following the interaction of these two components during dry process CRM asphalt production (mixing, laying and compaction). Issues such as the migration of lighter fractions of the bitumen into rubber, the swelling potential of rubber particles and the compatibility of crumb rubber particles and various commercial bitumens will be investigated. In addition, the chemical and mechanical properties of the bitumen following rubber-bitumen interaction will also be evaluated.

Task 3: Mechanical Analysis of Rubber-Bitumen Composites

Perform triaxial tests on idealised rubber-bitumen composite specimens to investigate the effect on the mechanical and rheological properties of the composite due to rubber-bitumen interaction. The tests will be used to measure changes in mechanical behaviour of the rubber-bitumen composites as a result of the absorption of bitumen components into the rubber and the swelling potential of the rubber after subjecting the specimen to various temperature and time conditioning regimes.

Task 4: Mechanical Properties of CRM Asphalt Mixtures

Perform elemental laboratory tests on CRM asphalt mixtures to characterise the mechanical properties such as stiffness modulus, fatigue and resistance to permanent deformation. A number of CRM asphalt mixtures will be produced with various crumb rubber fractions in the mixtures and the mixtures then subjected to various short-term conditioning regimes prior to compaction to evaluate the effect of rubber-bitumen interaction on the mechanical properties.

Task 5: Durability of CRM Asphalt Mixtures

Conduct a durability study on different CRM mixtures to assess the moisture susceptibility and long-term ageing characteristics. Durability studies will include moisture conditioning and draft oven ageing on CRM mixtures before subjecting them to stiffness modulus, fatigue and resistance to permanent deformation tests.

Task 6: Analytical Modelling

Perform an analytical study of the relative influence of flexible crumb rubber mixtures on pavement life. Analytical studies will be carried out using the computer based multi-layer linear elastic analysis programme BISAR3 and the Nottingham

Design Method to evaluate the effect of various CRM mixtures on pavement performance in terms of fatigue and permanent deformation.

1.5 THESIS LAYOUT

The thesis presents the methodology, results, analysis and discussion obtained from an extensive laboratory investigation. The thesis is divided into ten chapters, following the methodology described above. An overview of the execution of the tasks and corresponding thesis chapters is illustrated in the flowchart in Figure 1.1 and a brief description of the contents of each chapter is presented below:

In **Chapter Two**, detailed literature on the constituent materials (rubber, bitumen), production of crumb rubber, application of crumb rubber in asphalt mixtures and finally a review of bituminous mixtures including testing methods to study mechanical properties are discussed.

In **Chapter Three**, a brief literature review on the rubber-bitumen interaction and a review of previous field and laboratory applications of CRM mixtures are presented.

In **Chapter Four**, experimental investigations on rubber-bitumen interaction including a brief explanation of the swelling test methodology developed for this investigation are discussed.

In **Chapter Five**, the chemical and rheological properties of the residual bitumen are presented to show that the interaction has altered the performance of the residual bitumen.

In **Chapter Six**, the first part describes the method to produce idealised rubber-bitumen composite specimens for triaxial testing. The second part presents an experimental investigation on different composite mixtures to evaluate the mechanical interaction.

In **Chapter Seven**, a mixture design methodology including production of crumb rubber modified asphalt mixtures is presented.

Chapter Eight presents the elemental testing on CRM mixtures to evaluate the mechanical properties in terms of stiffness modulus, fatigue and resistance to permanent deformation. Tests results from various CRM mixtures are presented to assess the effect of rubber content, short-term ageing of the loose mixture and compaction effort.

In **Chapter Nine**, an investigation on the durability of the CRM asphalt mixtures in terms of moisture sensitivity and long-term ageing properties is presented. The results are interpreted in terms of stiffness modulus, fatigue and resistance to permanent deformation and are compared with similar mixtures tested in the unconditioned state.

An analytical study using multi-layer elastic analysis has been presented in **Chapter Ten**, to investigate the relative influence of CRM mixtures on pavement life.

Conclusions and recommendations for future research are presented in **Chapter Eleven**.

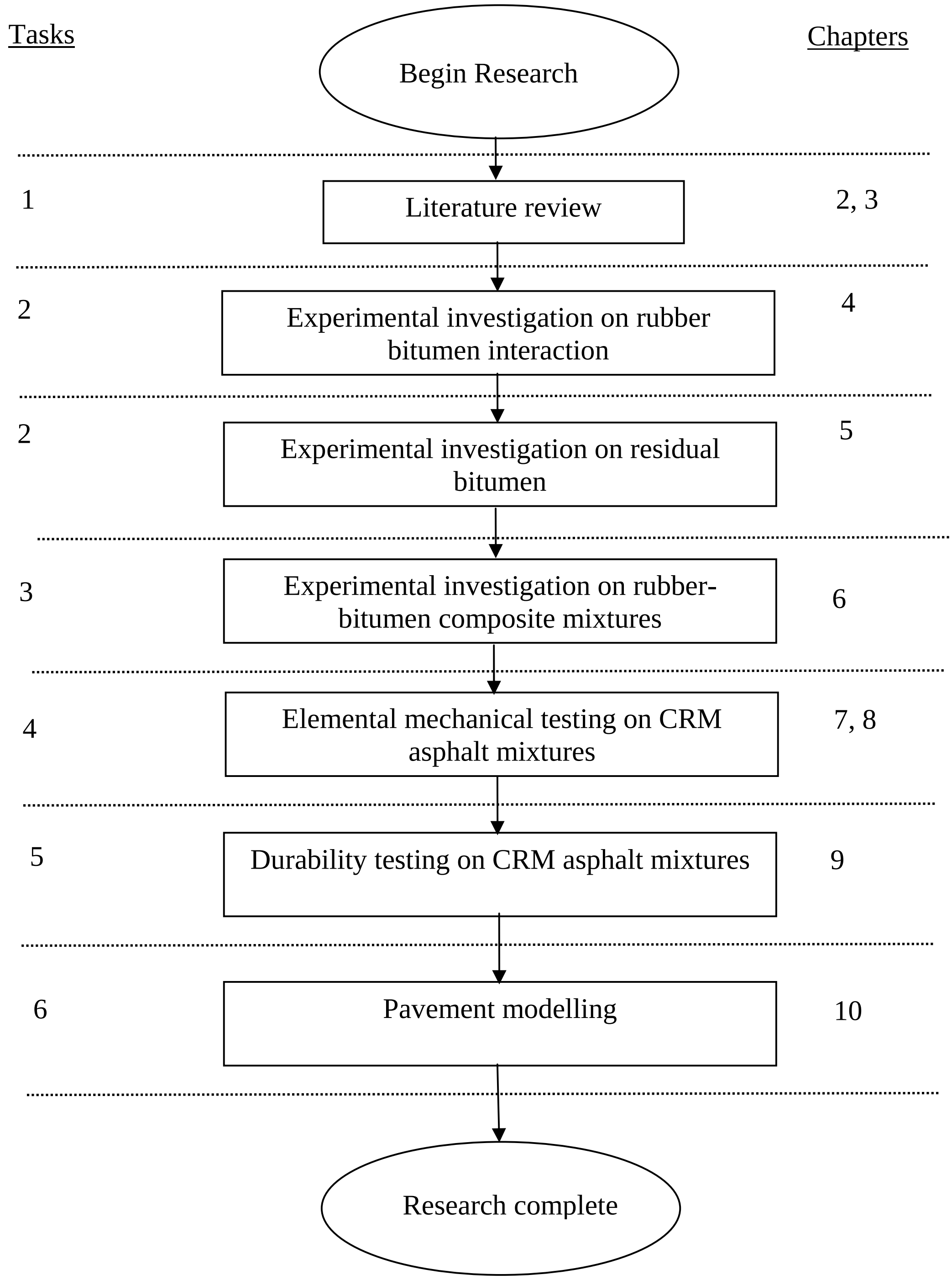


Figure 1.1: Crumb rubber modified asphalt mixture research methodology and thesis layout

CHAPTER 2

Constituent Materials

2.1 BITUMEN

Bitumen is manufactured from crude oil that originates from the remains of marine organisms and vegetable matter deposited on the ocean bed (Whiteoak, 1990). Over millions of years, the matter is accumulated and through the immense weight of the upper layers the matter in the lower layer is compressed. Combined with heat from the Earth's crust, the matter forms crude oil that is trapped by impermeable rock forming large underground reservoirs. The crude oil can sometimes rise through faults in the layers above, coming to the ground surface. Most crude oil is now extracted from underground by drilling (Whiteoak, 1991). There are many sources of crude oil but only a few of these produce a suitable raw material for bitumen.

British Standard 3690: Part2 (1989) defines bitumen as a viscous liquid, or a solid consisting essentially of hydrocarbons and their derivatives, which is soluble in trichlorothylene and is substantially non-volatile and softens gradually when heated. It is obtained by a refinery process from crude petroleum oil, as well as found as a natural deposit or as a component of naturally occurring asphalt combined with mineral matter.

2.1.1 Bitumen Composition and Structure

Bitumen is a complex mixture of organic molecules. Both the chemical (constituent) and the physical (structural) part of the bitumen comprise mainly hydrocarbons with minor amounts of functional groups such as oxygen, nitrogen and sulphur. As bitumen is extracted from crude oil, which has variable composition according to its origin, the precise breakdown of hydrocarbon groups in bitumen is difficult to determine. However, elementary analysis of bitumen manufactured from a variety of crude sources show that most bitumens contain:

- Carbon: 82-88%
- Hydrogen: 8-11%
- Sulphur: 0-6%
- Oxygen: 0-1.5%
- Nitrogen: 0-1%

The precise composition of bitumen varies according to the crude source. Although the chemical composition is very complex, it is possible to separate bitumen into four main chemical compositions (Whiteoak, 1990; Airey, 1997). These are:

- Asphaltenes
- Resins
- Aromatics
- Saturates

The four groups are not well defined and there is inevitably some overlap between the groups. A schematic representation of the bitumen structure is presented in Figure 2.1.

Asphaltenes are black or brown amorphous (without shape) solids. They are highly polar, complex materials of high molecular weight (between 1,000 and 100,000). Within a medium they have a tendency to associate together to form micelles with a molecular weight between 20,000 and 1,000,000. Asphaltenes typically constitute 5% to 25% of the bitumen. The molecular weight relates to the size of each molecule, so the higher the molecular weight, the larger the molecules.

The asphaltenes content has a considerable effect on the rheological characteristics of bitumen. Increasing the asphaltenes content produces harder bitumen with a lower penetration, higher softening point and higher viscosity. The association of asphaltenes is not fixed; on heating the gel structure (Figure 2.1) of the micelles is broken down and reformed on cooling. During long-term heating the asphaltenes micelles may break down, therefore, it is not unusual for the molecular weight of bitumen to decrease after heating. In short, asphaltenes define the stiffness and rigidity of the bitumen constituent.

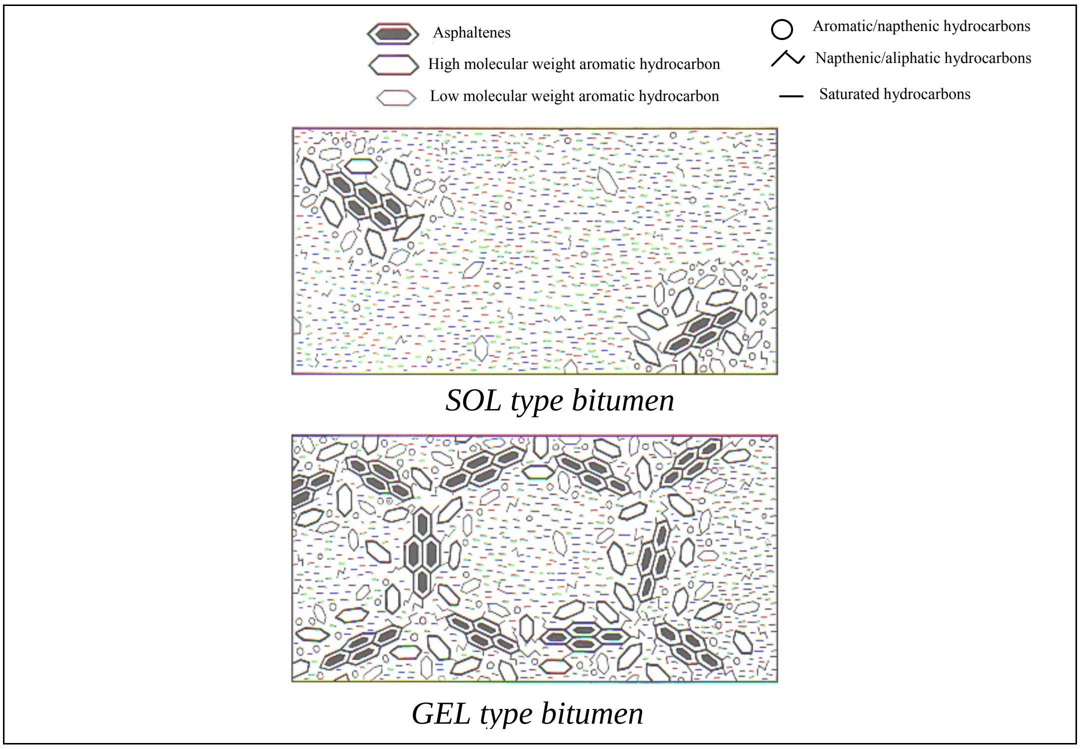


Figure 2.1: Schematic representation of typical bitumen structure (Whiteoak, 1990)

Resins are black or brown solid or semi-solid highly polar molecules. The high polarity makes the resins very adhesive. The molecular weight ranges from 500 to 50,000. The resins part of the bitumen acts as a peptising agent for the asphaltenes, therefore an increase in resins results in a solution (sol) structure whereas a reduction forms a gelatinous (gel) structure in the bitumen (Figure 2.1). Resins work as stabilisers, which hold everything together in the bitumen (Kerbs and Walker, 1971).

Aromatics have the lowest molecular weight and form the major proportion of the bitumen (40-65%). They have a very low polarity and form a dark brown viscous liquid that acts as a dispersion medium for the asphaltenes in the bitumen and have a molecular weight in the range of 300 to 20,000. They give the adhesive properties of the bitumen.

Saturates are similar to oil and give the fluid properties of the bitumen. They have the lowest molecular weight (similar to aromatics) and are straw or white in colour and form between 5-20% of the bitumen.

2.1.2 Conventional Physical Properties

There are several conventional physical property tests to evaluate the quality and consistency of manufactured bitumens. These tests are standardised in many specifications, e.g., British specifications, ASTM and European standards. However, all these methods are practically identical with negligible differences. Consistency tests are one of the main types of conventional tests that describe the degree of fluidity of bitumen at any particular temperature.

2.1.2.1 Penetration

The penetration test is an empirical method to measure the consistency of the bitumen. The test method is described in BS 2000, Part 49, 1983, in the Institute of Petroleum (IP) as IP 49/83 and in ASTM D5. According to BS 2000 Part 49, penetration is defined, as the distance a standard needle loaded with a 100g weight will penetrate into a bitumen sample for 5 seconds. Usually penetration is measured at 25^0C, which also approximates the average service temperature of the hot mix asphalt (HMA) pavements. However, other temperatures with different needle loads and penetration times may be used as well. In addition, Gershkoff (1991) developed a relationship (Equation 2.1) between empirical penetration tests results and the measured fundamental stiffness of the bitumen tested at the same temperature (25^0C) and loading frequency (2.5Hz).

$$Log(Penetration) = 4.55 - 0.52 Log\,|G^*|_{0.4} \qquad \textbf{(2.1)}$$

Where;

$|G^*|_{0.4}$ = Complex modulus at 25^0C with loading frequency 2.5Hz

2.1.2.2 Softening Point

The softening point test is also an empirical method to determine the consistency of a penetration or oxidised bitumen. In this test two steel balls are placed on two discs of bitumen contained within metal rings and these are raised in temperature at a constant rate (5^0C/min) in a water bath (bitumen with softening point 80^0C or below) or in glycerol (bitumen with softening point greater than 80^0C). The softening point is the temperature (^{0}C) at which the bitumen softens enough to allow the balls enveloped in bitumen to fall a distance of 25 mm into the bottom plate. In short, this test method measures a temperature at which the bitumen phase changes from semi-solid to liquid. The test method is described in BS 2000 Part 58, 1983, IP 58/83 and ASTM D36.

2.1.2.3 Viscosity

Viscosity, resistance to flow, is a fundamental characteristic of bitumen as it determines how the material will behave at a given temperature and over a range of temperatures. Viscosity (η) is related to stress and strain rate and is determined by;

$$\eta = \frac{Stress}{Strain\ rate} \qquad \textbf{(2.2)}$$

If a material behaviour is independent of the rate of shear then it is called Newtonian behaviour. Bitumen usually exhibits Newtonian behaviour at high temperature (60^0C) and sometimes at temperatures as low as 25^0C. At these low temperatures, however, many types of bitumen do not exhibit Newtonian flow but show shear-thinning (pseudo plastic) behaviour. Thus for viscosity measurements on bitumens to be meaningful, the rate of strain must be known.

Both absolute and kinematic viscosities are important for specification and comparative purposes. Specifications are based on absolute viscosity ranges at 60^0C and minimum kinematic viscosity at 135^0C. A minimum penetration at 25^0C is also included in most specifications. The absolute viscosity is measured by 'pulling' the bitumen through the viscometer with a vacuum, whereas for the kinematic viscosity, the bitumen flows under its own weight. Kinematic viscosity is related to dynamic viscosity of the material and is measured using a capillary tube viscometer. The basic principle of a capillary tube viscometer is to measure the time required for a fixed quantity of material to flow through a standard orifice.

In addition, sliding plate and rotational viscometers are used for determining viscosity at temperatures below 60^0C. In this project, a rotational viscometer was used according to ASTM D4402–87. This test measures the apparent viscosity of bitumen from 38^0C to 260^0C and uses a temperature controlled thermal chamber for maintaining the test temperature.

All the consistency tests described above cannot totally describe the overall behaviour of bitumen and relate it to pavement performance because of their empirical nature. In addition, these test methods do not give an indication of the viscoelastic nature of bitumen at any particular test temperature and do not have the flexibility to be undertaken at different loading modes (Bahia and Anderson, 1995). To overcome these problems more fundamental testing methods have been introduced which provide sound representation of the fundamental rheological properties under different temperatures and loading conditions.

2.1.2.4 Bitumen Rheology

The study of bitumen rheology is an important phenomenon to characterise the dynamic mechanical behaviour of binders. Bitumen is a thermoplastic, viscoelastic material. Viscoelasticity is a rate dependent material characterisation that includes a viscous contribution to the elastic straining. Which means, bitumen, as a viscoelastic material, behaves as glass-like elastic solid at low temperature and/or during high loading frequencies and as viscous fluid at high temperatures and/or low loading frequencies. The thermal and mechanical deformation of bitumen can be defined by its stress-strain-time-temperature response (Airey, 1997).

2.1.3 Dynamic Mechanical Analysis

Viscoelastic materials have high mechanical damping and mechanical vibrations do not build up easily at natural frequencies and high temperatures (McCrum *et al*, 1999). It is normal practise to use oscillatory type testing for doing dynamic mechanical analysis (DMA) to investigate the rheology of a viscoelastic material, like bitumen. DMA allows the viscous and elastic nature of the bitumen to be determined over a wide range of temperatures and loading times (Goodrich, 1991). In the dynamic test (Figure 2.2), the material is subjected to an oscillatory shear strain of angular frequency ω,

$$\gamma = \gamma_0 \sin \omega t \qquad \textbf{(2.3)}$$

Where,

ω = $2\pi f$

f = the frequency of the sinusoidal strain (Hz)

γ_0 = maximum strain amplitude

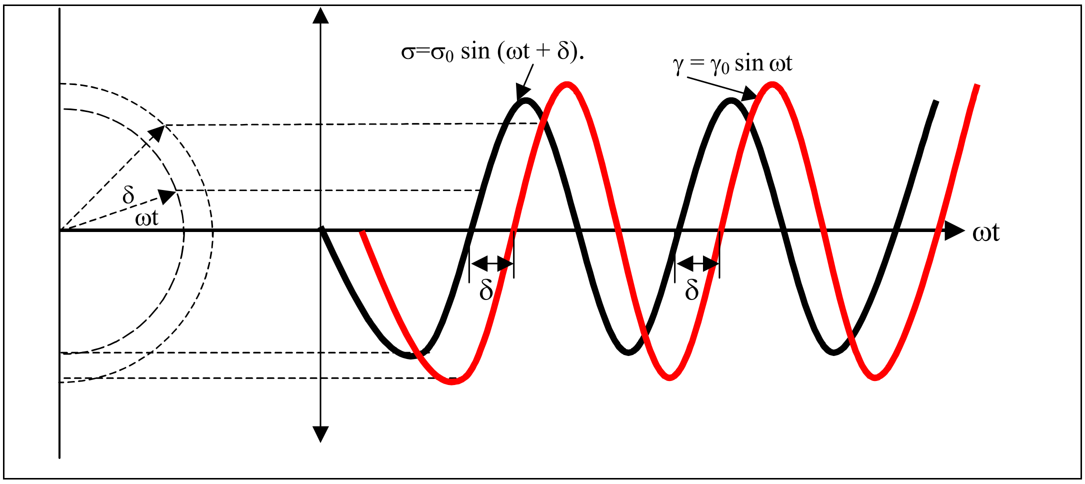


Figure 2.2: Stress-strain response of a viscoelastic material

For linear viscoelastic material the stress response is also sinusoidal, but is out of phase as shown in equation 2.4.

$$\sigma = \sigma_0 \sin(\omega t + \delta) \qquad \textbf{(2.4)}$$

Where,

σ_0 = maximum stress

δ = phase angle

Expanding equation 2.4,

$$\sigma = (\sigma_0 \cos\delta)\sin\omega t + (\sigma_0 \sin\delta)\cos\omega t \qquad \textbf{(2.5)}$$

the stress equation consists of two components, in phase with the strain (σ_0 cosδ) and 90^0 out of phase (σ_0 sinδ). The relationship between stress and strain can be defined by writing,

$$\sigma = \gamma[G^{'} \sin\omega t + G'' \cos\omega t] \qquad \textbf{(2.6)}$$

in which (from equation 2.5)

$$G' = \frac{\sigma_o}{\gamma_o}\cos\delta \qquad \textbf{(2.7)}$$

$$G'' = \frac{\sigma_o}{\gamma_o} \sin\delta \tag{2.8}$$

Thus the component of the stress $G' \gamma_0$ is in phase with the oscillatory strain; the component $G''\gamma_0$ is 90^0 out of phase.

2.1.4 Dynamic Shear Rheometry

The Dynamic Shear Rheometer (DSR) is a dynamic oscillatory test apparatus that can be used to describe the linear viscoelastic properties of bitumen over a range of temperatures and frequencies. It applies a sinusoidal shear strain to a sample of bitumen sandwiched between two parallel disks and is shown schematically in Figure 2.3 and as a picture of a modern DSR machine in Figure 2.4. The amplitude of the stress is measured by determining the torque transmitted through the sample in response to the applied strain. As the DSR only takes two measurements, namely torque (τ) and angular rotation (θ), the remaining mechanical properties are calculated by using these two parameters.

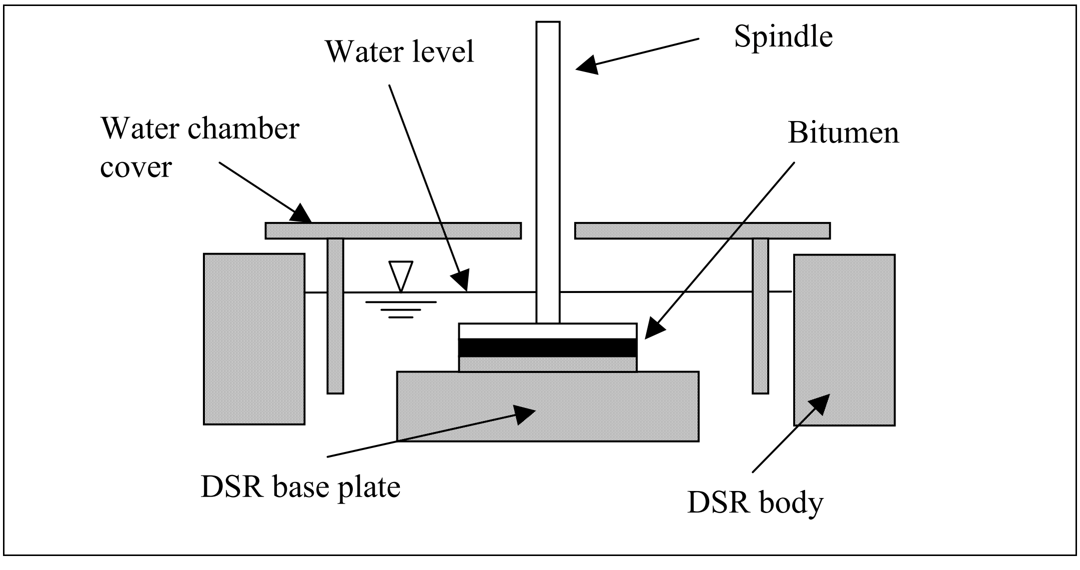


Figure 2.3: Schematic of DSR testing arrangement

Figure 2.4: Bohlin Gemini dynamic shear rheometer

Two basic equations are used to calculate stress and strain parameters (Airey *et al.*, 1998, Grandiner and Newcomb, 1995):

$$\tau = \frac{2T}{\pi r^3} \tag{2.9}$$

Where,

τ	=	maximum shear stress (N/ mm^2)
T	=	torque (N.m)
r	=	radius of the parallel disks (mm) (Figure 2.5)

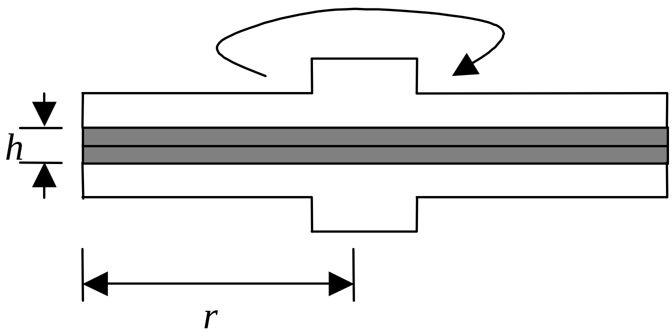


Figure 2.5: Testing geometry of DSR

$$\gamma = \frac{\theta r}{h} \tag{2.10}$$

Where,

γ	=	shear strain
θ	=	deflection angle (radians)
h	=	gap between parallel disks (mm) (Figure 2.5)

Therefore, from the calculation of strain and stress, the absolute complex modulus can be calculated according to the following formula:

$$G^* = \frac{\tau_{\max}}{\gamma_{\max}} \tag{2.11}$$

The shear stress and strain in equations 2.9 and 2.10 are dependent on the radius of the parallel disks and vary in magnitude from the centre to the perimeter of the disk. The shear stress, shear strain and complex modulus are calculated for the maximum value of radius. The phase angle, δ, is measured automatically using the instrument by accurately determining the sinusoidal waveforms of the strain and torque. The edge of the sample should be curved to get better results. Various parallel disk sizes can be used during DSR testing and the size of the disk that should be used to test the bitumen decreases as the expected stiffness of the bitumen increases. In other words, the lower the testing temperature, the smaller the diameter of the disk that needs to be used to accurately determine the dynamic properties of the bitumen.

The applied strain during DSR testing must be kept small to ensure that the test remains in the linear viscoelastic region. A linear region may be defined at small strains where the shear modulus is relatively independent of shear strain. This region will vary with the magnitude of the complex modulus and, therefore, the strains should be kept small at low temperatures and increased at high temperatures. The linear region can be found by plotting complex modulus versus shear strain from stress or strain sweep tests. According to Strategic Highway Research Program (SHRP), the linear region can be defined as the point where complex modulus decreases to 95% of its maximum value as shown in Figure 2.6 (Peterson *et al*, 1994).

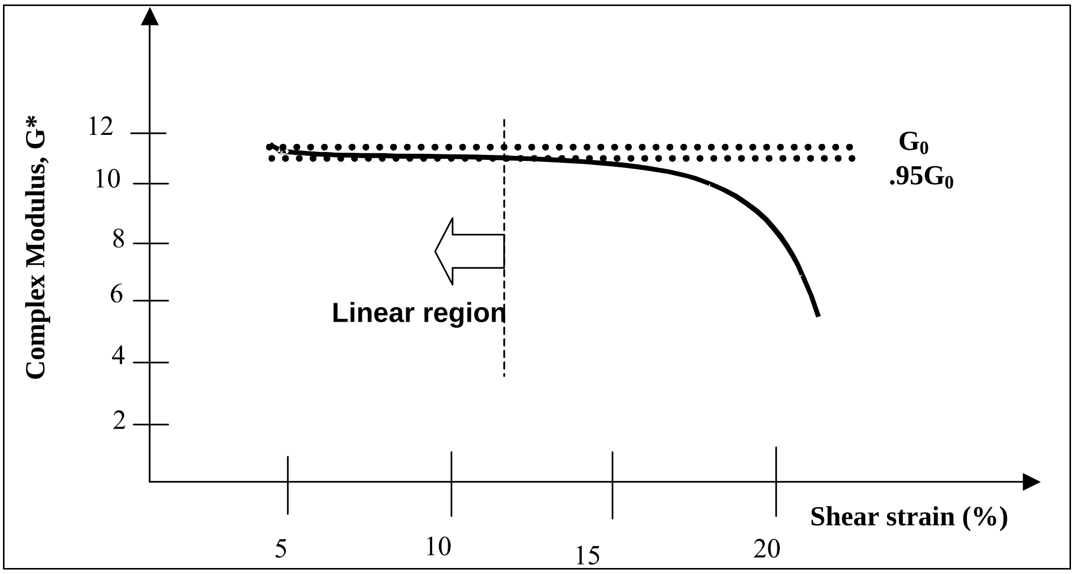


Figure 2.6: Strain sweep to determine linear region

2.1.4.1 Plate Diameter

The testing configuration of the DSR consists of a number of different parallel plate and cone/plate geometries to measure a wide range of bitumen stiffness. Different disk sizes and suggested testing temperatures have been proposed by various researchers based on different sample types. But the most widely used method is suggested by SHRP Project A-002A (Anderson *et al*., 1994), as presented in Table 2.1.

Table 2.1 SHRP suggested disk diameters for DSR rheology testing (Anderson et al., 1994)

Disk diameter	**Test temperature range**	**Typical G* range**
8 mm	0^0C to $+40^0$C	10^5 Pa to 10^7 Pa
25 mm	$+40^0$C to $+80^0$C	10^3 Pa to 10^5 Pa
40 mm	> 80^0C	$<10^3$ Pa

The proper choice of disk (plate) size or specimen should be dictated by the stiffness of the test specimen, rather than temperature. For instance, at low temperatures, large disks measure stresses lower than the true value (Collins *et al*., 1991) while in contrast reducing the plate diameter improves the results although it does not appear possible to measure the limiting elastic stiffness of 10^9 Pa using a DSR (Carswell *et al*, 1997).

2.1.4.2 Gap Width

Goodrich (1988) and Collins *et al*.(1991) studied dynamic oscillatory tests on thick bitumen samples, 1 to 2.5 mm and 1.5 mm to 2.2 mm respectively. However, the gap height between the two parallel disks is generally in the range between 0.5 to 1.0 mm and it is also recommended that when the complex shear modulus of the bitumen is greater than approximately 30MPa, parallel plate geometry should not be used as the compliance of the rheometer can be sufficient to cause errors in the measurements (Peterson *et al*, 1994). According to SHRP, the following guidelines should be used,

- Bending Beam Rheometer (BBR) or torsional bar geometry when G*> 30 MPa
- 8 mm parallel plates with a 2 mm gap when 0.1MPa <G* < 30 MPa
- 25 mm parallel plates with a 1 mm gap when 1kPa <G* < 100 kPa
- 50 mm parallel plates when G*< 1kPa

Although these recommended guidelines provide a useful indication of plate and gap geometry, care should be taken when using them over wide frequency sweeps and for different binders. This is particularly relevant at the transitions between the different sample geometries and, therefore, it is recommended that there should be an overlap of rheological testing with two disk and gap configurations being used at the transition points.

2.1.4.3 Calibration

Regular calibration of temperature and torque is essential to maintain reasonable repeatability and reproducibility of the rheological data. The circulating fluid (water) or air from the temperature control unit should be constant during testing as small variations in temperature significantly change the results. Most DSR manufacturers use standard fluid to carry out calibration to confirm constant temperature, displacement and torque (Carswell *et al*, 1997). The problem with using this standard liquid is that it has low viscosity in comparison with bitumen, therefore, it can be misleading when measuring high bitumen stiffness.

2.1.5 Rheological Data Representation

Dynamic shear tests can be performed at different temperatures and frequencies to measure stiffness and phase angle. There are different techniques of data presentation available to represent the results graphically. However, the most commonly used representative methods are discussed below.

2.1.5.1 Isochronal Plots

An isochronal plot is a curve, which represents the behaviour of a system at a constant frequency or loading period. In the dynamic test, the data can be presented over a range of temperatures at a given frequency. This technique has distinctive advantages as it gives a clear idea of the mechanical properties (complex modulus, phase angle) and temperature susceptibility in a single plot with different temperatures. A typical isochronal plot is presented in Figure 2.7.

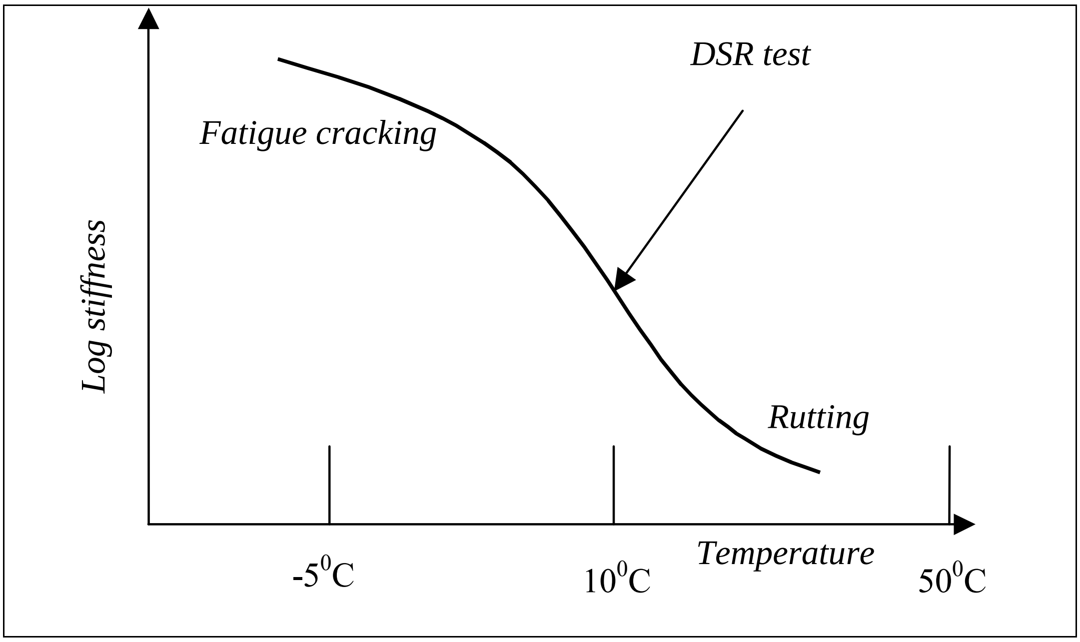


Figure 2.7: Typical isochronal plot

2.1.5.2 Isothermal Plot

An isothermal plot or isotherm is an equation, or a curve on a graph, representing the behaviour of a system at a constant temperature. In this type of plot, data at a given temperature is plotted over a range of frequencies or loading durations. The curve can be used to compare different viscoelastic functions at different loading times at a constant temperature (see Figure 2.8). In addition, this type of plot can be used to study the time dependency of the material. As DSR testing is only performed over a limited frequency range, it is impossible to represent a wide range of rheological properties in an isothermal plot. Therefore, master curves are used to extend the data over a wider range of loading times using the time temperature superposition principle.

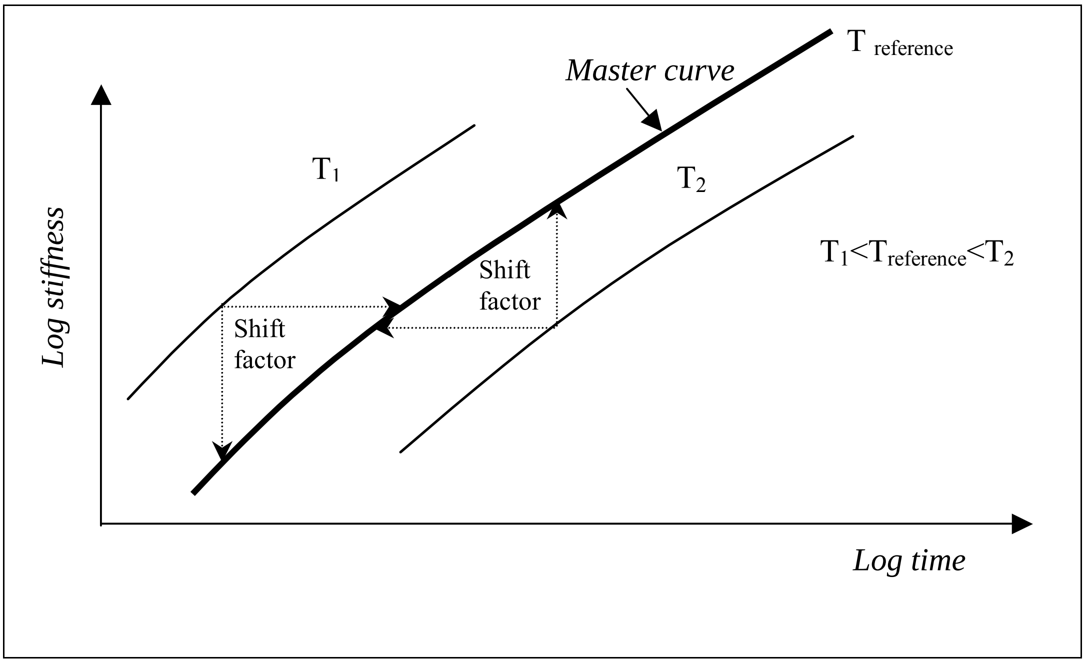


Figure 2.8: Typical isothermal plot and generated master curve

2.1.5.3 Time Temperature Superposition Principle (TTSP)

TTSP used in analysing dynamic mechanical data involves the construction of master curves. Work done by various researchers, have found that there is an interrelationship between temperature and frequency (or temperature and loading time) which, through shifting factors (described below), can bring measurements done at different temperatures to fit one overall continuous curve at a reduced frequency or time scale. This continuous curve represents the binder behaviour at a given temperature for a large range of frequencies. The principle that is used to relate the equivalency between time and temperature and thereby produce the master curve is known as the time-temperature superposition principle or the method of reduced variables (Anderson *et al*, 1991).

2.1.5.4 Master Curves

Master curves are used to present the extended data (mechanical properties) over a wide range of loading times and frequencies in one graph (several years of loading time). In their simplest form, master curves are produced by manually shifting modulus versus frequency plots (isotherms) at different temperatures along the logarithmic frequency axis to produce a smooth curve (Anderson *et al*, 1991, Airey, 1997). A numerical factor, called the shift factor (Williams *et al*, 1955, Dickinson and Witt, 1974), is used to shift the data at a specific temperature to the reference temperature (Figure 2.8). Breaks in the smoothness of the master curve indicate the presence of structural changes with temperature within the bitumen, as would be found for waxy bitumen, highly structured 'GEL' type bitumen and polymer-modified bitumen.

2.2 RUBBER AND TYRES

2.2.1 Molecular Structure of Rubber

There are two types of rubber; natural and synthetic. Natural rubber latex is obtained from the rubber tree called Hevea braziliensis. The primary composition of the raw rubber molecule is a long straight-chain isoprene hydrocarbon**.** The physical appearance of this hydrocarbon is of a spongy, flocculent nature. At temperatures below 100^0C this spongy rubber becomes stiff, hard whereas when warmed above 100^0C, it becomes flexible, soft and transparent (Blow, 1971).

Synthetic rubbers are made from petroleum products and other minerals and produced in two main stages: first the production of monomers (long molecules consisting of many small units), then polymerisation to form a rubber. There are various types of synthetic rubber available for different applications. Some of them are: Styrene-Butadiene Rubber (SBR, used in bitumen, tyres etc); Silicon rubbers (used in gaskets, seals etc); Fluorocarbon rubber (resistant to heat and chemical attack); and Epichlorohydrin rubber (jackets, hose, cable, packing etc) (Blow, 1971).

The functionality of the rubber depends on how the molecules are arranged. There are three types of molecular arrangements; linear, side branched and cross-linked.

2.2.1.1 Linear and Side Branched Polymers

The mechanical properties of this type of polymer are dependent on the length and shape (side branch) of the molecule. Both the linear and side branched polymers can be reversibly heated to melt and then cooled to crystallise time and time again. On melting they flow as a liquid and are therefore called thermoplastic. The number of the side branches can be varied by changing the polymerisation conditions. Even small variations in the number of side branches can cause appreciable changes in elastic modulus, creep resistance and toughness. Microwaveable food containers, Dacron carpets and Kevlar ropes are examples of products made with linear polymers. Soft, flexible shampoo bottles and milk jugs are examples of products generally made using branched polymers.

2.2.1.2 Cross-Linked Polymers

In cross-linked polymers, the chains are joined chemically at the tie points with cross-linking agents and formed into one simple giant molecular network. Many cross-linked networks are produced by chemical reactions triggered by heating. After heating, the network gets permanent shape and this state is called "Thermoset". Cross-linked polymers do not flow when heated. Tyres and bowling balls are two examples of products composed of cross-linked polymers. The mechanical behaviour of an elastomer depends strongly on cross-link density, which is shown schematically in Figure 2.9 (Hamed, 1992). It shows that the modulus and hardness increase monotonically with cross-link density and the network becomes more elastic. Fracture properties such as tear and tensile strength pass through a maximum as cross-linking is increased.

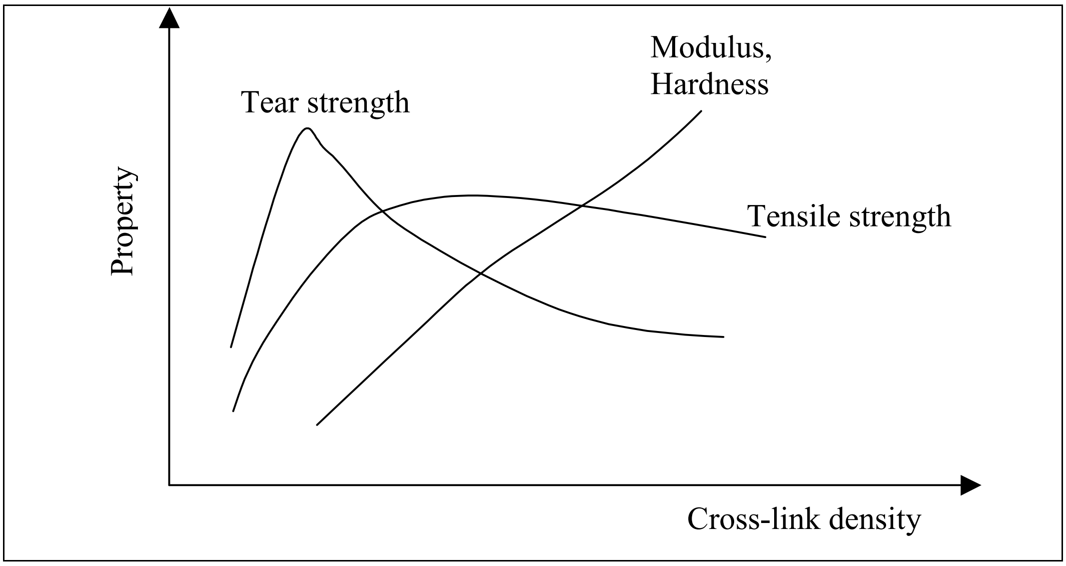


Figure 2.9: Effect of cross-link density on some mechanical properties of rubber (Hamed, 1992)

2.2.2 Processing of Rubber

2.2.2.1 Vulcanisation

Vulcanisation is the curing process of rubber, which transforms the raw rubber into a strong, elastic and rubbery hard state. There are two types of vulcanisation processes: hot (mould cured) and cold (pre-cure system). The hot process is used for the majority of rubber goods, including tyres. Cold vulcanisation is used to produce soft, thin rubber products such as surgical gloves or sheeting. Table 2.2 shows the effect of temperature on rubber.

Table 2.2: Effect of Different Temperatures on Rubber

Base structure	Hard transparent and solid
-10°C	Brittle and opaque
+20 °C	Soft, resilient and translucent
+50 °C	Plastic and sticky
120 °C-160 °C	Vulcanised when agents e.g., sulphur are added
~182 °C	Break down as in the masticator
200 °C	Decomposes

Natural rubber is insoluble with water, alkali and weak acids, but it is soluble in benzene, gasoline, chlorinated hydrocarbons and carbon bisulphate (Blow, 1971). While it is easily oxidised by chemical oxidising agents, atmospheric oxygen produces a very slow reaction. The most common vulcanisation is through sulphur. The proportion of sulphur agents to rubber varies from a ratio of 1:40 for soft rubber goods, to as much as 1:1 for hard rubber (Shulman, 2000). The sulphur is ground and mixed with the rubber at the same time as the other dry ingredients during the compounding process. When rubber is heated with sulphur to a temperature between 120^{o}C and 160^{o}C, it becomes vulcanised by combining the sulphur agents with the rubber molecules and produces a cross-linking network, which makes the rubber stronger and more durable and contributes to improve tyre wear and durability. The reactions of rubber to temperature extremes are an important factor in their applicability that produces improved strength and elasticity as well as greater resistance to changes in temperature, impermeability to gases, resistance to abrasion, chemical action, heat, and electricity. Vulcanised rubber exhibits high frictional resistance on dry surfaces and low frictional resistance on water-wet surfaces, it has good abrasive resistance, flexibility, elasticity and electric resistance (Blow, 1971). Although vulcanisation converts soft rubber into a hard, usable stage, it is essential to add certain chemicals and additives to make it readily usable in commercial applications. This formulation process is called compounding.

2.2.2.1 Compounding

Compounding is the process by which a number of ingredients are added to modify and improve the physical properties of rubber. The first reason for compounding is to incorporate the ingredients and ancillary substances necessary for vulcanisation. The second is to adjust the hardness and modulus of the vulcanised product to meet the end requirement. Different techniques are available to do compounding using the same fundamental constituents (Stern, 1954), such as base polymer, cross-linking agent, accelerator for the cross linking reaction, reinforcing filler (carbon black, mineral), processing aids (softeners, plasticisers, lubricants), diluents (organic materials, extending oils), colouring materials (organic or inorganic), specific additives (blowing agent, fibrous materials). When designing a mixture formulation for a specific end use, it is necessary to take account not only of those vulcanisate properties essential to satisfy service requirements but also the costs of the raw materials involved and the production processes by which these will be transformed into final products.

The rubber compound used in rubber tyres is a complex mixture. In the compounding process, a number of ingredients are added to modify and improve the physical properties of the rubber and to make it more readily useable (modify the hardness, strength, toughness and to increase resistance to abrasion in oil, oxygen, chemical solvents and heat) for various applications. It is important to note that some of the ingredients still remain when the tyres are recycled at the end of their life. As an example, the stabilisers which provide resistance to cracking and degrading of the tyre, also prolong the life of roads, sports and safety surfaces etc; the pigments which produce uniform colour in the tyre also contribute to the consistent and long term colour of roads which utilise post consumer tyre materials.

2.2.3 Tyres

2.2.3.1 Production

A tyre is made up of three main materials; elastomeric (rubber) compound, fabric and steel. The fabric and steel form the structural skeleton of the tyre with the rubber forming the “flesh” of the tyre in the tread, side wall, apexes, liner and shoulder wedge. The tyre skeleton consists of beads made of steel or fabric depending on the tyre application, which form the ‘backbone’ in the toe of the tyre (Figure 2.10). The beads are designed to have low extensibility and provide reinforcement for the rubber tyre. The tyre has a series of reinforcing cords or belts that extend from bead to bead transversely over the tyre.

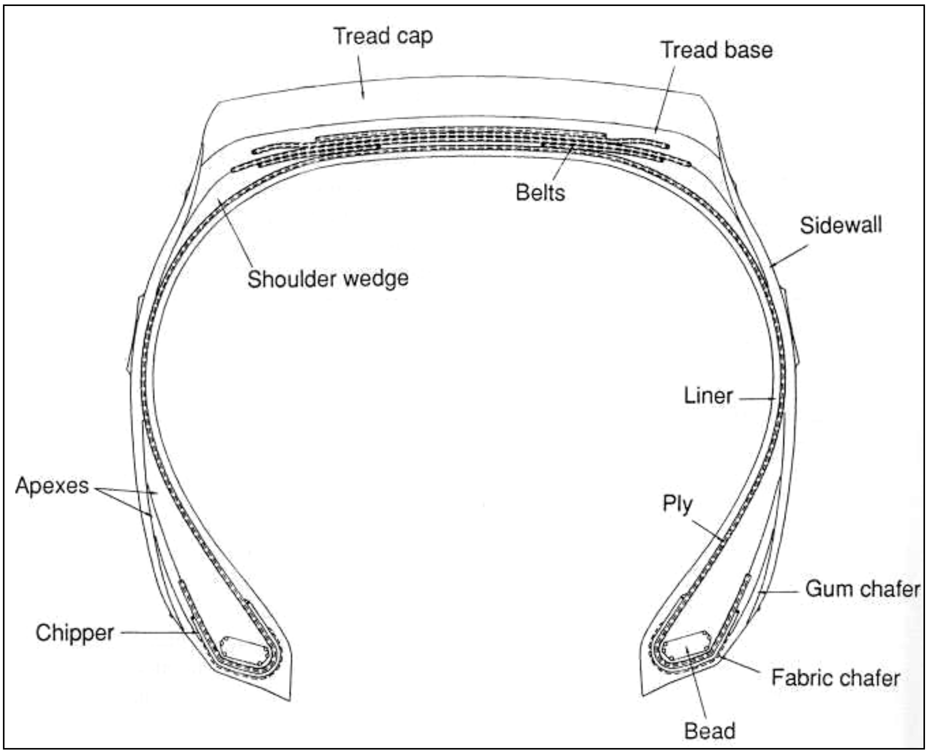


Figure 2.10: Schematic diagram of a typical truck tyre cross-section (Merk et al, 1994)

The belts are made of nylon fabric or steel but more commonly both types are used. The rubber treads then cover the belts providing the contact area for the tyre on the pavement. The objective of the skeleton is to reinforce the tyre to allow it to perform well without excessively deforming.

Tyre construction is a complex process of compounding to combine the elastomer (rubber), process the steel and fabric with the rubber, extrude the treads, sidewalls and then cure the tyre under heat and pressure. The inherent characteristics of tyres are the same worldwide. They include the resistance to mould, mildew, heat and humidity, retardation of bacterial development, resistance to sunlight, ultra-violet rays, some oils, many solvents, acids and other chemicals. Other physical characteristics include their non-bio-degradability, non-toxicity, weight, shape and elasticity. However, many of the characteristics, which are beneficial during their on-road life as consumer products, are disadvantageous in their post-consumer life and can create problems for collection, storage and/or disposal. Modern tyres have extremely high load bearing capacity up to fifty times of its own weight. The compressed air within the tyre carries 90% of the load. The complex structure of the shell or casing of the tyre is designed to carry the remaining 10%.

2.2.3.2 Composition

The composition of tyres consists of four main ingredients: rubber, carbon black, metal and textiles. The remaining materials are additives, which facilitate compounding, and vulcanisation. Table 2.3 is a summarised version of general tyre composition in cars and truck tyres in the EU (Shulman, 2000).

Table 2.3: Comparison of passenger car and truck tyres in the EU

Material	Car	Truck
Rubber/Elastomers	48%	45%
Carbon Black	22%	22%
Metal	15%	25%
Textile	5%	--
Zinc oxide	1%	2%
Sulphur	1%	1%
Additives	8%	5%

In general, tyres are composed of natural and synthetic rubber. The proportion varies according to the size and use of the tyre. The generally accepted rule of thumb is that the larger the tyre and the more rugged its intended use, the greater will be the ratio of natural to synthetic rubber.

The second most important component of a tyre is carbon black. This is not a generic product, which means that wide ranges of specific grades of carbon black are used depending upon the compounding formula used by the individual manufacturer. Carbon black is mainly used to enhance rigidity in tyre treads to improve traction, control abrasion and reduce aquaplaning; in sidewalls to add flexibility and to reduce heat build up (HBU) (Shulman, 2000). The particle size of the carbon black, as defined by its specific surface area and structure, impacts upon its integration and utilisation in compounding.

The third largest component is steel, mainly high grade steel. This provides rigidity, and strength as well as flexibility to the casing. New, higher strength metals are being tested by tyre manufacturers, some which are said to resist rusting as well as deterioration, which could impact upon the way that the tyre is recycled.

The most common traditional textiles used in rubber are nylon, rayon and polyester. In recent years, a range of new textiles, primarily amarid, which is an ultra-light weight material, have been substituted for more traditional materials, primarily in the more expensive tyres.

2.2.3.3 Tyre Recycling

The life cycle of a consumer product is defined as the time span of the product serving the purpose for which it was created. The life span for a tyre is approximately 5-7 years during which time a tyre can be retreaded. It comprises three principal periods: new, continued use (continued chain of utility), and consignment to a waste treatment system (end of tyre life). A post consumer tyre, which may or may not have a structurally sound casing or residual tread depth suitable for further road use, will be discarded and/or consigned to another use, such as scrap tyres in road construction. The brief life cycle of a tyre is shown in Figure 2.11.

Once the tyre is permanently removed from a vehicle, it is defined as waste (scrap tyre). A scrap tyre can be useable in different forms, such as a whole tyre, a slit tyre, a shredded or chipped tyre, as ground rubber or as a crumb rubber product. In the following paragraphs a brief description of the use of scrap tyres will be outlined.

Whole tyre: Typical weights of scrapped automobile (car and truck/bus) tyres are presented in the Table 2.4 including amount of recoverable rubber and percentage of natural and synthetic rubber. Although the majority of truck tyres are steel-belted radial, there are still a number of bias ply truck tyres, which contain either nylon or polyester belt material. Scrap tyres have a heating value ranging from 28000kJ/kg to 35000 kJ/kg, which is the same as coal, and therefore, have been widely used as a cement-making fuel worldwide for the last ten years (Shulman, 2000).

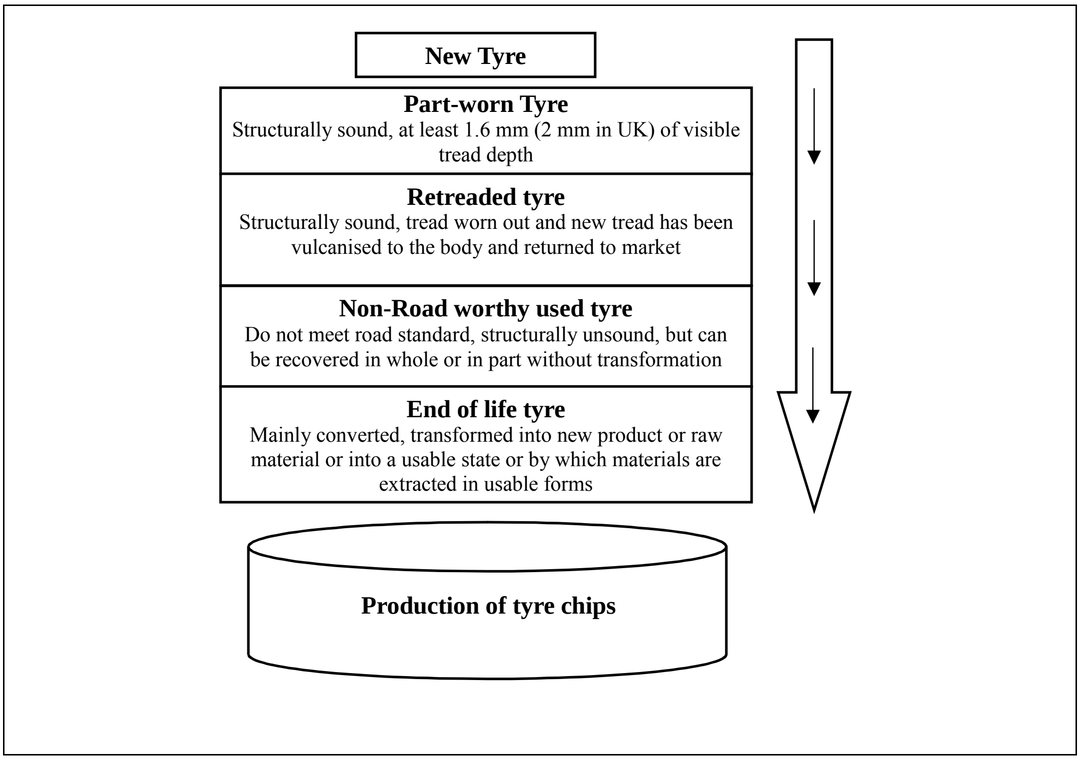


Figure 2.11: Flow chart of service life of a tyre

Slit tyres: Slit tyres are produced in tyre cutting machines. These cutting machines can slit the tyre into two halves or can separate the sidewalls from the tread of the tyre.

Table 2.4: Comparisons of car and truck tyres

Type of tyre	Weight (kg)	Recoverable rubber (kg)	% Rubber
Passenger car	9.1	5.4-5.9	35% natural, 65% synthetic
Truck	18.2	10-12.5	65% natural, 35% synthetic

Shredded or Chipped Tyres: In most cases the production of tyre shreds or tyre chips involves primary and secondary shredding. When the tyres are disposed, they can be used whole and/or as chips for different applications.

2.2.4 Characteristics of Recovered Rubber

2.2.4.1 Shredded Tyres

The shreds are basically flat, irregularly shaped tyre chunks with jagged edges that may or may not contain protruding sharp pieces of metal, which are parts of the steel plates or beads. The size of the tyre shreds may range from as large as 460 mm to as small as 25 mm, with most particles within the 100 mm to 200 mm range. The average loose density of the tyre shreds varies according to the size of the shreds, but can be expected to be between 390 kg/m^3 to 535 kg/m^3. The average compacted density ranges from 650 kg/m^3 to 840 kg/m^3 (Shulman, 2000). They are non-reactive under normal environmental conditions (Humphrey *et al*, 1993).

2.2.4.2 Tyre Chips

Tyre chips are finer and more uniformly sized than tyre shreds, ranging from 76 mm down to approximately 13 mm in size. Although the size of tyre chips, like tyre shreds, varies with the make and condition of the processing equipment, nearly all tyre chip particles can be gravel sized. The loose density of tyre chips can be expected to range from 320kg/m^3 to 490 kg/m^3. The compacted density of the tyre chips ranges from 570 kg/m^3 to 730 kg/m^3 (Bosscher *et al*, 1992). The chips have absorption values that range from 2.0 to 3.8 percent and are non-reactive under normal environmental conditions (Humphrey *et al*, 1993). The shear strength of tyre chips varies according to the size and shape of the chips with friction angles in the range of 19^o to 26^o, while cohesion values range from 4.3 kPa to 11.5 kPa. Tyre chips have permeability co-efficients ranging from 1.5 to 15 cm/sec (Humphrey *et al*, 1993).

2.2.4.3 Ground Rubber

Ground rubber particles are intermediate in size between tyre chips and crumb rubber. The particle sizing of ground rubber ranges from 9.5 mm to 0.85 mm.

2.2.4.4 Crumb Rubber

Crumb rubber used in hot mix asphalt normally has 100 percent of the particles finer than 4.75 mm. The majority of the particle sizes range within 1.2 mm to 0.42

mm. Some crumb rubber particles may be as fine as 0.075 mm. The specific gravity of the crumb rubber varies from 1.10 to 1.20 (depending on the type of production) and the product must be free from any fabric, wire and/or other contaminants (Heitzman, 1992, Schnormeier, 1992).

2.2.5 Production and Specifications of Crumb Rubber

To produce crumb rubber, it is usually necessary to further reduce the size of the tyre shreds or chips. The ambient and cryogenic processes are the two main methods normally used to produce crumb rubber.

2.2.5.1 Ambient Process

Ambient grinding can be classified in two ways: granulation and crackermill. Typically, the material enters the crackermill or granulator at “ambient” or room temperature. The temperatures rise significantly during the grinding process due to the friction generated as the material is being “torn apart”. The granulator reduces the rubber size by means of a cutting and shearing action. A screen within the machine controls product size. Screens can be changed according to end product size. Rubber particles produced in these methods normally have a cut surface shape and are rough in texture, with similar dimensions on the cut edges. Crackermills are low speed machines and the rubber is usually passed through two to three mills to achieve various particle size reductions and further liberate the steel and fibre components. The crumb rubber produced in the crackermill process is typically long and narrow in shape and has high surface area.

2.2.5.2 Cryogenic Process

In this process liquid nitrogen or a similar material/method is used to freeze tyre chips prior to size reduction. Most rubber becomes embrittled or “glass-like” at temperatures below -80^{o}C. The use of cryogenic temperatures can be applied at any stage of size reduction of scrap tyre. Typically, the size of the feed material is a nominal 50 mm chip or smaller. The material is cooled in a tunnel-style chamber or immersed in a “bath” of liquid nitrogen to reduce the temperature of the rubber or tyre chip. The cooled rubber is ground in an impact type reduction unit, usually a hammer mill. This process reduces the rubber to particles ranging from 6 mm to less than 0.85 mm. Steel from the scrap tyre is normally separated out of the product by using magnets. The fibre is removed by aspiration and screening. The resulting material appears shiny, clean, with fractured surfaces and low steel and fibre content due to the clean breaks between fibre, steel and rubber.

2.2.5.3 Specifications

Crumb rubber is classified as number one, two and so on, depending on quality and size. Table 2.5 presents a summary of crumb rubber grades. However, crumb rubbers produced in industry should maintain certain quality requirements with respect to their grades and specifications. There is no national standard available in the United Kingdom, but a European standard is now underway. Most industries in the UK use their own specifications, although ASTM standards are widely used in many parts of the world. ASTM D5603-96 and ASTMD5644-96 are the two most widely used grading standards.

Table 2.5: Crumb rubber specification

Grade	Size	Description
No.1 and 2 Tyre Granule (minus 40 grades)	6.35 mm to less than 0.635 mm	*Guaranteed metal free.* Magnetically separated materials are not acceptable. Fluff from tyre cord removed. Less than 0.635 mm refers to material that has been sized by passing through a screen with 40 holes per centimetre (referred as minus mesh 40 grades).
No.3 Tyre Granule (minus 4 grades)	less than 6.35 mm	*Magnetically separated materials (these materials cannot be certified as metal free due to residual metal/oxide content.* Metal is magnetically separated. Fluff from tyre cord removed. Less than 6.35 mm refers to material that has been sized by passing through a screen with 4 holes per centimetre.
No.4 Tyre Granule (minus 80 grades)	6.35 mm to less than 0.3175 mm	*Magnetically Separated*. Fluff from tyre cord removed. Less than 0.3175 mm refers to material that has been sized by passing through a screen with 80 holes per centimetre.

2.3 ASPHALT MIXTURES

The aim of a road pavement is to support the loads induced by traffic and to distribute these loads in such a way that the transmitted stresses do not exceed the capacity of the subgrade. Typically, UK flexible pavements consist of two main layer types (Figure 2.12): the bituminous layer including surfacing, binder course and base, and foundation layer including sub-base layer and subgrade. However, each layer of the pavement contributes to the overall performance of the road structure. Surfacing is principally to provide adequate skid resistance and has little structural significance. The binder course is to provide a smooth surface on which to construct the relatively thin surfacing and also to help to distribute the traffic load to the base, which is the main structural load bearing layer.

Brown (1997) suggested that in designing materials for bituminous layers, the designer should take account of the essential requirements of the following mechanical properties:

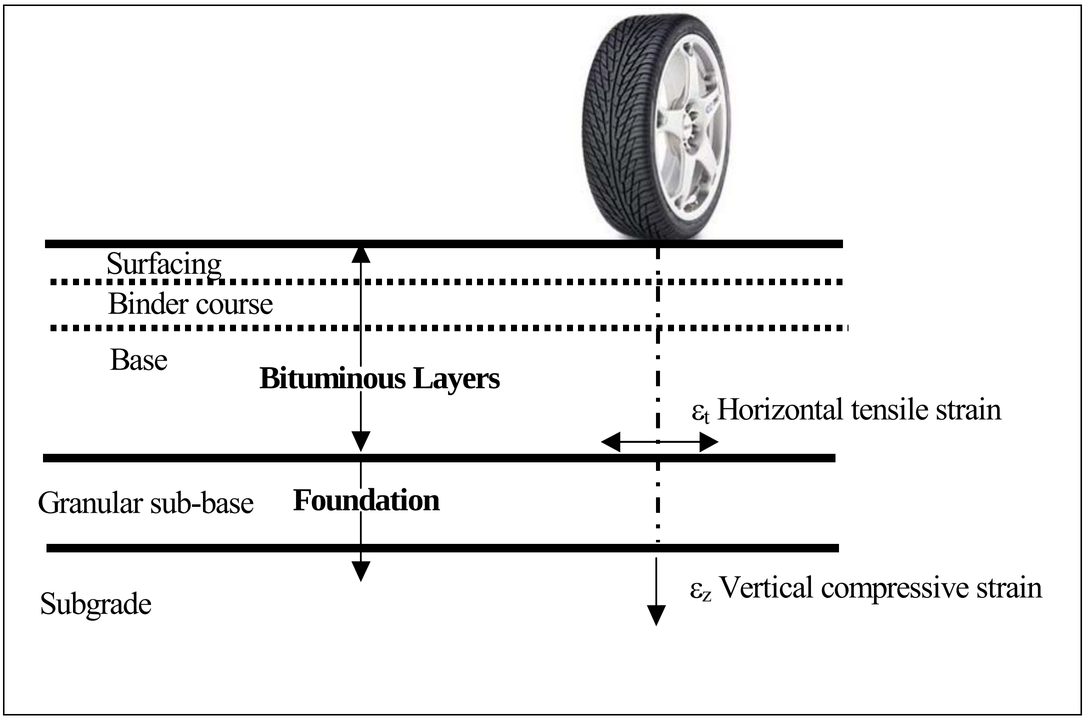


Figure 2.12: Typical UK flexible pavement structure

- High elastic stiffness to ensure good load spreading ability

- High fatigue strength to prevent the initiation and propagation of cracks due to repeated traffic load and due to environmental variation, i.e. temperature.
- High resistance to permanent deformation to prevent surface rutting.
- Adequate skid resistance in wet weather as well as a comfortable vehicle ride.

In order to design and evaluate road pavements it is necessary to have an understanding of their failure mechanism. The two main failure mechanisms for analytical pavement designs are illustrated in Figure 2.12. The maximum tensile strain at the bottom of the bituminous layer, which generally reflects the fatigue criteria and the maximum compressive strain at the top of subgrade that reflects permanent deformation criteria.

2.3.1 Mechanical Properties

2.3.1.1 Stiffness

The stiffness modulus is an important performance indicator for asphalt mixtures especially the binder and base layers. The elastic stiffness in a pavement is a measure of the material's ability to spread the traffic loading over an area. A mixture with high elastic stiffness spreads load over a wider area which reduces the level of strain experienced lower down in the pavement structure, dependent upon the temperature and frequency of loading. The stiffness of a bituminous material can be used in the calculation of required layer thickness in pavement design. The stiffness parameter is generally evaluated as the ratio between the maximum stress and the maximum strain (Equation 2.12).

$$E = \frac{\sigma}{\varepsilon} \tag{2.12}$$

Where;

E	=	elastic stiffness (MPa)
σ	=	applied stress (N/ mm^2)
ε	=	resultant strain

Although the stress-strain response of a bituminous material is viscoelastic, this elastic relationship has been found suitable for design purposes provided that a number of boundary conditions are applied. There are different methods available to determine the stiffness modulus of the bituminous mixtures. These are tension compression tests, shear tests, constant height shear tests, 2-point bending, 3-point bending, indirect tensile test, 4-point bending test etc (di Benedetto and de La Roche, 1998). Among them, the indirect tensile stiffness modulus test is currently the most convenient laboratory method as this test is relatively simple to perform, non-destructive in nature, can be conducted under static or repeated loading condition and can provide tensile strength, modulus of elasticity, Poisson's ratio, fatigue characteristics and permanent deformation characteristics of the material (Kennedy, 1977). A range of factors such as binder grade, binder content and mixture density influence the stiffness modulus. As mixture density is influenced by aggregate grading, aggregate shape and level of compaction, stiffness modulus is a valuable indicator of the quality of the material.

2.3.1.2 Fatigue

Fatigue can be defined as the phenomenon of fracture under repeated or fluctuating stress having a maximum value generally less than the tensile strength of the material. It consists of two main phases, crack initiation and crack propagation, and is caused by tensile strains generated in the pavement by not only traffic loading but also temperature variations and construction practices (Read, 1996; Rao *et al*, 1990). The fatigue characteristics of asphalt mixtures are usually represented in terms of initial stress or strain and the number of load repetitions to failure and can be expressed approximately as follows (Monismith *et al*., 1985);

$$N_f = A\left(\frac{1}{\varepsilon_t}\right)^b\left(\frac{1}{S_{\max}}\right)^c \qquad \textbf{(2.13)}$$

Where;

N_f = number of load application to failure

ε_t = tensile strain

S_{mix} = mixture stiffness

A,b,c = experimentally determined coefficients

The magnitude of the tensile strain produced due to the stress is dependent on the stiffness modulus and the nature of the pavement. Theoretical analysis and insitu measurements have indicated that tensile strain is in the order 30-200microstrain under a standard axle load (80kN) at the bottom of the main structural layer in a typical pavement construction (Read, 1996). Under these conditions the possibility of fatigue cracking exists. To represent actual traffic loading in fatigue testing, it is important to look at how the loading is applied to a pavement structure. Figure 2.13 shows how the stresses developed by a moving wheel on an element in the pavement change with time. The time of loading is dependent upon vehicle speed, depth below the pavement surface and wheel, axle and suspension configuration (Collop and Cebon, 1995). At a velocity of 60km/h and depth of 150 mm, for example, the time of loading will be approximately 0.015s and as bituminous materials are visco-elastic their properties are time dependent which will have an effect on the magnitude of the tensile strains developed in the structure and hence, the fatigue life.

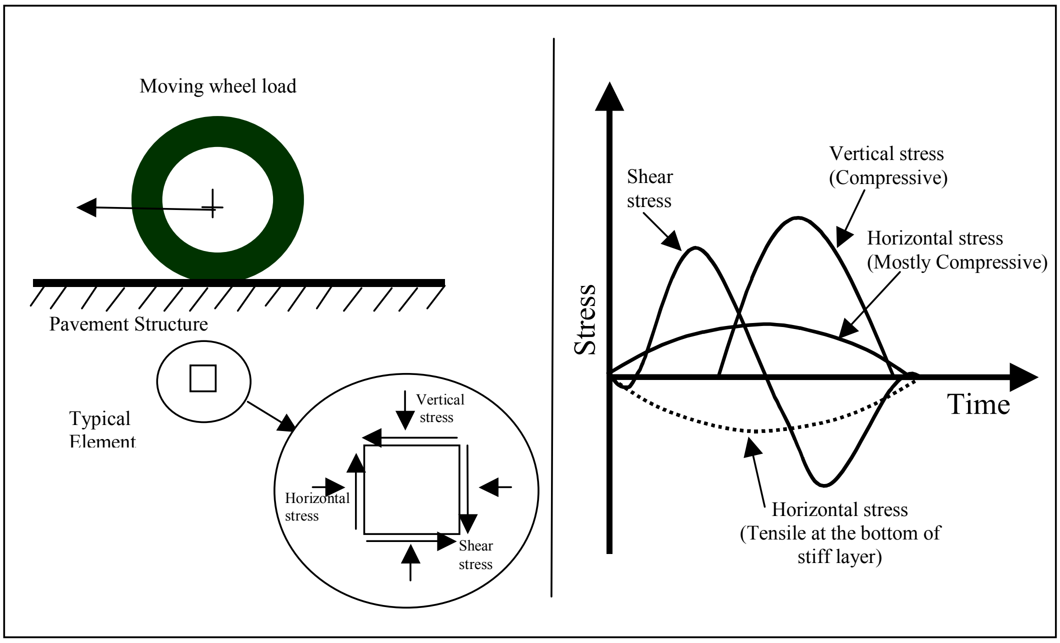


Figure 2.13: Stresses induced by a moving wheel load on a pavement element (Brown, 1978)

Van Dijk (1975) studied fatigue characteristics of asphalt mixtures using a wheel tracking test and observed that fatigue-cracking development is a three-stage process;

- The first stage is the crack initiation stage that produces hairline cracks. This is only a small portion of the fatigue life

- In the second stage, hairline cracks widen and a network of cracks form which propagate because of tensile strain. The number of load repetitions required to reach this stage is 4 times larger than that required to reach the first stage.

- Stage three is where real cracks are formed. The number of load repetitions required to reach this final stage is more than 20 times larger than that required to reach the first stage.

Apart from traffic loading and temperature variation, fatigue life of a pavement can be affected by mode of loading, loading pattern and mixture variables. As presented in Figure 2.14, the mode of loading could be either stress controlled or strain controlled (Read, 1996). In the controlled stress mode of loading, the stress is held constant and the strain gradually increases as the specimen is damaged until failure. In the controlled strain mode of loading, the strain is held constant and the stress gradually decreases as the specimen is damaged. It is important in the design process to consider fatigue characteristics over a range of traffic and environmental conditions to ensure adequate pavement service life.

Fatigue tests can be carried out by applying a load to a specimen in the form of an alternating stress or strain and determining the number of load applications required to induce failure of the specimen. Several test methods are available for the measurement of the dynamic stiffness and fatigue characteristics of bituminous mixtures. These are; bending tests using beams or cantilevers, compressive, compressive/tensile (push/pull) and indirect tensile tests. Whatever method is chosen, the test is performed under either controlled stress or controlled strain conditions.

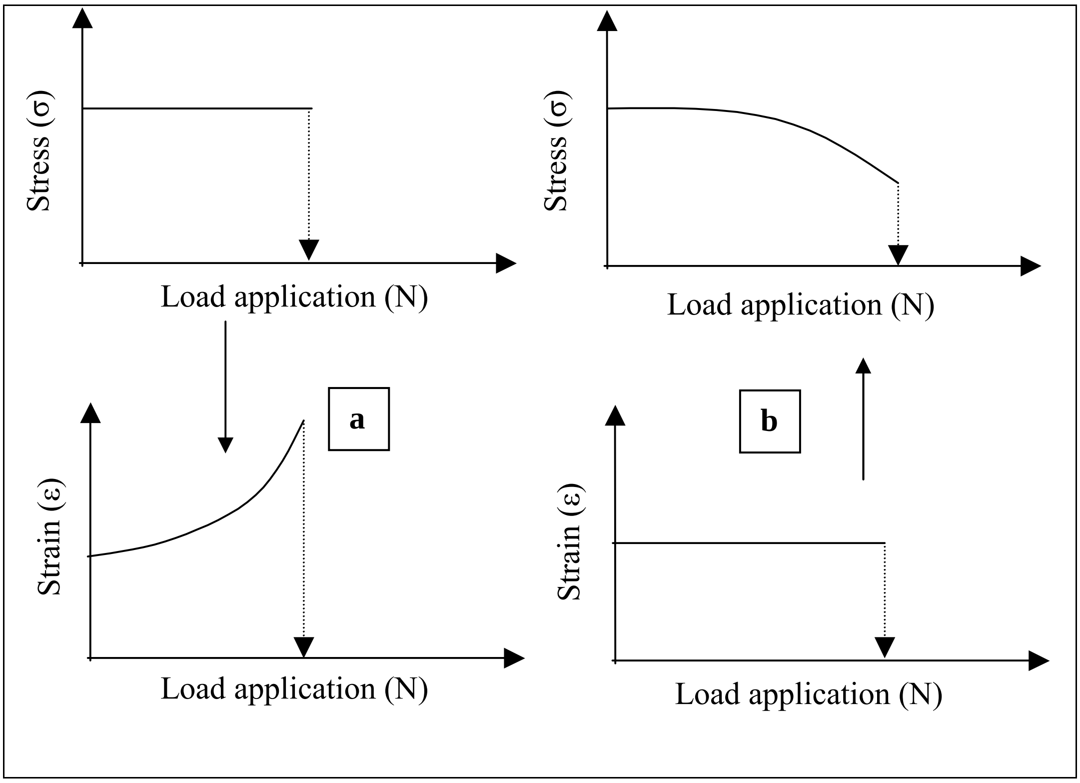


Figure 2.14: Graphical representation of (a) controlled stress and (b) controlled strain modes of loading (Epps and Monismith, 1971)

In a controlled stress test, which is more applicable to thick construction (>150 mm), the amplitude of the applied stress is held constant during the test whilst monitoring the resultant strain. The crack propagation time is quite fast and complete fracture of the specimen occurs. On the other hand, in controlled strain testing, more applicable to thin surfacings (≤ 50 mm), the strain is held constant and the stress gradually decreases as the specimen is damaged, thus requiring less stress to produce the same strain. The crack propagation is relatively slow and the failure is not where the sample literally fails but is an arbitrary end point due to the large amount of crack propagation time. However, in a control strain test the sample is usually deemed to have failed when the load required to maintain strain reaches 50% of its initial value. Therefore in general, a controlled strain test gives a prolonged fatigue life compared to control stress testing. In addition, it is possible to get a combined stress and strain controlled mode of loading in a pavement structure (Read, 1996).

In order to assess the fatigue relationship for a bituminous material it is necessary to define the failure of the test specimen in a consistent way. Depending on the loading condition, the fatigue relations can be explained in two different ways,

For strain controlled test; $$N_f = A\left[\frac{1}{\varepsilon}\right]^b \qquad \textbf{(2.14)}$$

For stress controlled test; $$N_f = C\left[\frac{1}{\sigma}\right]^d \qquad \textbf{(2.15)}$$

Where;

N_f	=	number of load application to failure,
ε, σ	=	tensile strain or stress repeatedly applied
A, b, C, d	=	material coefficients

Pell (1973) demonstrated that tensile strain is the important parameter for fatigue cracking and the resulting strain life relationship is considered to be a better basis for the comparison of different mixtures. Based on this criterion, the results from a stress-controlled test can also be presented by a strain value, as follows;

$$N_f = K_1\left[\frac{1}{\varepsilon_i}\right]^{K_2} \qquad \textbf{(2.16)}$$

Where;

N_f	=	number of load applications to failure at a particular level of initial strain
ε_i	=	initial tensile strain and
K_1, K_2	=	material coefficients.

2.3.1.3 Permanent Deformation

Rutting or permanent deformation is one of the predominant types of distress observed in pavement structures. Rutting primarily develops through shear displacement (Hofstra, 1972, Eisenmann, 1987), though there may also be a degree of densification under traffic, particularly in pavements that are not adequately compacted during construction (Mallick et al., 1995, Goncalves *et. al,* 2002). It is this longitudinal depression in the vehicle wheel path which has adverse effects on pavement structure, safety, loss of riding comfort, difficulty in changing lanes at high speed and water logging causing aquaplaning (Thrower, 1979). There are two types of rutting that most flexible pavements experience. Rutting due to deformation within the bituminous material (non-structural rutting) can generally be distinguished from structural rutting by the profile generated at the surface, which is characterised by the formation of "shoulders" at the edge of the ruts, as shown in Figure 2.15, due to lateral displacement of the material.

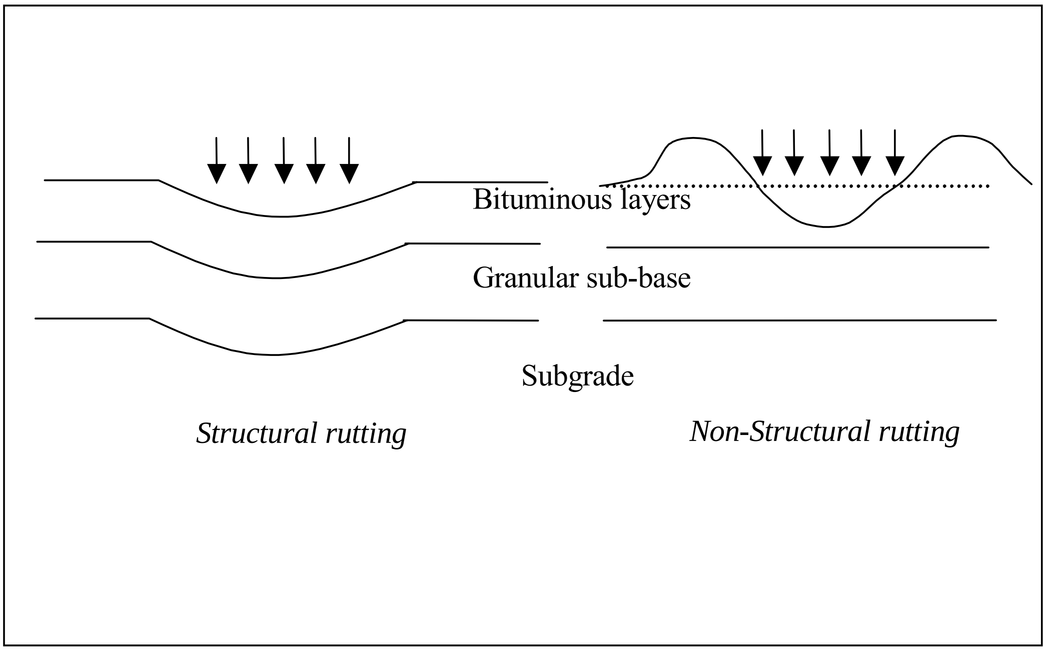


Figure 2.15: Structural and non-structural rutting

Structural rutting is considered to be the failure of the pavement to reduce the load on the formation to an acceptable level, while non-structural rutting is confined within the bituminous layers. The development of rutting is often a combination of the effect of both mechanisms (Brown, 1984).

As mentioned before, rutting in a flexible pavement is caused by deficiencies in layers through densification during construction and plastic movement of the asphalt mixture subjected to traffic loading at high temperatures. In general terms, bitumen behaves as an elastic solid at low temperature and high frequencies of loading, as a viscous fluid at high temperature and low loading frequencies and exhibits visco-elastic behaviour in the intermediate range. As a consequence, the response of a bituminous mixture to an applied load is in part viscous, the degree of viscous behaviour being dependent upon both temperature and the duration of loading. Figure 2.16a shows the response of a bituminous mixture to an idealised load pulse. It can be seen from Figure 2.16b that after the load has been removed there is a small amount of irrecoverable plastic and viscous deformation. Although, this deformation is small, the effect is cumulative and after a large number of load passes a rut will develop.

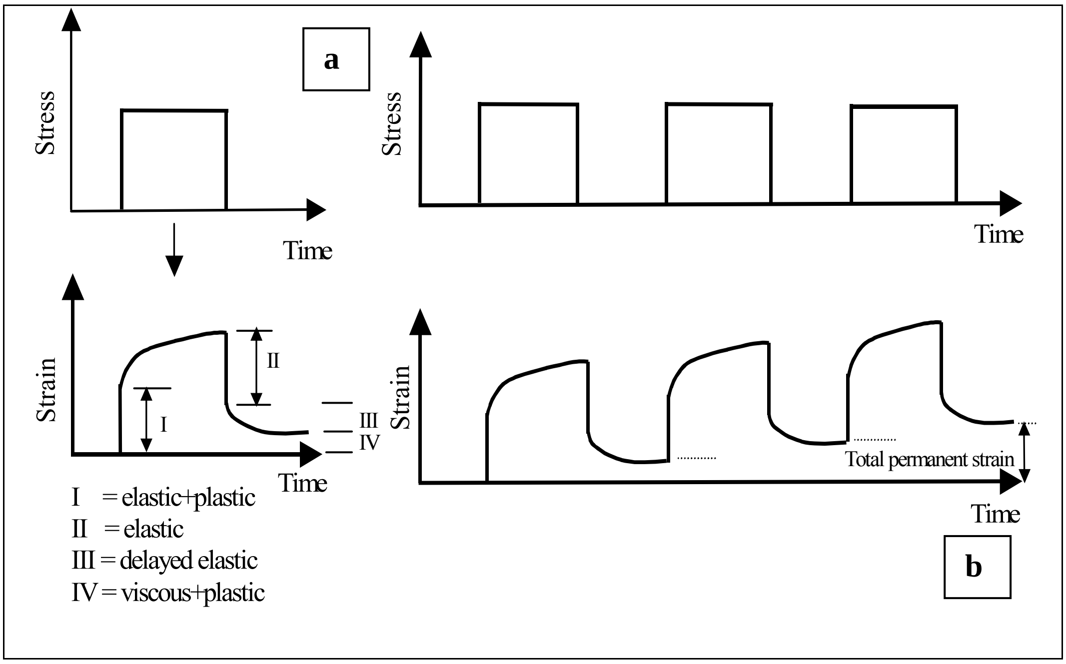


Figure 2.16: (a) Idealised strain (b) accumulation of permanent strain under repeated loading of a bituminous mixture

Resistance to permanent deformation is influenced by bitumen and aggregate properties, their proportion in the mixture and field conditions. Bitumen inherent effects are important but their influence is small compared to the effects of aggregates and air voids, especially at higher temperatures (e.g. 40^0C) or when the mixture is submitted to a stress state that amplifies aggregate influence. An important bitumen property in resistance to permanent deformation is the grade or hardness of bitumen, however the extent of the effect depends on the type of mixture. Hofstra and Klomp (1972) compared rut depths with various grades of bitumen and found that the hardest bitumen gave the smallest rut depth and subgrade deformation. In hot rolled asphalt, resistance to permanent deformation depends on the bitumen, filler, and fine mortar stiffness but for Dense Bitumen Macadam (DBM) or Stone Mastic Asphalt (SMA) it depends more on particle interlocking.

Aggregate gradation, shape and texture are also important factors that affect rutting (Sousa, 1991). A mixture with a skeleton of aggregates which are in intimate contact with each other after compaction will provide more load bearing and resistance to permanent deformation. Gap graded mixtures like HRA have less resistance to permanent deformation in comparison with continuously graded mixtures like DBM or asphaltic concrete. However, gap graded mixtures with high coarse aggregate content like SMA (aggregate content greater than 70%) have much greater contact of aggregate particles that provide more resistance to permanent deformation.

Volumetric composition of mixtures, such as voids in mixed aggregates (VMA), has an influence on resistance to permanent deformation. VMA is the combination of volume of air and volume of bitumen. Cooper *et al*. (1985) reported that optimum VMA increases aggregate contact which is desirable for resistance to permanent deformation but very low VMA or very dense graded bituminous mixtures are susceptible to permanent deformation because the bitumen/filler mortar is forced between coarse aggregate particles and reduces the number of contact points. Hence a minimum VMA should be ensured in the mix by adjusting the maximum density grading.

VMA is also influenced by bitumen content and degree of compaction. There is optimum bitumen content which results in minimum VMA and maximum particle contact points and greater resistance to permanent deformation. On the other hand, a mixture with excessive bitumen content creates thicker bitumen films around the aggregates, reduces void contents and results in a decrease in resistance to permanent deformation. Low air void contents cause reductions in resistance to permanent deformation and this is the reason why mixture design methods should have a limiting minimum air void content.

2.3.2 Nottingham Asphalt Tester

The Nottingham Asphalt Tester (NAT) can be configured for testing specimens in the indirect tensile mode, for stiffness determination following the theory explained in the next sub-section, or for fatigue testing, or in the repeated load axial testing mode for assessing the permanent deformation of a mixture (Armitage *et al*, 1996, Cooper and Brown, 1989).

The NAT has four main units, a main test frame placed in a temperature control cabinet, an interface unit for the acquisition of data and controlling of the test, a pneumatic unit connected with the interface unit and an actuator mounted above the test frame for controlling the applied load. A picture of the NAT testing configuration is shown in Figure 2.17. The computer controls a voltage/pressure the (V/P) converter to operate a solenoid valve via the interface unit. Using software, a pre-determined pressure is introduced into a reservoir in the pneumatic unit and the solenoid valve is switched to apply a load to a specimen. The load is then measured using a load cell and the V/P converter is adjusted to achieve the required level.

Using the NAT, a pulsating load is applied vertically across the diameter of the cylindrical specimen and the horizontal deformation is measured using two linear variable differential transformers (LVDT), as shown in Figure 2.17, which are mounted diametrically opposite one another in a rigid frame clamped to the test specimen. Instantaneous recoverable deformation is difficult to measure, as there is no well-defined point of inflection on the unloading portion of the deformation curve. So, for determination of resilient modulus, the total horizontal deformation is used.

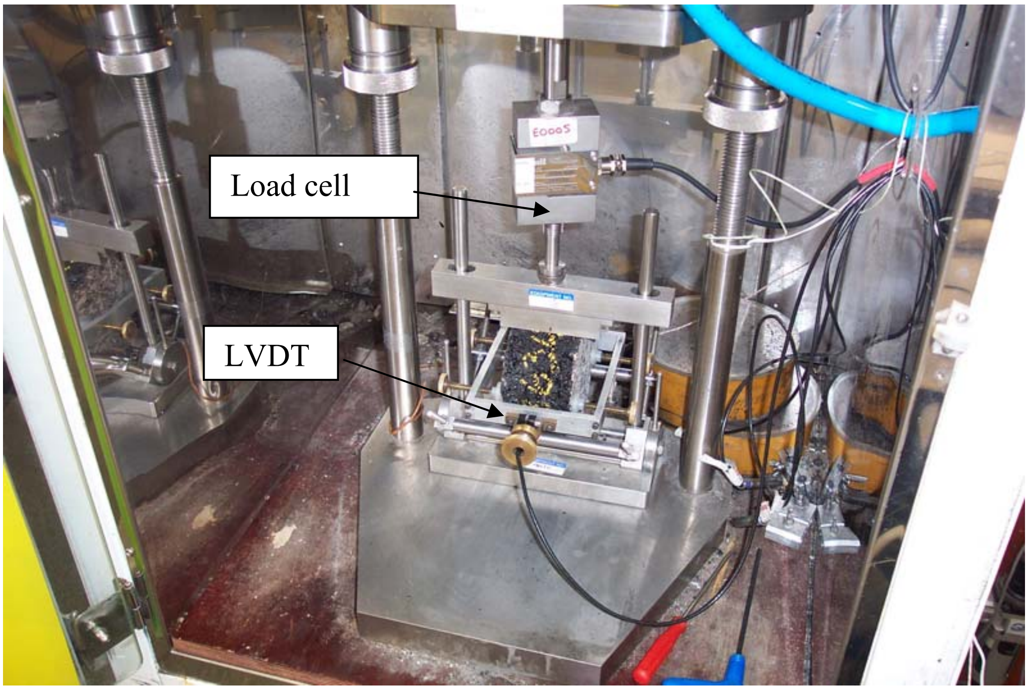


Figure 2.17: ITSM testing arrangement in the NAT

2.3.2.1 Indirect Tensile Stiffness Modulus Test (ITSM)

In the ITSM test, a number of assumptions are required to get the linear elastic method to be applicable (Hudson *et al*, 1968). These are;

- The specimen is subjected to plane stress conditions ($\sigma_z = 0$)
- The material behaves in a linear elastic manner
- The material is homogenous
- The material behaves in an isotropic manner
- Poisson's ratio (ν) for the material is known
- The force (P) is applied as a line loading

Based on the above assumptions, the test is conducted for the measurement of small strains on bituminous mixtures by applying an impulse loading on two diametrically opposed generating lines of a cylindrical specimen. The central part of the sample is then subjected to a tensile stress as the vertical loading produces both a vertical compressive stress and a horizontal tensile stress on the diameter of the specimen. The magnitudes of the stresses vary along the diameter as shown in Figure 2.18, but are at a maximum in the centre of the specimen.

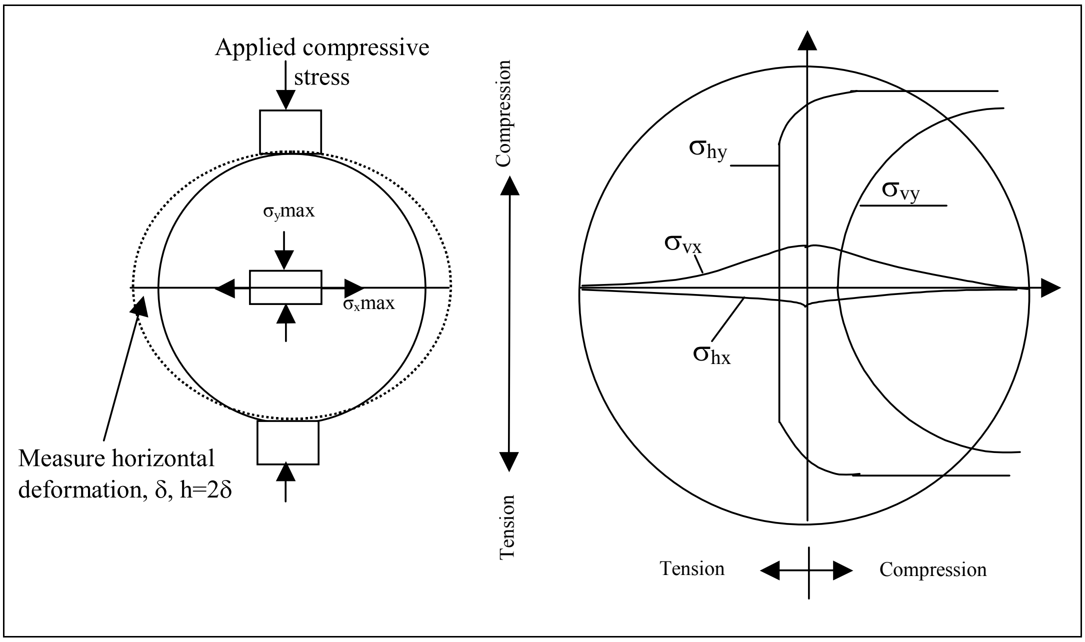


Figure 2.18: Tensile and compressive stresses in a cylindrical specimen

The orientation of the sample is chosen in such a way that the stress at the centre of the sample is representative to the actual stress distribution in a bituminous road layer. The critical location for load induced cracking is generally considered to be at the bottom of the bituminous layer and immediately underneath the load, where the stress state is longitudinal and transverse tension combined with vertical compression (Roque and Buttler, 1992).

The maximum and average stresses on the horizontal diameter are as follows;

$$\sigma_{hx(\max)} = \frac{2P}{\pi dt} \qquad \textbf{\textit{(2.17)}}$$

$$\sigma_{vx(\max)} = \frac{-6P}{\pi dt} \qquad \textbf{\textit{(2.18)}}$$

Average horizontal stress, $$\overline{\sigma}_{hx} = \frac{0.273P}{dt} \qquad \textbf{\textit{(2.19)}}$$

Average vertical stress, $$\bar{\sigma}_{vx} = \frac{-P}{dt} \qquad \textbf{\textit{(2.20)}}$$

Where;

P	=	applied load
d	=	specimen diameter
t	=	specimen thickness
σ_{vx}	=	vertical compressive stress across x-axis
σ_{hx}	=	horizontal tensile stress across x-axis
σ_{hx}	=	horizontal stress across y-axis
σ_{vy}	=	vertical compressive across y-axis

Considering the average principal stresses in a small element subjected to biaxial stress conditions, the horizontal strain (ε_{hx}) would be:

$$\bar{\varepsilon}_{hx} = \frac{\sigma_{hx}}{S_m} - \upsilon \frac{\sigma_{vx}}{S_m} \qquad \textbf{\textit{(2.21)}}$$

$$\bar{\varepsilon}_{hx} = \frac{0.273P}{S_m dt} + \frac{\upsilon P}{S_m dt} \qquad \textbf{\textit{(2.22)}}$$

Horizontal deformation, $\Delta h = \varepsilon_{hx}$ d

Therefore;

$$\Delta h = \frac{0.273P}{S_m t} + \frac{\upsilon P}{S_m t} \qquad \textbf{\textit{(2.23)}}$$

The stiffness modulus of the material can then be calculated from:

$$S_m = \frac{P}{\Delta h t}(0.273 + \upsilon) \qquad \textbf{\textit{(2.24)}}$$

Where;

S_m = Stiffness modulus of the material

ν = Poisson's ratio

A brief summary of the ITSM testing protocol illustrated from BS DD213: 1993 is presented in Table 2.6.

Table 2.6: Summary of BS DD213: 1993 for ITSM test

Feature	**NAT method**
Rise time (milliseconds)	125±10
Deflection requirements	<7µm
Load (N)	As required
Pulse duration	3s
Number of conditioning pulses	Minimum 5
Number of test pulses	5
Test temperature	20±0.5
Poisson's ratio	0.35[a]
Rotation of sample	90^0+10^0
Time to reach equilibrium	>4hrs
Specimen height (mm)	30-80 mm
Test result	The mean of two measurements on the one specimen 90^0 apart and which do not differ by more than 10% from the mean

2.3.2.2 Indirect Tensile Fatigue Test (ITFT)

The Indirect Tensile Fatigue Test method is particularly popular for routine fatigue testing due to its simplicity relative to other methods, it uses cylindrical specimens, which can be easily manufactured in the laboratory or cored straight from the pavement and it appears be able to discriminate between mixtures containing different binders based upon both stiffness and cycles to failure. In addition, the biaxial state of stress applied in ITFT test better represents the field conditions than simple flexural tests. However, there are several disadvantages that also need to be taken into considerations. These are;

[a] as a result of the incorporation of the more flexible crumb rubber material into bituminous mixtures, it is likely that Poisson's ratio may be higher than the assumed value of 0.35

- ITFT tends to significantly underestimate actual fatigue life of the material if the principal stress is used as the damage determinant (Read, 1996).
- It is also not possible to vary the ratio of vertical and horizontal components of the biaxial stress condition, and, therefore, the ITFT fails to replicate the stress state at critical locations within the pavement.
- It is not possible to induce stress reversal, and the results are unwanted accumulation of permanent deformation.

In ITFT, a repeated diametrical line loading is applied along the vertical diameter, which produces an indirect tensile stress on the horizontal diameter. The magnitudes of the stresses vary along the diameter but are at the maximum at the centre of the specimen. Linear elastic theory can be applied to calculate maximum strain developed at the centre of the specimen by assuming that;

- The specimen is subjected to plane stress conditions ($\sigma_z = 0$)
- The material behaves in a linear elastic manner
- The material is homogenous
- The material behaves in an isotropic manner
- Poisson's ratio (ν) for the material is known
- The force (P) is applied as a line loading

Using the above assumptions and when the width of the loading strip is less than or equal to 10% of the diameter of the specimen and the distance of the element of material from the centre is very small (Timoshenko, 1934, Sokolnikoff, 1956), then the maximum horizontal tensile stress (σ_{max}) and strain (ε_{max}) at the centre of the specimen are;

$$\sigma_{x\max} = \frac{2P}{\pi dt} \qquad \textbf{(2.25)}$$

$$\varepsilon_{x\max} = \frac{\sigma_{x\max}}{S_{mix}}(1+3\upsilon) \qquad \textbf{(2.26)}$$

Where;

$\sigma_{x\ max}$	=	maximum horizontal tensile stress at the centre of the specimen (kPa)
$\varepsilon_{x\ max}$	=	maximum initial horizontal tensile strain at the centre ($\mu\varepsilon$)
S_{mix}	=	the indirect tensile stiffness modulus (MPa)
d	=	diameter of the test specimen (mm)
t	=	thickness of the test specimen (mm)
P	=	applied vertical load (N)
ν	=	Poisson's ratio (assumed to be 0.35)

The ITFT test in the NAT is reported in BS DD ABF (2002) and the testing arrangements are shown in Figure 2.19. Important features of the specifications are listed in Table 2.7.

Table 2.7: Summary of BS DD ABF 2002 for ITFT test

ITFT Feature	**NAT method**
Rise time (milliseconds)	125±10
Load frequency	40 pulse/minute = 0.67Hz
Specimen dimension	D=100±3 mm, H=40±5 mm
Stress level	The target stress level for first specimen is 500 or 600kPa. The following stress level should be chosen in such a way that minimum spread of lives must be three orders of magnitude so that the maximum value of N at failure is at least one thousand times greater the minimum value
Test temperature	20 ± 0.5^0C
Poisson's ratio	0.35 at 20^0C
Test result	The result is plotted in a log-log scale using tensile strain vs. number pulses required to fail the sample. The correlation coefficient (R^2)>0.90 in a linear regression of 10 samples
Application	This method is applicable to wearing courses, base courses and roadbase containing penetration grade or modified bitumen
Failure indication	9 mm vertical deformation

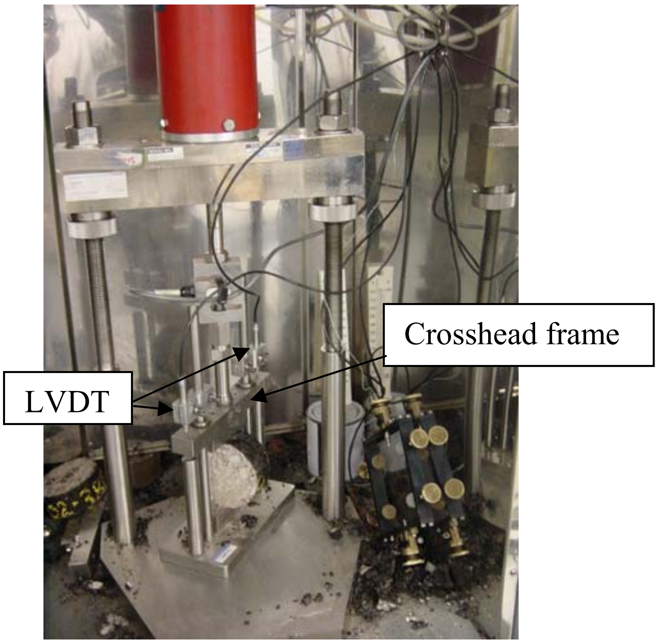


Figure 2.19: ITFT testing arrangement in NAT

2.3.2.3 Confined Repeated Load Axial Test

The Confined Repeated Load Axial Test (CRLAT) is a modified version of the Repeated Load Axial Test (RLAT) developed at the University of Nottingham (Nunn *et al*, 1999). The RLAT is mainly used to investigate the permanent deformation behaviour of bituminous mixtures. The main advantage of using the CRLAT over the RLAT is that the CRLAT includes the influence of aggregate gradation, size and shape in permanent deformation as well as the influence of different binders (Oliver *et al*, 1996). In addition, different studies (LCPC, 1996, Ulmgren, 1996, Ulmgren *et. al*, 1998, Nunn *et al*, 1999) have shown that the CRLAT discriminates better between different aggregate gradations over a broad range of asphalt mixtures (porous asphalt, gussasphalt and asphalt concrete).

In the CRLAT, a load pulse is applied by a rolling diaphragm pneumatic actuator using compressed air which is governed by a solenoid valve. The operation of this is controlled by a microcomputer via a digital to analogue converter. The specimen is sealed within a rubber membrane and is secured at both ends by 0-rings, which rest in purpose cut grooves around the perimeter of two specially designed platens. The upper plate has an outlet pipe fitted in the base which connects via a pressure regulator and gauge to a vacuum pump (Figure 2.20). The test is performed according to BS DD 226, as for the conventional RLAT method, but the air is extracted from the specimen using a vacuum pump to reduce the pressure inside the specimen to 70kPa below atmospheric pressure, which produces an effective confining stress of 70kPa.

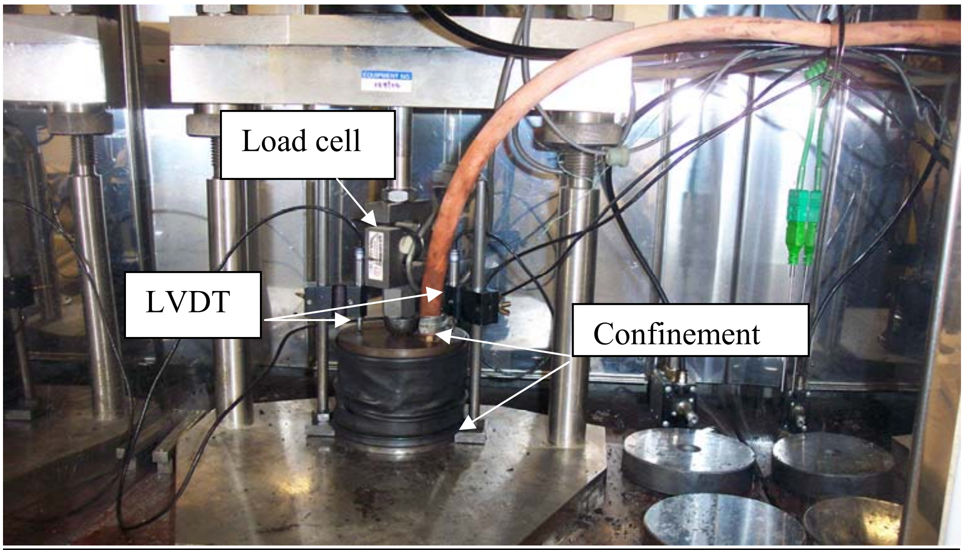


Figure 2.20: CRLAT testing arrangement in NAT

The load is applied vertically to the specimen and the resultant deformation is monitored by two vertical LVDT fixed on top of the upper platen. Outputs from these devices are converted using an analogue to digital converter and acquired in a computer. The use of microcomputer control and data acquisition allows flexibility, continuous monitoring and recording of data automatically throughout the test (Cooper *et al*., 1989, BS DD 226, 1996).

2.4 DURABILITY OF ASPHALT MIXTURES

A durable product is one, which can withstand a long period of time without significant deterioration. In terms of application to bituminous paving materials, engineers primarily restrict the term durability to those effects, which are related to the environment; namely moisture and ageing, assuming that the bituminous pavement layer is constructed perfectly according to the specifications. Terrel and Al-Swailmi (1994) found in their research that environmental factors such as temperature, air, and water could have profound affects on the durability of asphalt concrete mixtures. Scholtz and Brown (2000) proposed the definition of durability as the ability of the material comprising the mixture to resist the effect of water, ageing, and temperature variations, in the context of a given amount of traffic loading, without significant deterioration for an extended period. However, as the cost of the maintenance and rehabilitation of pavement structures is expensive, the issue of durability must be taken into account during material selection, design and the implementation stage.

In the following sections, the two most important damage factors on durability, moisture damage and age hardening are briefly explained.

2.4.1 Moisture Damage

The two main mechanisms by which water can damage the structural integrity of the bitumen-aggregate interface are, firstly loss of cohesion (strength) and stiffness of the bitumen; and secondly, loss of the adhesive bond between the bitumen and the aggregate in the mixture (stripping) (Kennedy, 1985; Terrel and Al-Swailmi, 1994). These two water damage mechanisms result in decreasing the strength of the pavement layer (Scholtz and Brown, 1996). The detachment of bitumen from the aggregate (or stripping) is associated with mixtures, which are permeable to water. The lower the air voids content in a compacted mixture the less risk of stripping (Whiteoak, 1991). The loss of adhesion often accelerates the pavement deterioration and may result in a total loss of the capital invested in the pavement structure (Mostafa *et al*, 2003).

2.4.1.1 Moisture Damage Mechanism

Many moisture damage theories have been proposed, but only a few of the most commonly accepted theories are summarised below.

- ***Detachment Mechanism*** is the microscopic separation of a bitumen film from the aggregate surface by a thin layer of water with no obvious break in the bitumen film. The bitumen will peel cleanly from aggregate. The thin film of water probably results from aggregate that was not completely dried, interstitial pore water that vaporised and condensed on the surface, or possibly water that permeated through the bitumen film to the interface.

- ***Displacement Mechanism*** occurs when the binder is removed from aggregate surface by water. Compared to detachment, the free water gets into the aggregate surface through a break in the bitumen coating. The break may be from incomplete coating during mixing or from bitumen film rupture (Asphalt Institute, 1987). In this case, the aggregate, the bitumen, and the free water are all in contact. Thus the free water will tend to displace the bitumen from the aggregate surface as governed by the surface energy theory of adhesion (Kiggundu and Roberts, 1988).

- ***Spontaneous Emulsification Mechanism*** occurs when an inverted emulsion, water droplets in bitumen rather than bitumen droplets in water as found in common emulsified bitumen, is formed. This mechanism seems to be enhanced under traffic on mixtures laden with free water.

- ***Film Rupture Mechanism*** is not considered as a stripping mechanism on its own, but it is believed to initiate stripping. Film rupture is marked by a crack in the bitumen film that occurs under stresses of traffic at sharp aggregate edges and corners where the bitumen film is the thinnest. Once a break in the film is present, water is able to find its way to the interface and initiate stripping (Asphalt Institute, 1987).

- ***Pore Pressure Mechanism*** initiates stripping when water is allowed to circulate freely through the interconnected voids of high void content mixtures. Induced traffic loads cause pore pressure to build up to the point of stripping the bitumen from the aggregate.

- ***Hydraulic Scouring Mechanism*** occurs more in the surface course than the lower courses of the pavement structure. When the pavement is saturated, wheel action causes water to be pressed into the pavement in front of the tyres and to be sucked out behind the tyres. This water tends to separate bitumen from aggregate. This scouring action can be worsened by the presence of abrasive material, such as dust, on the surface of the roadway.

2.4.1.2 Moisture Sensitivity Tests

Numerous test methods have been developed in an attempt to investigate the damage mechanisms with various degree of success (Scholz and Brown, 1994, Lottman, 1982, Tunnicliff and Root, 1984, Terrel and Al- Swailmi, 1994). It is generally agreed that moisture can reduce the integrity of the bituminous mixture in two ways (Mostafa et al, 2003, Kennedy, 1985):

- Moisture causes failure of the adhesion between the mineral aggregate particles and bitumen films commonly referred to as stripping.

- Moisture also reduces the cohesive strength and stiffness of the mixture.

The Link Bitutest testing protocol developed by Scholz (1995) has been used for water sensitivity testing. It combines the merits from different popular test methods. The test method involves determining the ratio of conditioned to unconditioned indirect tensile stiffness modulus values as measured with the NAT. A brief testing procedure is described in the following section. The testing procedure is:

- The conditioning consists of saturation under a partial vacuum of 510 mm Hg at 20^{0}C for 30 minutes.

- Percentage saturation is calculated using the following formula;

$$S = \frac{M_w - M_d}{\frac{M_d}{G_{mb}} - \frac{M_d}{G_{mm}}} X100 \qquad \textbf{\textit{(2.27)}}$$

Where;

S = percent saturation

M_d = mass of dry specimen, g
M_w = mass of wet specimen, g
G_{mb} = bulk specific gravity and
G_{mm} = maximum specific gravity

- The samples are then transferred to a preheated water bath at 60^0C under atmospheric pressure for 6 hours and moved to another water bath at atmospheric pressure at 5^0C for 16 hours. The samples are finally conditioned under water at 20^0C (atmospheric pressure) for 2 hours prior to stiffness testing.
- Determine stiffness ratio = $\frac{ITSM_{Ci}}{ITSM_U}; i = 1,2,3....$
- Above steps are repeated for subsequent cycles.

2.4.2 Age Hardening

Age hardening of the bitumen also has a significant effect on the durability of the pavement structure. Ageing is primarily associated with the loss of volatile components and oxidation of the bitumen during asphalt mixture construction (short-term ageing) and progressive oxidation during service life in the field (long-term ageing). Bitumen slowly oxidises when in contact with air (oxygen) increasing the viscosity and making the bitumen harder and less flexible. The degree of viscosity is highly dependent on the temperature, time and the bitumen film thickness. Excessive age hardening can result in brittle bitumen with significantly reduced flow capabilities, reducing the ability of the bituminous mixture to support traffic and thermally induced stresses and strains, which contribute to various forms of cracking in the asphalt mixture.

2.4.2.1 Mechanism of Age Hardening

Traxler (1963) found that time, temperature and film thickness are the main factors affecting age hardening. Peterson (1984) studied the durability of asphalt mixtures and stated that; " Durability is determined by the physical properties of the bitumen, which in turn are determined directly by chemical composition. An understanding of the chemical factors affecting physical properties is thus fundamental to an understanding of the factors that control (bitumen) durability." Peterson (1984) also identifies three composition related factors that govern the changes that could cause hardening of bitumen in pavements as follows:

- Changes in chemical composition of bitumen molecules from reaction with atmospheric oxygen.

Peterson (1984) explains that when thin bitumen films are exposed to atmospheric oxygen, a rapid and irreversible oxidation occurs mainly in the polar aromatic and asphaltene fractions, as they have the highest content of hydrocarbon, resulting in a great increase in viscosity and altering the complex flow properties. This phenomenon often leads to embrittlement of the bitumen and ultimately pavement failure. Vallerga and Halstead (1971) found that for mixtures with less than 2% voids contents (very low permeability), oxidative ageing is not likely to significantly affect the rheological properties of the pavement.

- Physical hardening of bitumen occurs when bitumen is at ambient temperatures and results from reorientation of the molecules and the slow crystallisation of waxes.

Physical hardening of the bitumen occurs due to the molecular restructuring. This is a slow and largely reversible process, which appears to occur concurrently and synergistically with oxidative ageing. Physical hardening reduces flow properties of the bitumen without changing chemical composition, but the process is a significant factor causing embrittlement of bitumen, and thus, reduced durability of the mixture. Peterson (1984) stresses, however, that this phenomenon is difficult to quantify as the recovery processes (i.e., use of solvent, heat and mechanical working to obtain neat bitumen from bituminous mixtures) destroy most if not all the structuring.

- Loss of the oily components of bitumen (exudative hardening) by volatility or absorption by porous aggregates.

The loss of volatile components (i.e., the non polar saturate or oily fraction of bitumen) occurs during mixing, storage, transport and lay down of the bituminous mixture and due to the absorption of the polar components by porous aggregate. Peterson (1984) states that, "with the current specifications and construction practices, volatility is probably not a significant contributor to pavement hardening."

2.4.2.2 Short-Term Ageing Test Protocol

The short-term ageing of mixtures generally involves heating the loose mixture in an oven prior to compaction in an attempt to simulate hardening during plant mixing. Various methods have been developed to evaluate the effect of accelerated ageing on bituminous mixtures. Although the various procedures are effective in producing aged mixtures in an accelerated manner, none have been conclusively validated with the short-term field performance of bituminous pavements (Scholz, 1995). However, for this project, the Link Bitutest testing protocol was used (Scholz, 1995, Brown *et al*, 1995). The procedure requires loose mixtures, prior to compaction, to be aged in a forced draft oven at a temperature either 130^0C or related to the desired compaction temperature, whichever is higher, and that the period of conditioning is limited to two hours (Brown and Scholz, 2000).

2.4.2.3 Long-Term Ageing Test Protocol

As there is no standard test method available for ageing a bituminous mixture in the UK, The SHRP methodology, as set out in project A-003A (AASHTO, 1994) for oven ageing was, therefore, adopted in this project. ITSM tests were carried out on samples before and after long-term oven ageing to determine stiffness modulus changes due to ageing.

Several durations of Long-Term Oven Ageing (LTOA) were introduced depending on the expected service life of bituminous mixture. A 2-day oven-ageing regime appears representative of up to 5 years in service and an ageing period of 4 or 5 days is used to simulate the ageing process for 10-year-old projects. The procedure for this method is as follows;

- LTOA test is performed on compacted mixtures that have already undergone the short-term oven ageing procedure.
- Compacted specimens are placed in a forced draft oven at 85^0C for 120 hours (5days)
- At the end of the ageing period, the oven is switched off and left to cool to room temperature before removing the specimens. The specimens are not tested until at least 24 hours after removal from the oven.
- ITSM test is performed following the same testing protocol as the unaged conditioned specimens.

CHAPTER 3

Crumb Rubber Modified Asphalt Mixtures

3.1 INTRODUCTION

Scrap tyre rubber can be incorporated into asphalt paving mixtures using two different methods, which are referred to as the wet process and the dry process (Heitzman, 1991). In the wet process, crumb rubber acts as an asphalt binder modifier, while in the dry process, granulated ground rubber and/or crumb rubber is used as a portion of the coarse and/or fine aggregate. The dry process is normally used only with hot bituminous mixtures, whereas the wet process has been applied in crack sealants, surface treatments and hot bituminous mixtures. Historically rubber in asphalt mixtures has been used to improve the elasticity of the binders and mixtures. However, the application of rubber into asphalt mixtures requires careful consideration as rubber reacts with bitumen at high temperatures, consequently changing the performance of the mixtures (Singleton, 2000, Heitzman, 1991). Therefore, it is important to understand the interaction process of rubber in solvents.

In the first section of this chapter a brief overview of the interaction of rubber with solvents and bitumen will be presented together with the effect of temperature, viscosity, particle size, liquid concentration and presence of filler on the interaction process. Both the wet process and dry process, including their use, design considerations and performance, will be discussed in the next section. Finally a brief summary of the chapter will be presented.

3.2 INTERACTION OF SOLVENTS AND BITUMEN WITH RUBBER

As mentioned previously, crumb rubber is a conglomeration of vulcanised natural and synthetic rubber with permanently cross-linked three-dimensional molecular networks (Flory and Rehner, 1943). The cross-linkages may consist of primary valence bonds connecting the chains directly, or of an intermediate group or atom such as sulphur which is bonded to each of the chains (Blow, 1971). Swelling of vulcanised rubber in solvents is by imbibition processes to a degree depending on the solvent and the structure of the polymer. Imbibition is the displacement of a non-wetting fluid (such as air) by a wetting fluid (such as water) where at low injection rates, the invading fluid enters the narrowest pores before any other is considered. The shape of the cross-link network remains the same due to swelling and the swollen gel exhibits elastic rather than plastic properties (Flory and Rehner, 1943). The process of imbibition of the solvent into the polymer is known as diffusion.

3.2.1 Diffusion Theory

Diffusion theory was developed by Crank in 1956. The theory predicts that, at the start of the process, the rubber at the surface of a component has high liquid concentration while the liquid concentration in the bulk of the component is zero. Subsequently, the liquid molecules diffuse into the rubber just below the surface and eventually into the bulk of the rubber. As the diffusion process proceeds, the dimensions of the rubber component increase until the concentration of the liquid is uniform throughout the component and equilibrium swelling is achieved (Crank, 1956, Blow, 1971). The theory also predicts that at the early stage of swelling, the mass of liquid absorbed by rubber per unit area is proportional to the square root of the time. This rate is linear up to at least 50 percent of equilibrium swelling.

However, the amount of a given solvent that will diffuse into the rubber until it reaches equilibrium depends upon the number of cross-links per unit volume of rubber. The greater the number of cross-links per unit volume, the shorter the average length of rubber chains between cross-links and the lower the degree of swelling (McCrum *et al*., 1999). In addition, the lower the molecular weight of the solvent, the more readily it will diffuse into the rubber (Treloar, 1975). In addition, the degree of swelling depends upon the compatibility of the rubber and solvent on a molecular scale, the complex chemical nature and viscosity of the solvent and the temperature (Treloar, 1975).

3.2.2 Swelling of Crumb Rubber in Bitumen

Green and Tolonen (1977) studied the diffusion of bitumen into crumb rubber in the wet process to investigate the rate and maximum swelling for suitable storage times. In the wet process crumb rubber acts as a bitumen modifier where the reaction time is longer and the size of the crumb rubber is very small. However, the main purpose of rubber-bitumen reaction is to increase the viscosity and elasticity of the binder. The dry process, on the other hand, uses larger sizes of rubber particles, as a partial replacement of aggregate and the reaction time is usually shorter as mixtures are only kept at high temperatures during the production, transportation and compaction stages. Therefore, the effect of swelling of rubber particles in the dry process is different from the wet process as the binder modification and rubber swelling could change the mechanical properties of the mixtures. Research conducted by different researchers (Flory and Rehner, 1943, Southern, 1967, Green and Tolonen, 1977) found that the rate of swelling depends mainly on temperature, rubber particle size, viscosity, and the concentration of solvent. Brief explanations of how these factors affect the interaction process are presented in the following sections.

3.2.2.1 Effect of Temperature

Flory and Rehner (1943) explained the effect of temperature using the concept of entropy and the Holtzman equation of free energy. They found that temperature has two effects on swelling as long as there is no change in the constitution of the rubber. Firstly, as the temperature increases, the rate of swelling increases and secondly, the extent of swelling decreases with increasing temperature. At high temperatures, as solvent enters the network, a diluting force develops and the networks of chains expands. This expansion creates an elastic retractive force in the network to pull back the chains to their original positions. This elastic force is stored in the network as entropy. As more and more solvent enters the network, the retractive force increases and the diluting force decreases. When these two forces become equal, a state of equilibrium swelling is reached. In other words, at equilibrium swelling, the force of the solvent molecules pushing to get into the polymer network is the same as the force of the chains trying to pull themselves back together. However, as the temperature increases and then is maintained, more solvent enters, which makes the rubber particle stretch. Consequently, more work is needed to maintain the network and the energy associated with the molecular structure in the polymer network will be decreased. Therefore, the equilibrium of swelling will be at a lower volume of swelling and maximum volume of swelling will decrease with an increase in temperature. But if the constitution of the rubber is changed then the maximum amount of swelling will be increased with increasing temperature.

A study conducted by Green and Tolonen (1977) demonstrated that the rate of swelling increases with increasing temperature. When the reaction takes place above 155^0C, the amount of maximum swelling increases with increasing temperature because of the reversion of the rubber network. When this happens, the cross-link density is reduced and thus entropy required to expand the rubber network is reduced. Therefore, the maximum amount of swelling will increase. Singleton (2000) conducted swelling tests on crumb rubber particles using flux oil at different temperatures and found that maximum swelling increases almost proportionally with temperature and the rate of swelling increases but at a rate faster than the increase in temperature.

3.2.2.2 Effect of Particle Size

Using the Crank diffusion theory, Southern (1967) calculated the penetration rate of the swelling liquid into the rubber. As the boundary of the swollen and unswollen rubber is normally quite sharp, they assumed that all the absorbed liquid is contained in a layer of swollen rubber of uniform concentration. From this assumption, they obtained a simple formula for the rate of penetration;

$$P = \frac{M_t}{C_0 A \sqrt{t}} = \frac{l}{\sqrt{t}} \qquad \textbf{\textit{(3.1)}}$$

Where,

P	=	rate of penetration
M_t	=	mass of liquid absorbed
C_0	=	concentration of the liquid (g/cm^3)
A	=	surface area of the polymer
t	=	time
l	=	depth of the swollen layer

Equation 3.1 demonstrates that thin or small particles of rubber will swell more rapidly than thicker or bigger particles. Green and Tolonen (1977) during their research on the wet process also demonstrated that the rate of swelling increases with decreasing rubber particle size. Recent research on Impact Absorbing Asphalt at the University of Nottingham (Singleton, 2000) showed that the rate of swelling is faster for smaller particles and also the maximum amount of swelling is greater for smaller particles.

3.2.2.3 Effect of Liquid Viscosity

Southern (1967) investigated the effect of liquid viscosity on the penetration rate using a large number of natural rubbers. He found that the rate decreased as the viscosity of the swelling liquid increased but the total amount of liquid absorption does not depend only on the viscosity but also on the chemical nature of the swelling liquid. Figure 3.1 demonstrates that the penetration rate increases with decreasing viscosity of different solvents.

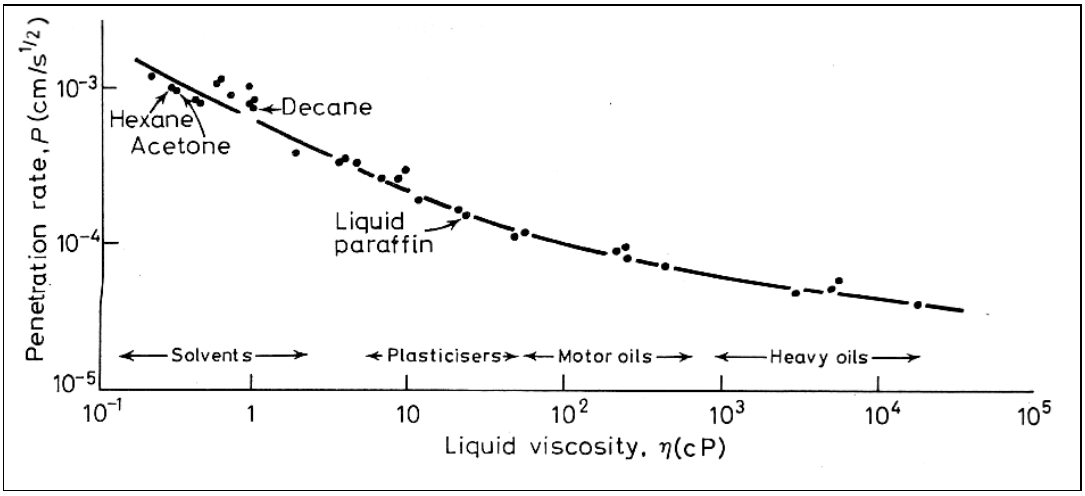


Figure 3.1: Effect of liquid viscosity on the penetration rate of liquid into natural rubber (Southern, 1967)

3.2.2.4 Effect of filler in Rubber

The influence of filler on swelling is relatively small compared with the effect of rubber-liquid interaction (Blow, 1971). However, Blow (1971) showed that increasing the amount of carbon black has some effect on decreasing swelling compared to other types of non-black fillers.

3.2.2.5 Effect of Concentration

The effect of liquid concentration could be expressed using Equation 3.1. It can be seen that the rate of penetration decreases with increasing liquid concentration provided that other variables such as time of reaction and surface area of the polymer are constant. Flory and Rehner (1943) found that the higher the activity level of the solvent, the greater the initial free energy available to cause the rubber to swell. The initial free energy represents the available work to swell, therefore, the greater the free energy, the greater the maximum amount of swell. Figure 3.2 shows how the concentration of polymer is related to the free energy in the system.

Figure 3.2 illustrates that as the volume fraction of the polymer is increased, the activity of the solvent decreases. Therefore, one would expect the maximum amount of swelling to decrease, with an increase in the polymer fraction or alternatively, a decrease in the concentration of solvent.

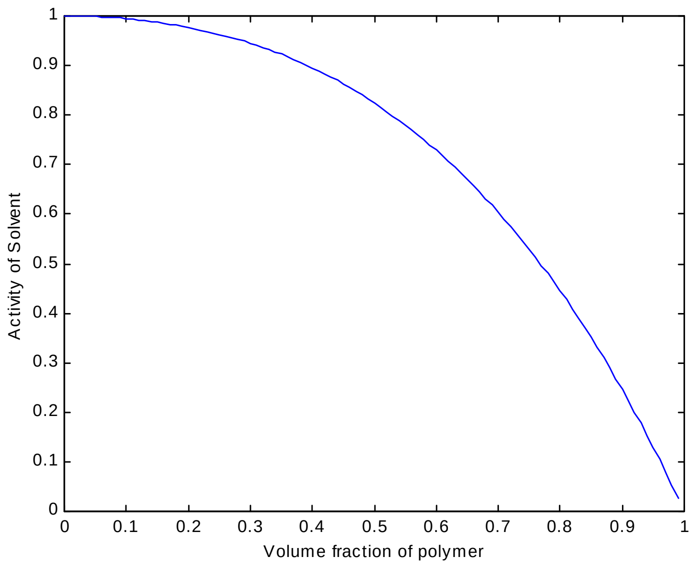


Figure 3.2: Activities of solvent dissolved in a cross-linked polymer as a function of the volume fraction of the polymer (Flory and Rehner, 1943, Part II)

3.3 WET PROCESS

3.3.1 Definition

The wet process was first developed by Charles H. McDonald (McDonald, 1981) and refers to the modification of bitumen with 5-25% by mass of fine tyre crumb at an elevated temperature. The wet process includes the blending of crumb rubber with bitumen at high temperatures and produces a viscous fluid through rubber-bitumen interaction (Takallou, 1988). The interaction process depends on a number of variables, such as blending temperature, blending time, type and amount of mechanical mixing, crumb rubber type, size and specific surface area of the crumb rubber and the type of bitumen. During the interaction process, the aromatic fraction of the bitumen is absorbed into the polymer network of the natural and synthetic rubber (Heitzman, 1991).

The method of blending rubber and bitumen can be divided into three categories: batch blending, continuous blending and terminal blending. In batch blending the batches of fine rubber crumb are mixed with bitumen during production. Continuous blending describes those wet process technologies that have a continuous production system. Terminal blending is associated with wet process technologies that have products with extended storage (shelf life) characteristics and are produced at bitumen supply terminals. Special care must be taken during mixing to ensure accurate proportioning of crumb rubber, right temperature and a uniform blend to enable an even reaction within the blend. Special pumps are required to ensure a uniform accurate binder discharge for mixture production (Heitzman, 1991).

3.3.2 Mixture Design Considerations

Construction of modified hot bituminous mixtures is typically the same as conventional paving procedures. The wet process is compatible with the Marshall and Hveem methods of mixture design although the stability is lower compared to conventional mixtures because of the 1-2% higher binder content (Heitzman, 1992). Typically, 15-22% by mass of binder, ground crumb rubber is used with a typical range of 1.2 mm to 0.075 mm (Heitzman, 1992). Grading requirements are finer for dense graded mixtures. An increased volume of mineral aggregate is required to maintain adequate air voids. Another requirement is to open the gradation of the mixture to allow room for the swelled rubber particles.

The mixing and laying temperatures are slightly higher due to increased bitumen viscosity. It should be noted that the blend of bitumen-rubber must be kept stirred or agitated to prevent stratification or separation of the crumb rubber modifier (Epps, 1994). The mixing temperatures ranges from 175-205^{o}C and the modified binder are then transferred to a heated reaction tank for 30-60 minutes at a temperature of 165-190^{o}C. In terms of compaction, pneumatic tyre roller compaction is not suitable for wet processed mixtures, as bitumen-rubber tends to stick to rubber materials. Steel-wheel drum rollers have been successfully used, with some liquid detergent added to the drum water to help lubricate the drum (Green and Tolonen, 1977, Takallou, 1988, OSU, 1997, Heitzman, 1992, Epps, 1994).

Recently mixture design using rubber-modified bitumen has come under scrutiny because of the discovery that the acid content in the bitumen (carboxylic acid) has a dramatic effect on the bitumen-rubber blend (OSU, 1997). Although, this may not be significant, it is difficult to overlook when developing a mixture design, as compatibility of the bitumen/polymer system is essential to produce durable mixtures. The compatibility of binder/polymer systems is determined by the structural arrangement of the polymer particles, chains or groups within the bitumen matrix (Green and Tolonen, 1977).

Studies conducted by Takallou and Sainton (1991) demonstrated that rubberised binder has to be used within 1 hour of its production as the interaction process continues after mixing while the mixture is at storage temperatures. The viscosity reaches its maximum after approximately 45 minutes, remaining high for 1 to 2 hours and then decreases. The assumed reason is that due to the thermal ageing, the cross-linked networks of the rubber are broken reducing the polymer to its original structure that causes degradation of the binder. They also found that storage time significantly increased if extender oil and catalysts (ethylene unsaturated polymer or copolymer) are used in the blend.

3.3.3 Performance of Wet Process

The performance of CRM mixtures using the wet process has been investigated by different researchers under both laboratory and field conditions. In terms of mechanical performance such as fatigue, the wet process showed more tolerance to higher deflections and presented a better resistance to fatigue compared to conventional mixtures (Harvey *et al*, 2000, Oliver, 2000). In Louisiana, similar results were observed after 5 to 7 years of trafficking using the wet process in gap graded surfacing and dense graded surfacing and binder courses (Huang *et al*, 2002). Oliver (2000) also found that mixtures produced with 6% or less binder contents had low and variable fatigue life. This is attributed to the absorption of bitumen components by the rubber particles rendering the mixture dry with a tendency to ravel. A laboratory study conducted by Khalid and Artamendi (2002) on Stone Mastic Asphalt (SMA) and DBM wearing courses demonstrated that compared to the control mixtures, the stiffness and resistance to permanent deformation improved with increasing rubber modified bitumen content in the mixture. In addition, whilst moisture sensitivity increased due to CRM modification, the retained stiffness values remain constant or slightly increase with increasing CRM content.

A test conducted by the Alaska DOT showed that the wet process had the best thermal cracking resistance, compared to dry and control sections (Saboundjain and Raad, 1997). Both laboratory and field trials indicated that asphalt rubber mixtures are less temperature susceptible than conventional mixtures.

Studies conducted in Texas and Nevada (Epps, 1994) suggest that mixtures containing crumb rubber and conventional mixtures have similar resistance to permanent deformation. But wet process mixtures used in Louisiana showed similar or lower rut depth than control sections (Huang *et al*, 2002). Similar results were found in laboratory studies. A Virginia study indicated that samples produced in the laboratory using the wet process had less resistance to permanent deformation, whereas in the field the same mixtures have performed as well as conventional mixtures (Maupin, 1992 &1996). On the other hand, eight years field observations in Oregon demonstrated that the wet process mixture had generally better rutting resistance compared to control mixtures (Hunt, 2002). In addition, field accelerated loading (ALF) conducted in Louisiana showed that the wet process applied in wearing course and base course mixtures improved the rutting resistance (Mohammed *et al*, 2000). Similar tests were conducted by the California Department of Transportation (Caltrans) on gap-graded overlay using different loading types (Harvey *et al*, 2000). The results demonstrated that CRM mixtures have superior performance under dual/radial loading and wide-base single loading conditions. A recent study using dynamic modulus testing showed improved performance in low temperature cracking and high temperature permanent deformation resistance (Kaloush *et al*., 2003).

In terms of functional properties, the wet process has been reported to provide better performance in terms of noise reduction, smoother surfaces, increased resistance to moisture damage and resistance to bleeding and flushing (Ballie and Roffe, 2000, Belshe and Turner, 2000).

3.4 DRY PROCESS

3.4.1 Definition

The process was first developed in Sweden in the 1960s and widely known in Europe under the name “Rubit”. It was then patented in the USA under the name “PlusRide” in 1978 (Heitzman, 1991). In this process, the crumb rubber is added to aggregate in a hot mix central plant operation before adding bitumen. The basis of the dry process is to use the crumb rubber to replace a percentage of aggregate and modify the grading. During the production to laying stage some reaction is known to take place depending on the gradation of the crumb rubber (Buncher, 1994). However, evidence from work carried out by Green and Tolonen (1977) has shown that the degree of reaction is also affected by the time the mixture is held at the mixing temperature, the proportion of rubber to bitumen and the type of bitumen. As the reaction time is much shorter and the grading size of crumb rubber is larger with less specific surface area, it was deemed unimportant to the properties of the material compared to the wet process (Takallou, 1988). However, recent research at University of Nottingham (Singleton, 2000) demonstrated that the interaction of rubber-bitumen makes the bitumen stiffer and more elastic and, consequently, changes the properties of the asphalt mixture.

There are three main types of dry process technology used in the industry. These are:

- PlusRide (patented)
- Generic (not patented)
- Chunk rubber (not patented)

3.4.1.1 PlusRide

PlusRide is a modified gap-graded mixture with replacement of up to 3% aggregate by mass of the total mixture by granulated crumb rubber. Aggregate gradation, CRM content and gradation (typically 2 to 6.3 mm) are relatively fixed by the patent description. The bitumen content is typically 7-9%.

The underlying philosophy used with the PlusRide dry process is to limit the time that the bitumen and crumb rubber are maintained at the mixing (reaction) temperature and by specifying a coarse granulated crumb rubber (cubical shape, low surface area and smooth sheared surfaces) with a low rubber-bitumen reactivity so that the rubber can maintain its shape and rigidity within the CRM asphalt mixture (Heitzman, 1992). The PlusRide Hot Mix Asphalt is designed to modify the stability of a gap-graded aggregate matrix using the elastic properties of crumb rubber and allowing a certain amount of binder modification (reaction).

Conventional mixture design such as Marshall stability and flow, could not be used because of the lower stability and higher flow of the mixtures. Therefore, the mixture is designed in terms of percentage of air voids, which is typically 2-4%. Traditional equipment is used for preparing the specimen with slight modifications. Compaction is similar to traditional hot mix asphalt, but compaction must continue until the PlusRide mat cools below 60^{o}C. Construction practice is vitally important, as poor production, placement, and/or compaction control would lead to premature failure of the pavement.

3.4.1.2 Generic Dry Technology

Generic dry process also known as the “TAK” system, was developed in the late 1980s to early 1990s to produce dense-graded and gap-graded hot mix asphalt mixtures. The concept was originated by Barry Takallou as a result of his research and practical experience with PlusRide (Takallou and Hicks, 1988). In this method, both coarse and fine crumb rubber are used to match aggregate grading and to achieve improved binder modification. The benefit is mainly based on the following two phenomena:

- CRM particles act as a flexible substitute to the aggregate they replace.

- Binder modification occurs from finer crumb rubber particles.

The process is prepared by adding up to 3% by mass of fine and course rubber particles to a dense graded mixture. Generally the generic dry process system uses lower amounts of crumb rubber and smaller sizes as compared to PlusRide. The size of rubber particle for this process is from 2 mm to 180 μm. The intention of the generic dry process is to use conventional aggregate gradations and a generic gradation of the mixture can be attained for the aggregate. The theory and construction guidelines are very similar to PlusRide technology. One of the advantages of this technology is that greater binder modification is possible by specifying smaller rubber particle sizes and the dry process combined with the hot mix asphalt production sequence may be sufficient to permit the crumb rubber and binder to achieve a substantial degree of reaction before placement and compaction of the mixture.

The disadvantage of this method is that the design process must determine the appropriate crumb rubber modified material gradation for the proposed mixture properties. However, at present, the grading flexibility is very much limited in most crumb rubber modified plants. If an unusual crumb rubber modified material grading is required, the gradation may not be attainable and also the cost of the mixture will be increased.

3.4.1.3 Chunk Rubber Process

The US Army Corps of Engineers Cold Region Research Engineering Laboratory (CRREL) developed this method to evaluate the ice-disbonding characteristics of asphalt paving materials (Heitzman, 1992). The typical range of rubber particles is 4.75 mm to 9.5 mm with 3, 6, and 12 percent by weight of aggregate. The mixture gradations are required to be adjusted to provide space in the aggregate matrix for the substitute rubber particles. The optimum bitumen content is 6.5 percent for 3 percent crumb rubber to 9.5 percent for 12 percent crumb rubber. Although this process has not yet been field evaluated, laboratory wheel test results indicate that the higher rubber content in the mixture could potentially increase the incidence of ice cracking (Oliver, 1981, Epps, 1994).

3.4.2 Material Handling and Storage

Both batch and drum plants can be used to produce dry process CRM mixtures. The reclaimed granulated rubber is usually packed and stored in 110 kg plastic bags. Additional manual labour and conveying equipment, such as work platforms, are needed in order to introduce the granulated rubber into the asphalt mixtures, regardless of the type of mixing plant used. A batch plant has a quality control advantage over a drum plant because the number of pre-weighed bags of granulated rubber can be easily counted prior to their addition into each batch. The bags may be opened and the granulated rubber placed on a conveyor, or the bags may be put into the pugmill or cold feed bin if low melting point plastic bags are used.

Control of the feeding of granulated rubber is necessary because the correct rubber content is critical to the performance of the asphalt mixtures when using the dry process. Such control is more difficult to maintain in a drum-dryer system, due to the nature of the feed operation. Some drum-dryer plants have used recycled asphalt concrete hoppers to feed the granulated rubber, although a number of agencies (NCHRP, CRREL) recommend that the rubber be introduced into the mixture through a central feed system. The process can be automated by the addition of a conveyor and hopper, plus scales to ensure accurate proportioning of the granulated rubber.

3.4.3 Construction Procedures of Dry Process Mixtures

Binder contents in dry process mixtures are typically 10 to 20 percent higher than those of conventional mixtures. During the laboratory mixing process, the granulated rubber is dry mixed with the aggregate before adding the bitumen. After compaction, the sample is cooled to room temperature. The air void content is determined after extrusion. The field compaction of CRM asphalt mixtures requires high compaction temperatures compared to conventional mixtures. In batch plants, a dry mix cycle is followed to ensure that the heated aggregate is mixed with the crumb rubber before bitumen is added and the mixtures are produced at 149°C to 177°C (Epps, 1994, FWHA, 1997). Laying temperature is at least 121°C and compaction starts immediately behind the spreader and continues until the mat temperature reaches 60^0C (Amirkhanian, 2001) to reduce the possibility of swelling of rubber (Takallou *et al*, 1986).

Parameters that must be monitored during mixing for dry process mixtures include rubber gradation, rubber percentage in the total mixture, rubber pre-reaction or pre-treatment, and time of plant mixing. Since dry process binder systems are partially reacted with the rubber, it is not possible to directly determine the properties of the binders. In addition, when using partially reacted asphalt-rubber binder, consideration should be given to the match between the aggregate gradation and rubber gradation to fit spongy, swelled rubber into spaces which are typically occupied by aggregate (Epps, 1994).

3.4.4 Performance of Dry Process CRM Asphalt Mixtures

Research conducted by Oliver (2000) on the dry process showed that mixtures produced using 8% bitumen and 2.5% rubber by mass of aggregate have at least three times better fatigue life compared to the wet process and eight times better than a control mixture produced with similar grading and bitumen content. Similar results were found in an Alaska laboratory test but no differences between control and test sections were noted in the field (Saboundjain and Raad, 1997). Both dry and wet CRM mixtures deform more and at a faster rate than conventional materials, possibly due to the lack of aggregate-to-aggregate contact and interlock in high CRM dense graded mixtures.

A recent study conducted by Oregon DOT (Hunt, 2002) showed that within six years of service CRM sections using the TAK system (1.5% to 2% rubber) and Plus Ride® process (3% weight by total mixture), performed worse compared to sections built using the wet process and conventional methods, even though the construction cost was increased by 50% to 100% over conventional mixtures. Only one section with dense graded dry process base course showed good performance. Most of the failures were in terms of alligator or block cracking and medium to high severity ravelling. The resistance to rutting was also unacceptable in the dry process sections. On the other hand, Louisiana (Huang *et al*, 2002) experienced similar or better fatigue and rutting resistance after 5 to 7 years of traffic on test sections built using Generic (2% crumb rubber by weight of total mixture and 40 mm surfacing) and PluseRide® (3% crumb rubber by weight of total mixture and 40 mm surfacing) dry processed mixtures. No visible sign of cracks were observed. Field trials in Kansas (Fager, 2001) demonstrated that the dry process in general produced mixed performance in terms of rutting and cracking. After five years in service, sections build using the dry process with an open graded mixture showed acceptable rutting but cracking had formed in all areas. It was concluded that the use of crumb rubber in hot mix asphalt is possible but not economically feasible.

Colorado DOT investigated two dry processes (Harmelink, 1999); the PlusRide® process and a process which involved just adding rubber to the mixture. Three PlusRide projects showed early distress in the form of ravelling; one PlusRide project performed well. After five years, the control and test sections performed equally, although the mixture cost per ton with 1% crumb rubber was 21% higher.

Ontario evaluated 11 rubber modified asphalt demonstration projects between 1990 and 1993 (Emery, 1995). Eight of the projects were built using the dry process and one with the wet process. The short-term performance indicated generally poor performance for the dry process CRM sections in terms of ravelling, considerable pop-outs, poor longitudinal and transverse joints. Life cycle costs indicated that the cost of dry process CRM mixtures was always higher than conventional mixtures.

Arkansas DOT studied CRM asphalt mixtures (wet and dry) to compare mixture design parameters of unmodified and crumb rubber modified hot mix asphalt concrete using Marshall and the SHRP level 1 Procedure (Gowda *et al*., 1996). In addition, performance related properties (permanent strain, resilient modulus) were studied for aged and gyratory compacted specimens. Although the mixtures were kept as consistent as possible (identical aggregate gradation, same binder type and the amount of rubber) to facilitate meaningful comparison within and between the mixture design types, the volumetric properties between mixture design types were not similar. CRM modified asphalt mixtures (wet and dry) showed improved permanent deformation resistance compared to unmodified mixtures. The dry process CRM mixtures showed a slight reduction in the permanent strain at 2% relative to 1% crumb rubber in the crumb rubber mixtures but mixtures with 3% crumb rubber exhibited substantially higher permanent strain. The resilient modulus and tensile strength of the dry process mixtures generally decreased with increasing bitumen content. A laboratory study conducted on the dry process by Khalid and Artamendi (2002) has demonstrated that using maximum 3 mm rubber particle with up to 10% by weight of total aggregate has an adverse effect on the volumetric, stiffness, durability and permanent deformation properties of the wearing course and road base mixtures. However, although the properties were slightly improved when rubber was pre-treated with extender oil prior to mixing, the overall performance was still inferior compared to control and wet processed mixtures.

Minnesota DOT studied five CRM test sections (dry process) and two control sections to investigate moisture sensitivity, low temperature behaviour, resilient modulus, tensile strength and permanent deformation (Stroup-Gardiner *et al.*, 1996). A 2.38 mm (No. 10) mesh crumb rubber from waste passenger tyres was pre-treated with a low viscosity petroleum-based product and used as an aggregate replacement in the asphalt mixture. Laboratory test results indicated that CRM mixtures would exhibit a greater ability to dissipate thermal stresses through a better ability to strain at cold temperatures. Some light to moderate improvements were seen in both moisture sensitivity and low temperature behaviour when the crumb rubber was treated. After two years, all the test sections performed well with no significant difference in the layer moduli and no damage due to snowplows.

3.5 SUMMARY

A brief literature review on rubber-solvent interaction and the use of crumb rubber in asphalt mixtures has been reported in this chapter. Fundamental issues such as the effect of temperature, viscosity and concentration on the interaction process together with the effect of particle size and presence of filler were discussed. The literature review suggested that when the rubber is added to the bitumen or solvent, it imbibes proportions of the solvent and swells. The amount of swelling mostly depends on the temperature, particle size, duration of the test, viscosity and complex chemical nature of the solvent, but was not significantly influenced by the presence of filler.

Two crumb rubber-modified asphalt mixtures, wet process and dry process, were discussed including their applications, type, and field performance. In the wet process, rubber is primarily used to increase the viscosity and elasticity of the bitumen and in the dry process larger rubber particles are used to become part of aggregate system. However, case studies on both dry and wet process revealed that the field performance of the dry process is not as consistent as the wet process, and the distresses are mainly in the form of cracking, ravelling and rutting. Although application of dry process is logistically identical to conventional mixtures and consumes larger quantities of waste tyres, only limited fundamental studies have been conducted to resolve those issues and therefore, it is less popular than the wet process.

As rubber absorbs bitumen at high temperatures and consequently may alter mixture performance, the work reported in this thesis deals with the factors that influence the interaction and the properties of bitumen, rubber-bitumen composites and dry process CRM asphalt mixtures. In addition, the durability of the dry process CRM asphalt mixtures is investigated to assess the performance and suitability of the material for future field application in UK. The next chapter presents an investigation into the rubber-bitumen interaction using different commercial bitumens and the influence of different variables such as temperature, concentration and reaction time on the interaction process.

CHAPTER 4

Rubber-Bitumen Interaction

4.1 INTRODUCTION

As described in section 3.2 in Chapter 3 that rubber swells at ambient temperatures in some solvents, such as flux oil, white spirit and dichloromethane. The reaction also occurs with bitumen but at higher temperatures where the viscosity of bitumen is reduced. In the dry process, crumb rubber is added directly to hot aggregate prior to mixing with bitumen and mixing is continued until full binder coating is achieved before the mixture is discharged to the insulated trucks ready for transportation (Visser and Verhaeghe, 2000). The total transportation time from the mixing plant to the actual job site could be up to 6 hours. During transportation, the mixture is kept in the insulated truck, where temperatures range between 150^0C to 177^0C. Depending on the type of bitumen used, a typical laying temperature is at least around 120^0C and the compaction continues until the mixture temperature drops below 65^0C (Epps, 1994, Amirkhanian, 2001). Most of the previous work has considered the reaction between the rubber and the bitumen to be negligible due to the shorter reaction period and larger particle sizes used in the dry process. Consequently, previous work on rubber-bitumen interaction was most likely carried out with reference to the wet process, which is fundamentally different from the way rubber is used in the dry process. Singleton (2000) and Airey *et al* (2002) found that the interaction of rubber-bitumen is significantly higher in the dry process than previously thought. Singleton (2000) conducted limited research where rubber and bitumen were allowed to react in a beaker for different periods of time. The research noted that the interaction reduces the amount of available binder in the mixture as rubber absorbs the lighter fractions of bitumen, consequently making the bitumen stiffer and more elastic that results in a less durable mixture in service. However, it is still not known how this interaction occurs in the dry process and how temperature, bitumen constitution and type of rubber affect swelling. In addition, it is almost impossible to recover partially reacted bitumen directly from CRM mixtures using conventional binder recovery techniques as the rubber reacts with the solvent used in the recovery process, thereby, altering the chemical composition of both the bitumen and rubber (Epps, 1994). Previous trials suggested that the length of time the mixture is held at mixing temperature affects the mechanical durability of the material (Singleton, 2000). In addition, the literature review revealed that the swelling of rubber largely depends on the mixing temperature (Green and Tolonen, 1977, Singleton, 2000), viscosity and chemical nature of the solvents (Southern, 1967).

This chapter concentrates on the process of rubber-bitumen interaction and outlines the experimental investigation of some parameters, like curing time, types and grades of binders and rubber-bitumen concentration that affect the amount and rate of absorption by the rubber. Particular emphasis has been given to relate the interaction between the rubber and bitumen to what happens during mixing, transportation and compaction of dry process CRM material, when the mixture is held at an elevated temperature. A simple test procedure has been developed to conduct rubber-bitumen interaction tests and is presented in this chapter.

In the first section, the consistencies of different rubber particles are examined to assess the chemical similarity among crumb rubber batches, which are produced by tyre recycling at different times. In the next section, rubber-bitumen interaction testing methodology and tests results are presented in terms of the effect of temperature, rubber-bitumen concentration, bitumen types and grades. The final sections of this chapter mainly discuss the findings with a brief summary at the end.

4.2 MATERIAL AND TESTING VARIABLES

Rubber produced from truck tyre ranging from 2 to 8 mm used in this investigation was collected from Charles Lawrence Recycling Ltd. It can be noted that the sizes of the rubber particles lie in between ground and crumb rubber classification (see section 2.2.4.2). However, for simplicity the terminology "crumb rubber" was adopted for this project to identify the rubber particles ranges from 2 to 8 mm. The gradation of the crumb rubber is presented in Table 4.1. Eight penetration grade bitumens (denoted as M1 to M4 and V1 to V4) from two different crude sources (Middle East and Venezuelan) were included in this bitumen-rubber study. The physical and chemical properties of the penetration grade bitumens are presented in Table 4.2. The penetration of the bitumen was determined using BS 2000:Part 49 and the chemical compositions were determined using BS 2000:Part 143:1996 to determine the asphaltenes content in the bitumen.

Table 4.1: Granulated crumb rubber gradation

Sieve Size (mm)	Cumulative Percentage Passing (%)
9.5	100
6.3	70
2.36	30
1.18	0

Although the binders from the two crude sources were chosen to have the same nominal penetration grade, the Middle East binders were marginally harder than their comparative Venezuelan binders. In terms of chemical composition, the four Middle East binders have slightly lower asphaltenes and, therefore, higher maltenes (light fractions) compared to the Venezuelan bitumens. Each of the eight binders were mixed with a 5g sample of crumb rubber at three rubber-bitumen ratios of 1:4, 1:6 and 1:8, as described in Section 4.3 in the basket drainage testing methodology, and then subjected to high temperature swelling tests (curing tests) using the basket drainage bitumen absorption method (see Section 4.3). The ratio was chosen to ensure enough bitumen to coat all rubber particles and maintain this coating throughout the testing period.

Table 4.2: Physical and chemical composition of bitumen

Binder types	Bitumen Source	Penetration Grade	Penetration (dmm)	Asphaltenes (%)	Maltenes (%)
M1	Middle East	160/220 pen	192	8	92
M2		70/100 pen	88	11	89
M3		40/60 pen	53	14	86
M4		35 pen	32	18	82
V1	Venezuelan	160/220 pen	203	10	90
V2		70/100 pen	90	11	89
V3		40/60 pen	55	13	87
V4		35 pen	36	19	81

The bitumen absorption tests were performed in two series. In the first series, the testing temperature was kept constant at 160^0C to simulate the mixing and transportation temperatures typically used in dry process CRM asphalt mixtures production (Epps, 1994). In the second series, the viscosity of the bitumen was kept constant at 0.2Pa.s to simulate typical hot mix asphalt mixing viscosities (Whiteoak, 1990, ASTM D 2493). High temperature viscosities (Figure 4.1) of the binders were measured over a temperature range of 100 to 200^0C using a rotational viscometer and temperatures corresponding to 0.2Pa.s are presented in Table 4.3. It is interesting to note that, although the binders from the two crude sources have approximately the same penetration grades, the M4 bitumen has a 15^0C higher equiviscous temperature than V4 bitumen.

Table 4.3: Equiviscous temperature of bitumen used in the study

Binder types	Penetration grade (dmm)	Equiviscous temperature (^{0}C)
M1	160/220 pen	140
M2	70/100 pen	150
M3	40/60 pen	170
M4	35 pen	180
V1	160/220 pen	135
V2	70/100 pen	150
V3	40/60 pen	165
V4	35 pen	165

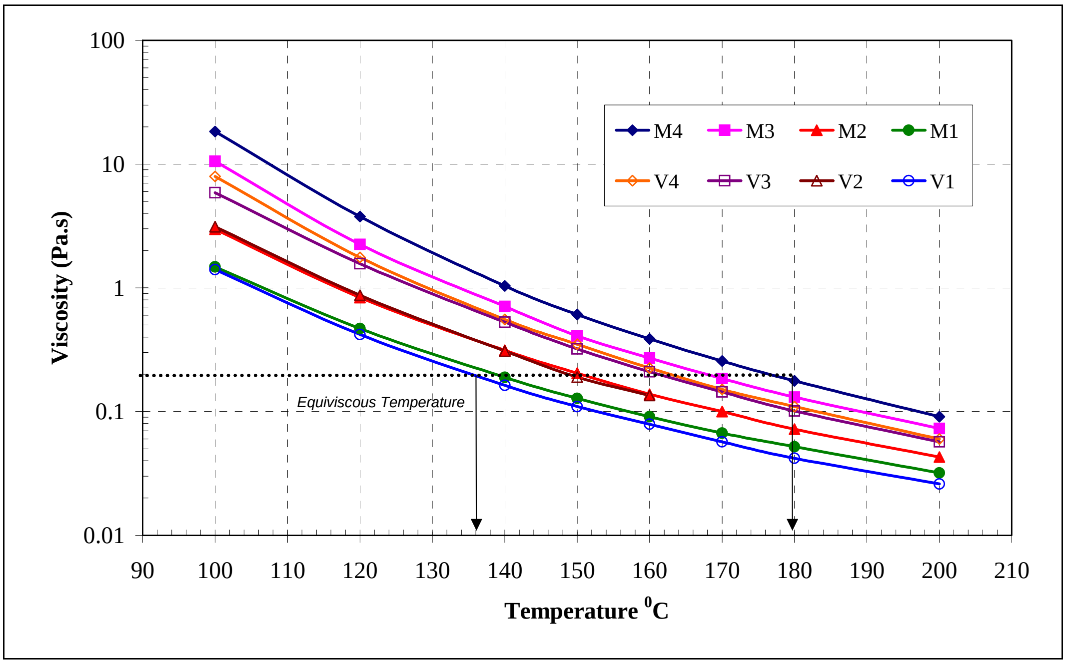


Figure 4.1: High temperature viscosity

4.3 BASKET DRAINAGE TESTING METHODOLOGY

The amount of swelling (bitumen absorption) and the rate of swelling of the crumb rubber were calculated by measuring the mass of the rubber before and after rubber-bitumen interaction at 1, 4, 24 and 48 hours. Increases in rubber mass and not rubber volume were calculated, as gravimetric measurements were considered easier to obtain than volumetric measurements. In addition, Green and Tolonen (1977) found that measuring the increase in mass for their rubber swelling tests was more accurate than measuring the increase in volume.

The swelling tests consisted of placing 5g of the crumb rubber particles into wire mesh baskets and then placing the baskets into 400ml beakers as illustrated schematically in Figure 4.2. The baskets were constructed with 0.28 mm square aperture steel mesh with cross sectional dimensions of 50 mm by 50 mm and a height of 50 mm. Bitumen was then heated for two hours at 160°C (or equiviscous temperatures) in a fan-assisted oven and fixed quantities of the binder poured into each of the beakers. The proportion of rubber to bitumen was altered by changing the mass of bitumen with three quantities of 20, 30 and 40 g of bitumen being added to the 5 g of rubber to give rubber-bitumen ratios of 1:4, 1:6 and 1.8.

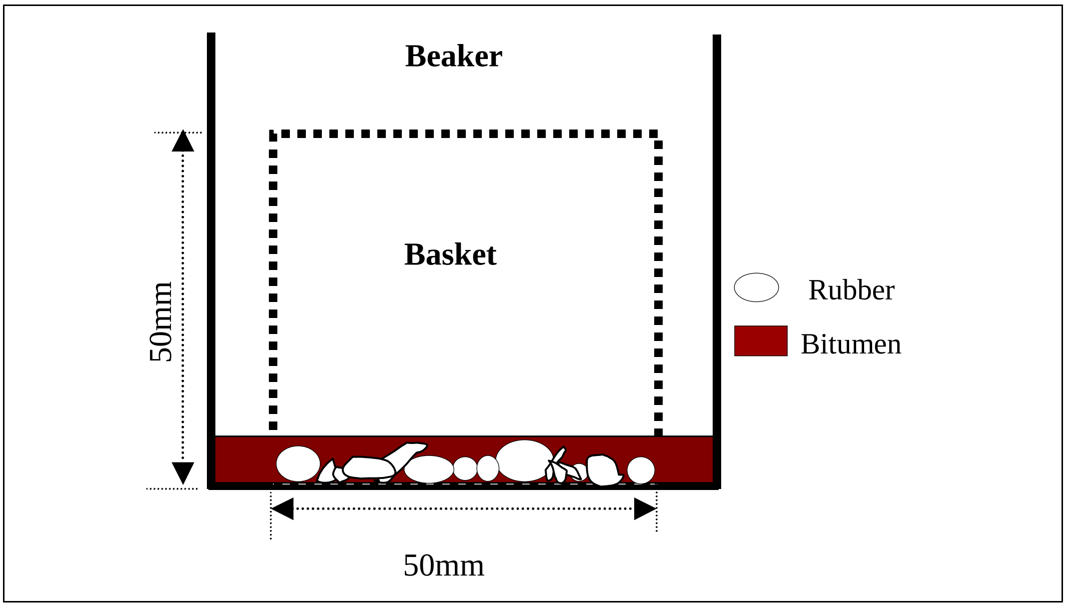


Figure 4.2: Schematic of the basket drainage test

Immediately after the bitumen was added to the beakers, the baskets were removed and suspended above the beakers in a fan-assisted oven at 160°C (or equiviscous temperature) for five minutes to allow the bitumen to partially drain from the crumb rubber and the bitumen coated wire mesh. The purpose of this initial drainage period was to determine the initial mass of the bitumen coated rubber (no absorption and no swelling) plus wire mesh basket, which could then be used as the reference mass to ascertain the increase in rubber mass due to bitumen absorption during the swelling test. The baskets were then re-placed into the beakers with the rubber particles being completely submerged in bitumen. The removal, five minutes drainage and weighing procedure were then repeated at test intervals of 1, 4, 24 and 48 hours. At the end of 48 hours, the bitumen was drained and the residual bitumen either subjected to asphaltene content or rheological property testing or stored at 5°C for later analysis.

In addition to the three beakers containing crumb rubber, a fourth beaker containing 30 g of bitumen and no rubber was included as a bitumen ageing (curing) control. A total of eight sets of these four beakers (three rubber-bitumen concentrations and one control) were prepared for each rubber-bitumen combination and cured at a temperature of 160°C (or equiviscous temperature) for a period of 48 hours. Finally, swelling curves were produced based on binder absorption to give an indication of the rate of interaction taking place in the mixing, transportation and compaction (i.e. at elevated temperatures) period. The physical, chemical and rheological properties of the bitumen following rubber-bitumen interaction are presented in Chapter 5.

4.4 SWELLING TEST RESULTS

4.4.1 Consistency of Crumb Rubber

As mentioned in Chapter 2, crumb rubber is primarily composed of natural and synthetic rubbers with carbon black as filler and other additives to provide strength, toughness and elasticity. Although the fundamental properties of the truck tyres are similar, the proportion of these ingredients varies depending on the tyre manufacturer and the process of manufacturing. Therefore, it is fundamentally important to check the consistency of the crumb rubber used in this project as this might influence the accuracy of the rubber-bitumen interaction test. The main purpose of this study was to perform a quick assessment of the swelling potential of different crumb rubber particles produced from scrap truck tyres.

Five different batches of crumb rubber were selected from one single source with similar gradation and size but produced at different periods over six months. The test was conducted with M3 bitumen at a rubber to bitumen ratio of 1:6. The percentage increase in rubber mass (bitumen absorption) for the five batches after 1, 4, 24 and 48 hours is presented in the form of a bar chart in Figure 4.3. The results indicate that the swelling potential (bitumen absorption) of the different batches of 2 mm to 8 mm truck tyre crumb rubber is not significantly different. In addition, it also suggests that the crumb rubber of this particular source has consistent chemical and physical properties. However, to ensure that the consistency of the crumb rubber is maintained throughout the study, rubber samples from only one batch were used in this laboratory investigation.

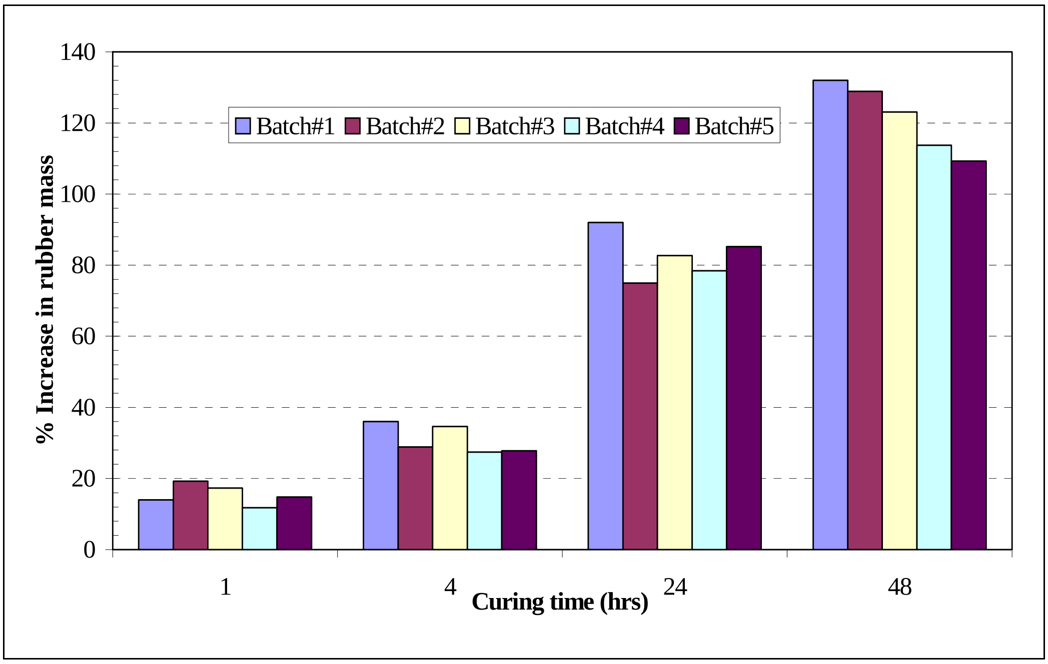


Figure 4.3: Absorption of bitumen M3 at a rubber-bitumen ratio 1:6 for five batches of crumb rubber produced from scrap truck tyre and obtained from a single source

4.4.2 Bitumen Concentration

The binder absorption (rubber swelling) results for bitumen V3 tested at 160^{0}C with the three rubber to binder ratios of 1:4, 1:6 and 1:8 are presented in Figures 4.4 and 4.5. The graphs are plotted in terms of percentage increase in rubber mass and percentage bitumen absorbed versus curing time. For brevity only the absorption data for bitumen V3 has been shown although similar plots were obtained for the other seven binders and presented in Appendix B. Due to the high repeatability of the test procedure, as demonstrated by previous studies (Singleton *et al*., 2000a; Airey *et al*., 2002), only one sample was used for each rubber-bitumen combination.

It can be seen in Figure 4.4 that for all three rubber to bitumen ratios there is an increase in rubber mass (rubber swelling through absorption of the light oil fractions of the bitumen) with curing time. However, this increase in rubber swelling is not uniform with the rate of absorption decreasing with curing time as shown by the logarithmic trend lines.

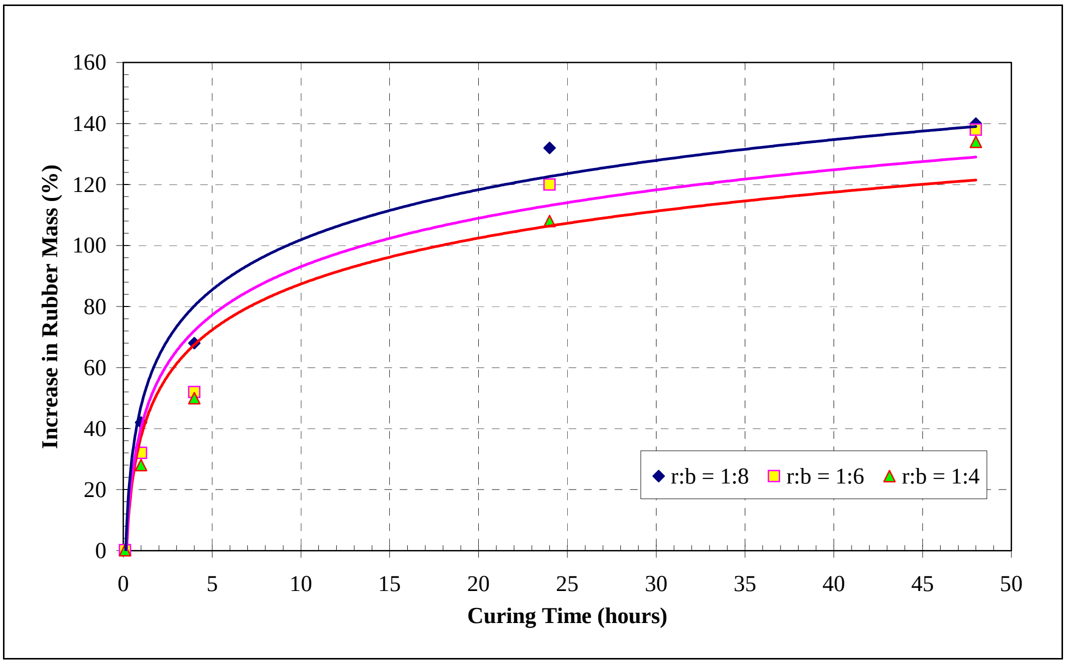


Figure 4.4: Increase in rubber mass for bitumen V3 at 160^{0}C at three rubber-bitumen ratios

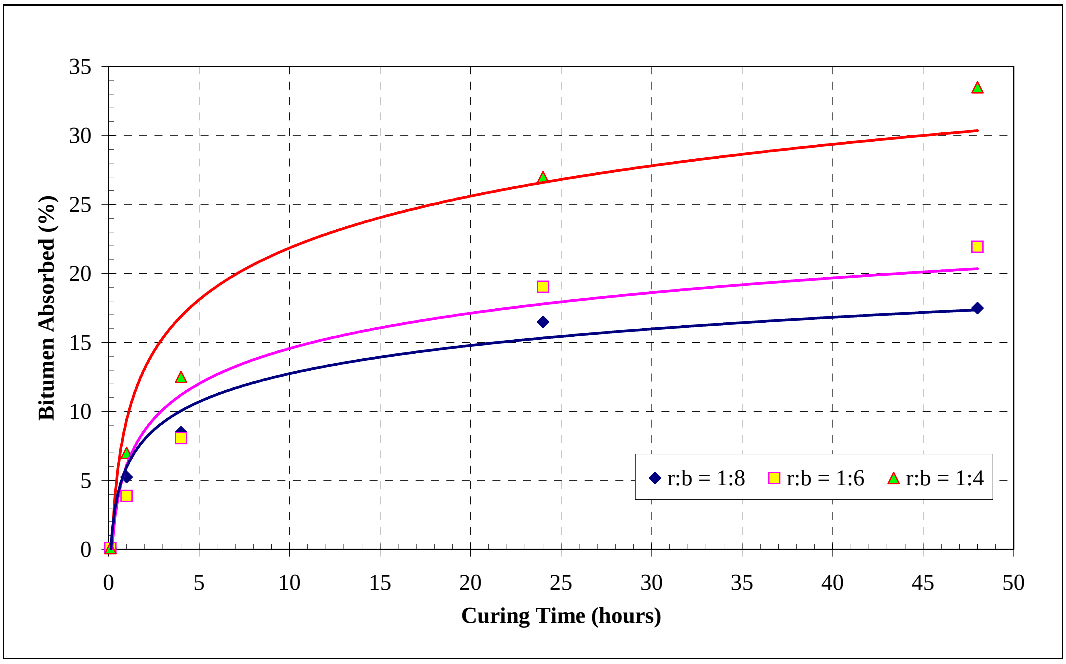


Figure 4.5: Absorption of bitumen V3 at 160^{0}C at three rubber-bitumen ratios

These results are similar to those seen for rubber cured in a styrene-butadiene-styrene (SBS) polymer modified bitumen (PMB) (Singleton *et al*., 2000b; Airey *et al*., 2002) and support the theory that a solvent (i.e. the maltenes fraction of bitumen) will diffuse into rubber until an equilibrium condition is reached or until the source of solvent is depleted. An important observation, based on the data in Figure 4.4, as well as swelling data for the other seven bitumens, is that although there is a slight decrease in swelling with a decrease in bitumen mass (i.e. rubber-bitumen ratio 1:4 compared to 1:8), the increase in rubber mass versus curing time is very similar for all three ratios. The negligible difference in swelling for the three ratios is indicative of a maximum threshold in terms of the amount of bitumen that the rubber is physically able to absorb as a proportion of its own mass. This suggests that, although there may be more bitumen available for absorption, the maximum increase in mass of this particular source of crumb rubber is approximately 140 percent at 160^{o}C. Or stated differently, provided that there is a sufficient quantity of bitumen available for absorption, the rubber will swell to approximately 140 percent of its original mass.

Figure 4.5 shows the swelling results for bitumen V3 plotted in terms of percentage-absorbed bitumen versus curing time. As the rubber has absorbed approximately the same amount of bitumen at each of the three ratios, the percentage bitumen absorbed is understandably higher for the lower initial bitumen mass (ie rubber-bitumen ratio 1:4 with 20g of bitumen) compared to the higher bitumen masses. Previous research has shown that as it is the lighter fractions (maltenes) that are absorbed by the rubber, most bitumens will generally have more that 50 percent of their mass available for absorption (Airey *et al.*, 2002b; Treloar, 1975; Vonk and Bull, 1989). As less than 35 percent of the bitumen in the 1:4 ratio test has been absorbed after 48 hours, it can be deduced that all three rubber-bitumen ratios have sufficient bitumen to allow the swelling process to progress to the maximum absorption threshold of the rubber. Therefore, as the swelling data is similar for all three rubber-bitumen ratios, only the 1:6 test data has been used to investigate the influence of crude source and penetration grade (viscosity) on the high temperature rubber-bitumen interaction.

4.4.3 Bitumen Crude Source and Grades

The influence of bitumen crude source and penetration grade was investigated at 160^{0}C and at equiviscous (0.2Pa.s) temperatures to compare the effect on rubber-bitumen interaction due to an increase or decrease in temperature from 160^{0}C. The rubber to bitumen ratio for this part of the investigation was chosen as 1:6 to ensure enough bitumen in the system for the duration of test. The summaries of the test results in terms of test duration and percentage increase in rubber mass are presented in Table 4.4 and 4.5 (presented at the end of this Chapter) and are plotted in Figures 4.6 to 4.9.

4.4.3.1 Interaction at Constant Temperature Testing

The increase in rubber mass against curing period for the Middle East and Venezuelan bitumens are shown in Figures 4.6 and 4.7. In general, the crumb rubber cured (soaked) in the higher penetration grade (lower viscosity) bitumens (M1, M2, V1, V2) tends to absorb more of the bitumen and swell (increase in mass) to a greater extent than the rubber cured in the harder, low penetration grade bitumens (M3, M4, V3, V4).

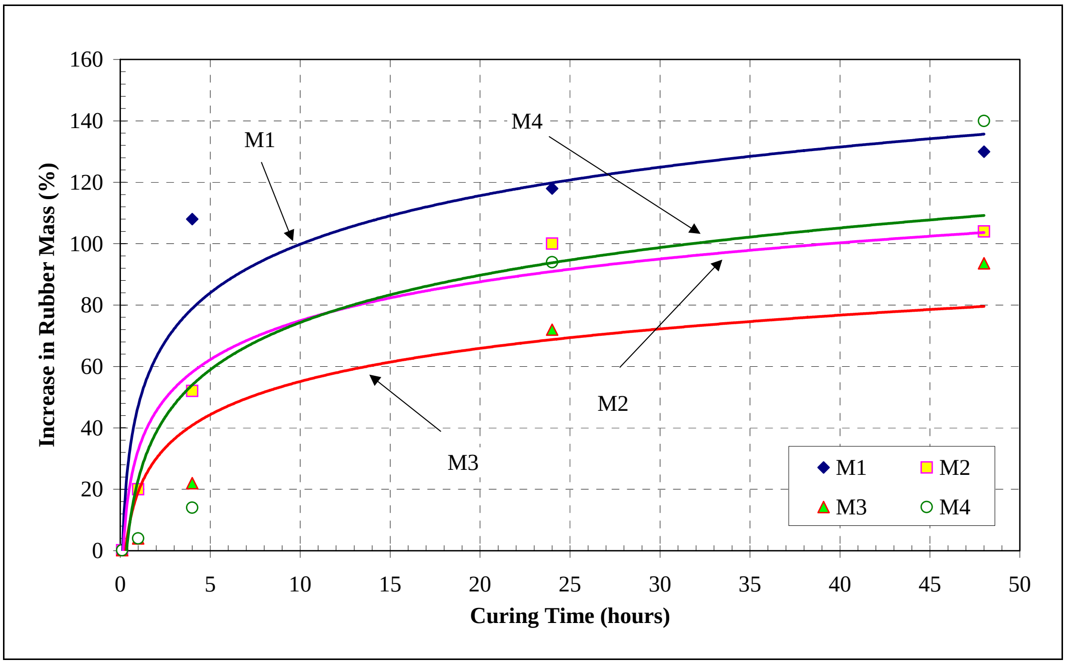


Figure 4.6: Increase in rubber mass for Middle East crude bitumen tested at 160^{0}C with rubber-bitumen ratio 1:6

However, this finding is inconclusive as for both the Middle East and Venezuelan crude sources, one of the harder bitumens (M4 and V3) showed comparable rubber swelling behaviour to that found for the softer binders. But in general, with regard to the influence of crude source on rubber swelling, both the Middle East and Venezuelan bitumens showed similar results, with the increase in rubber mass being between 80 and 140 percent of its original mass. Therefore in terms of the maximum amount of swelling, the influence of crude source and penetration grade seems to be less marked with the increase in rubber mass after 48 hours being possible more dependent on the nature of the crumb rubber (size, type etc) than the viscosity and nature of the bitumen. However, these findings were further investigated using constant viscosity swelling tests.

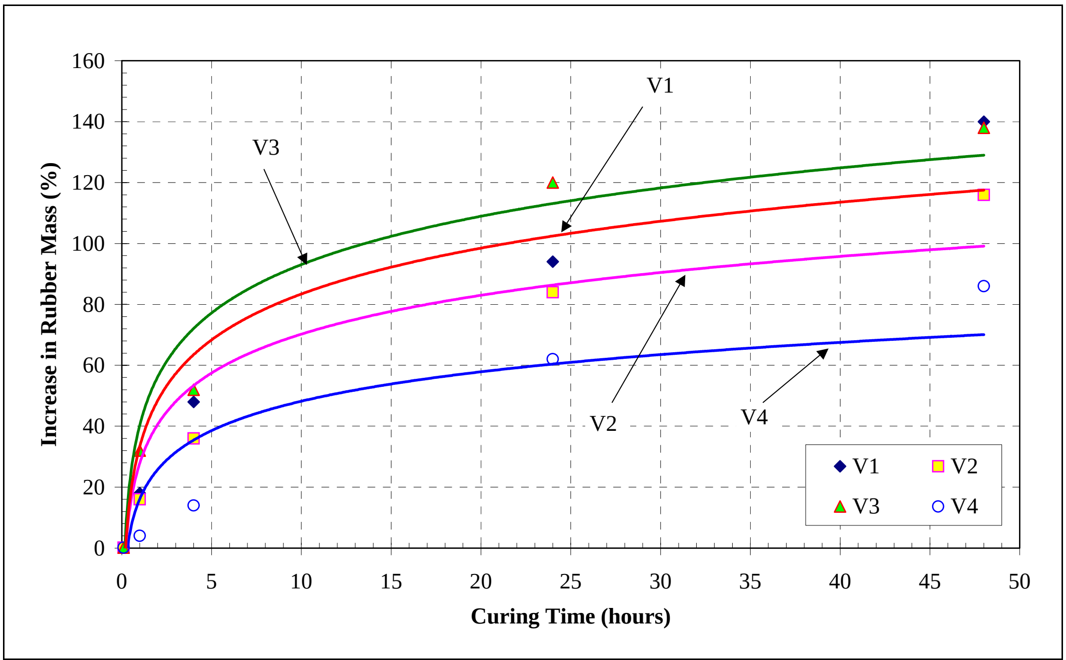


Figure4.7: Increase in rubber mass for Venezuelan crude bitumen tested at 160^0C with rubber-bitumen ratio 1:6

4.4.3.2 Interaction at Equiviscous Temperature Testing

The results obtained from constant viscosity swelling tests are presented in the Figures 4.8 and 4.9. In general, with the exception of M4 bitumen, the maximum amount of swelling was approximately 80% after 48 hours curing for all penetration grade bitumens irrespective of their crude source. The M4 bitumen showed more than 100% swelling, probably due to the very high testing temperature (180^0C), which may have degraded the molecular structure of the rubber crumb and, therefore, decreased the cross-link density allowing the rubber to absorb more bitumen. Another important observation is that the chemical and physical composition of different bitumens is less important (as long as there is enough aromatics in the system) compared to the testing temperature. In addition, the softer bitumens tended to be more affected by temperature variation. For example, results presented in Tables 4.4 and 4.5 show that after 48 hours curing, the amount of swelling increased up to 57% for V1 bitumen and 47% for M1 bitumen due to differences of 25^0C and 20^0C between constant temperature and constant viscosity testing. On the other hand, the amount of swelling increases by up to 23% for M4 bitumen due to a 20^0C difference in temperature. This indicates that as softer bitumens have more aromatic fractions compared to harder bitumen (Read and Whiteoak, 2004), they are more susceptible to temperature variations.

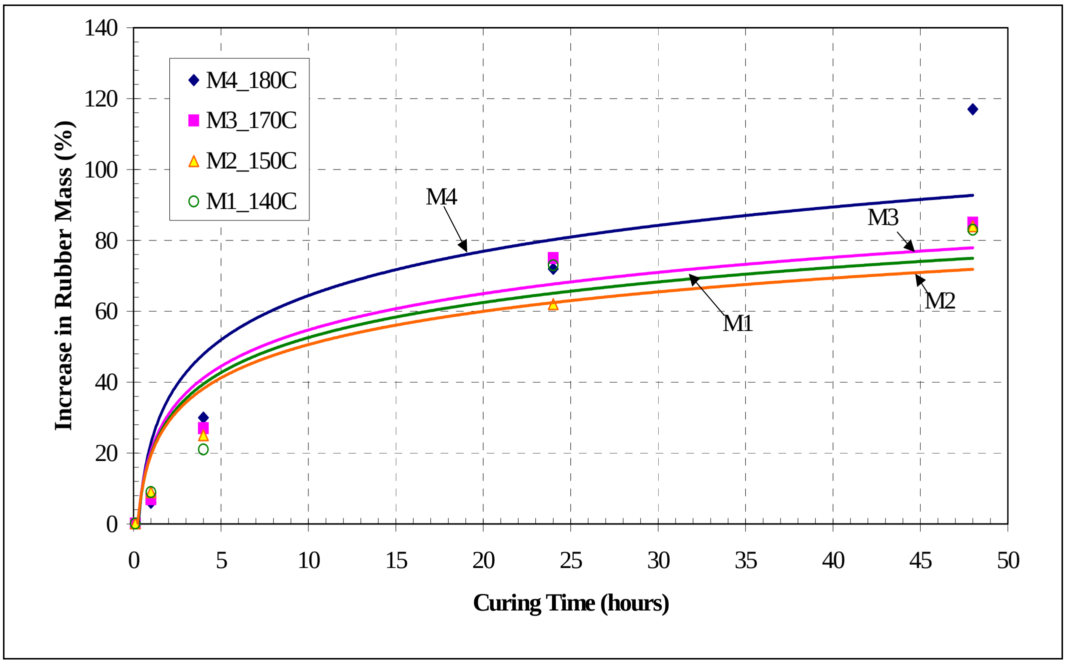


Figure 4.8: Increase in rubber mass for Middle East crude bitumen tested at equiviscous temperature with rubber-bitumen ratio 1:6

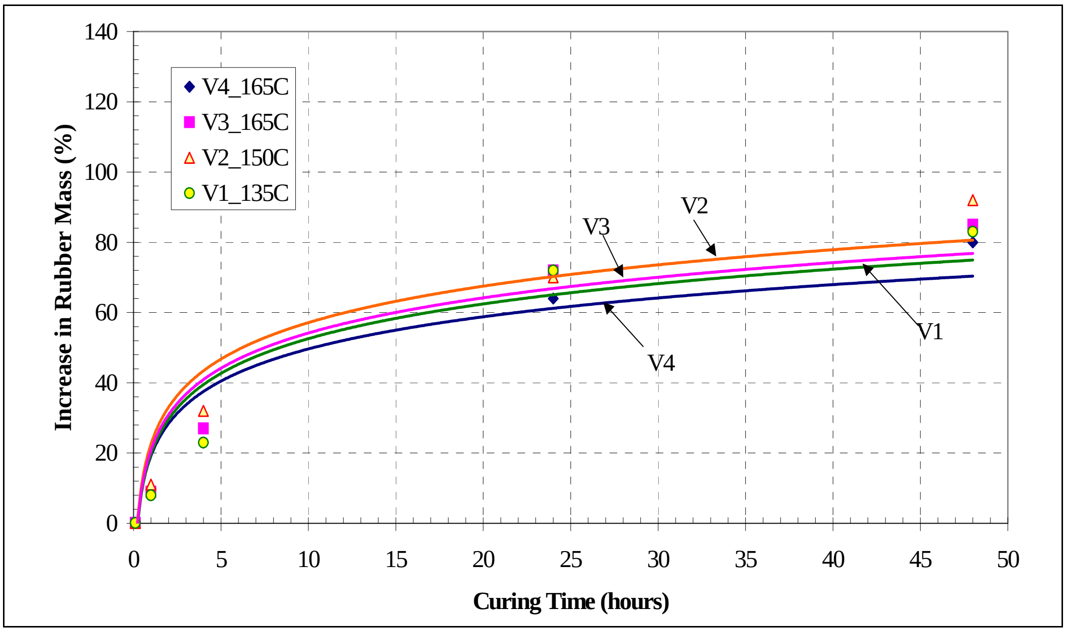


Figure 4.9: Increase in rubber mass for Venezuelan crude bitumen tested at equiviscous temperature with rubber-bitumen ratio 1:6

4.4.4 The Rate of Swelling

The influence of bitumen penetration grade and crude source on the rate of absorption (swelling) has been depicted in Figures 4.10 and 4.11 as the absorption rates (calculated as the average absorption rate between the time intervals of 0 to 1 hour, 1 to 4 hours and 4 to 24 hours and plotted at the midpoints of these time intervals) versus time for the eight binders.

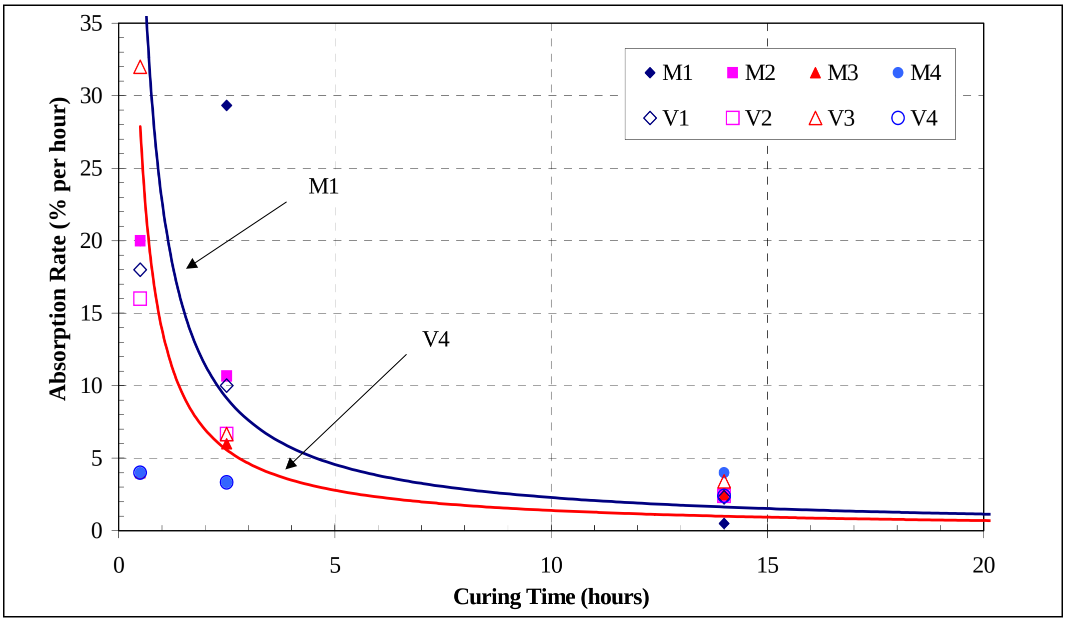


Figure 4.10: Average absorption rate versus curing time for Middle East and Venezuelan crude bitumens at 160 °C at a rubber-bitumen ratio of 1:6

In addition to the individual points, the logarithmic trend lines in Figures 4.6 to 4.9 have been differentiated to obtain trend lines of absorption rate versus time. Two of these trend lines for bitumens M1 and V4 (representing a soft and a hard bitumen respectively) have been included in Figures 4.10 and 4.11. It can be seen that the trend lines differ significantly from the real data due the lack of fit between some of the logarithmic best-fit curves in Figures 4.6 to 4.9 and the real data. However, the results show that for the tests conducted at a constant temperature of 160^{0}C, initially (after 1 to 4 hours) the rate of absorption is greater for the softer, high penetration grade bitumens (M1, M2, V1, V2) compared to the harder, low penetration grade bitumens (M3, M4, V3, V4) with the rates becoming more comparable after 24 hours. On the other hand, for the tests conducted at equiviscous temperatures the rate of absorption (Figure 4.11) is comparable for all bitumen grades at all stages of testing and the percentage of absorption is less compared to constant temperature tests.

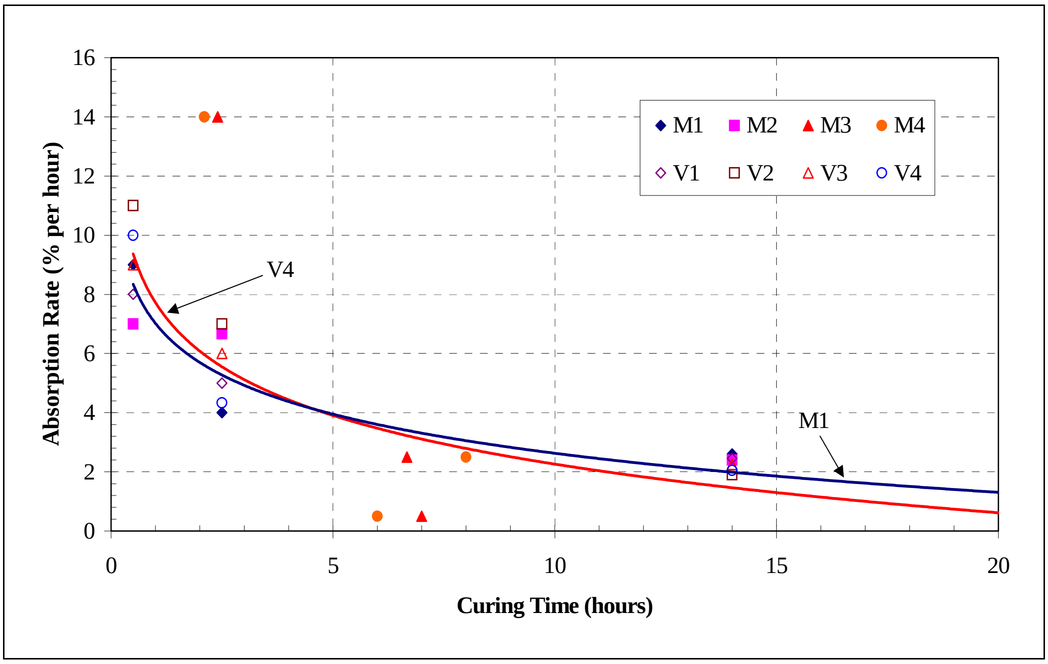


Figure 4.11: Average absorption rate versus curing time for Middle East and Venezuelan crude bitumens at equiviscous temperature at a rubber-bitumen ratio of 1:6

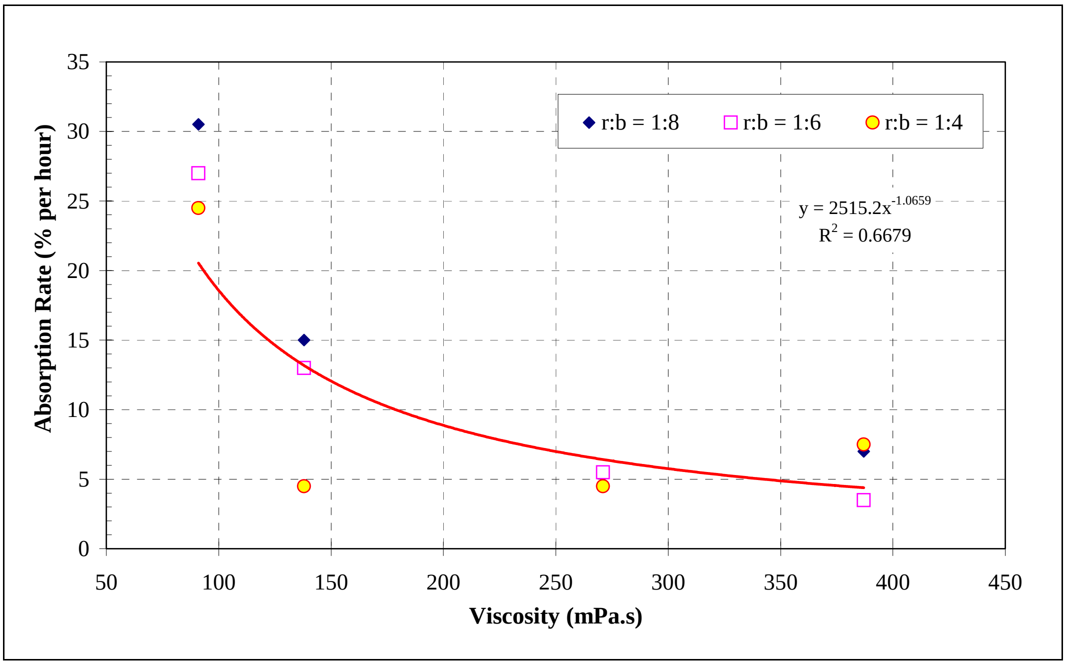


Figure 4.12: Relationship between the absorption rate of the Middle East bitumens after 150 minutes and the viscosity of the different penetration grade bitumens at 160°C at three rubber-bitumen ratios

Comparing crude source, the absorption rates after 1 and 4 hours tend to be slightly higher for the Middle East bitumens compared to the Venezuelan bitumens, probably due to the slightly lower asphaltenes and, therefore, higher maltenes content of the Middle East bitumens as shown in Table 4.2.

As the swelling of rubber in bitumen has been shown to be dependent on the viscosity of the liquid/solvent (bitumen) and the nature of the solvent (crude source, see page 91-92), a relationship between absorption rate at two and half hours and initial binder viscosity at 160°C has been established in Figures 4.12 and 4.13. The relationship between absorption rate and viscosity in Figure 4.12 for the four Middle East bitumens, at the three rubber-bitumen ratios of 1:4, 1:6 and 1:8, indicates that the rate of absorption of the "soft" 160/220-penetration grade bitumen is approximately four times greater than that of the "hard" 35 and 40/60 penetration grade bitumens.

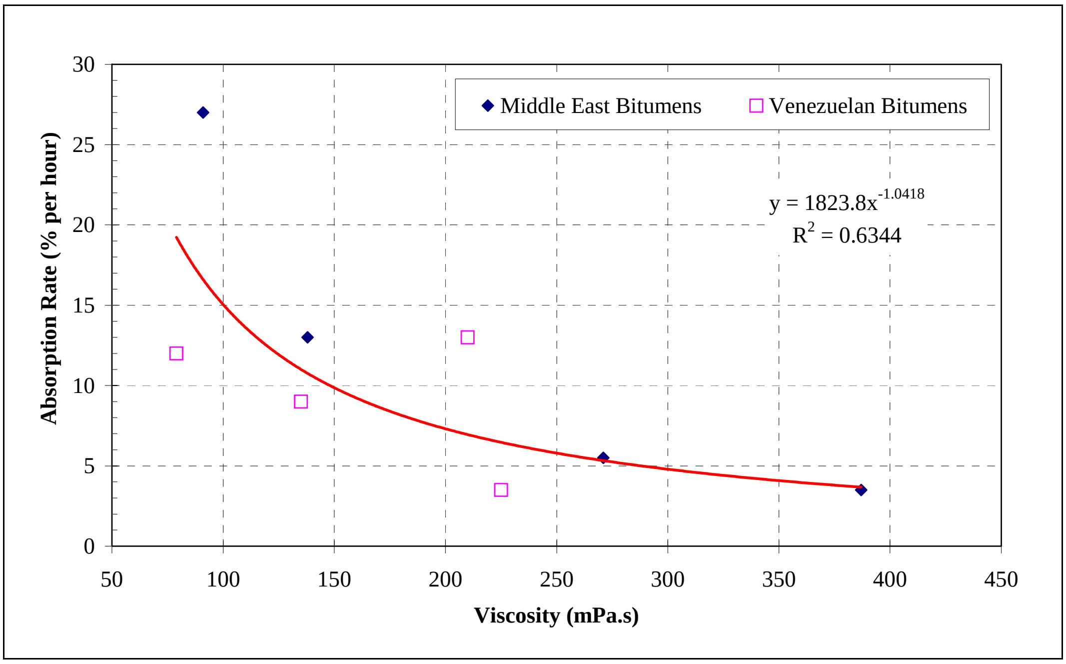


Figure 4.13: Relationship between the absorption rate of the Middle East and Venezuelan bitumens after 150 minutes and the viscosity of the different penetration grade bitumens at 160°C at 1:6 rubber-bitumen ratio

Figure 4.13 shows a combined relationship between absorption rate and initial binder viscosity for both the Middle East and Venezuelan bitumens at a rubber-bitumen ratio of 1:6. The results again show the higher rates of absorption for the softer binders compared to the lower penetration grade bitumens. In addition, although the penetration grades for the Middle East and Venezuelan bitumens are similar (within each of the four specification limits) as shown in Table 4.2, their high temperature viscosities differ considerably particularly for the harder binders. This implies that the absorption rates of the low penetration grade Venezuelan bitumens will be greater than that of the nominally identical (similar penetration values at 25°C with similar asphaltene and maltene percentages) Middle East bitumens.

4.5 DISCUSSION

Polymers, such as rubber, are known to absorb solvent and swell with the amount they absorb being dependent on the temperature, viscosity, and chemical nature of the solvent. In addition, the molecular arrangement (chemical cross-links) of the polymer network also significantly affects the degree of swelling. The swelling of rubber in organic solvents is a diffusion process, where the solvent diffuses into the bulk of the rubber, increasing the dimensions of the rubber network until the concentration of the liquid is uniform and equilibrium swelling is achieved (Crank, 1956; Blow, 1971). The theory predicts that, in the early stages of swelling, the mass of liquid absorbed per unit area of rubber is linearly proportional to the square root of time up to fifty percent of the equilibrium swelling. This relationship also applies to the materials included in this study, although the linear region may not be as high as fifty percent of the equilibrium swelling as shown in Figure 4.14. Based on this knowledge, Southern (1967) was able to calculate the penetration depth of a swelling liquid into rubber and demonstrate that thinner or smaller particles of rubber swell more rapidly than thicker or bigger particles.

At the start of the diffusion process, the surface of the rubber has a high liquid concentration while the liquid concentration in the bulk of the rubber is zero. The liquid molecules then diffuse into the rubber just below the surface and eventually into the bulk of the rubber. As the diffusion process proceeds, the dimensions of the rubber particle increase until the concentration of liquid is uniform throughout the particle and equilibrium swelling is achieved (Blow, 1971). The amount of a given solvent that will diffuse into the rubber until it reaches equilibrium depends upon the number of cross-links per unit volume of rubber. The greater the number of cross-links per unit volume, the shorter the average length of rubber chains between cross-links and the lower the degree of swelling. In addition, the degree of swelling depends upon the compatibility of the rubber and solvent on a molecular scale (lower molecular weight solvents diffuse quicker into rubber), and the amount and type of filler present in the rubber. Southern (1967) found that during the early stages of the diffusion reaction, the rate of penetration (diffusion) is controlled by the viscosity of the swelling liquid (lower penetration rates at higher viscosity), but that the total amount of liquid absorption depends on the chemical nature of the rubber.

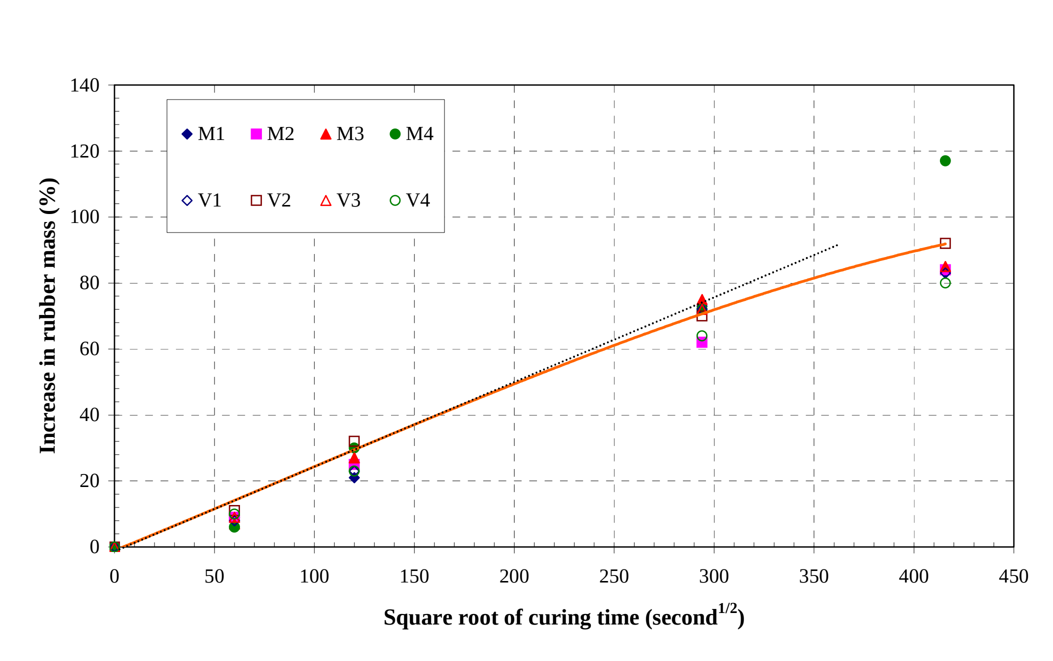


Figure 4.14: Bitumen absorption versus the square root of curing time for all bitumens at equiviscous temperature and rubber bitumen ratio of 1:6

The reaction that occurs between crumb rubber and bitumen in the dry process differs slightly from the standard type of diffusion described above in that the chemical composition and viscosity of the solvent (bitumen) changes as the reaction progresses. This means that in addition to the diffusion reaction slowing down as the rubber becomes saturated, the rate of swelling will also decrease as the viscosity of the binder increases and as the percentage of maltenes decreases. Therefore, the interaction process will be slowed down at a given temperature through the inability of the bitumen to penetrate into the rubber particle.

4.6 SUMMARY

The interaction between the single source of crumb rubber and the eight penetration grade bitumens, in terms of the increase in rubber mass and bitumen absorption were studied using a basket drainage bitumen absorption method. The interaction occurred through the absorption of the light molecular weight fractions of the binders and was found to be independent of the rubber-bitumen ratio provided that there is sufficient bitumen (light fractions) to allow absorption to occur. In addition, tests carried out on selected bitumen types and grades have confirmed a number of observations found in a previous research (Singleton, 2000) that crumb rubber will swell in bitumen at high temperatures and that the maximum amount of swelling and the rate of swelling will be increased with increasing temperature.

The maximum increase in rubber mass (bitumen absorption) after 48 hours at 160°C appears to be independent of crude source and only marginally related to the penetration grade of the bitumen with the softer, high penetration grade bitumens generally resulting in a greater amount of absorption compared to the harder, low penetration grade bitumens. The initial rate of bitumen absorption is directly related to the viscosity (penetration grade) as well as the chemical composition (crude source) of the binders with the softer (less viscous) and lower asphaltene content binders having the highest rates of absorption. Comparing between two tests conditions, constant temperature and viscosity, significantly higher increases in bitumen absorption were observed for M1 and V1 bitumen due to an increase in testing temperature. Tests conducted at very high temperatures for M4 bitumen increased bitumen absorption as rubber particles started to degrade. Therefore, depending on the type of bitumen used, it is important to find the right mixing temperature to control the rate and amount of swelling. Overall, the results demonstrated that significant rubber-bitumen interaction takes place at high temperatures and consequently the rheological and chemical properties of residual bitumen will be affected. To investigate this, chemical and rheological tests on the residual bitumen were carried out and described in the next chapter.

Table 4.4:The increase in rubber mass of Middle East Bitumens tested at constant and equiviscous temperatures using 1:6 rubber bitumen ratio

Reaction time (hour)	**% Increase in rubber mass (R: B=1:6)**											
	M1			**M2**			**M3**			**M4**		
	160^0C	140^0C	Swelling difference	160^0C	150^0C	Swelling difference	160^0C	170^0C	Swelling difference	160^0C	180^0C	Swelling difference
1	20	9	**-11**	18	9	**-9**	4	7	**+3**	4	6	**+2**
4	108	21	**-87**	52	25	**-27**	22	27	**+5**	14	30	**+16**
24	118	73	**-45**	100	62	**-38**	72	75	**+3**	94	72	**+22**
48	130	83	**-47**	104	84	**-20**	94	85	**+9**	140	117	**+23**

Table 4.5 The increase in rubber mass of Venezuelan Bitumens tested at constant and equiviscous temperatures using 1:6 rubber bitumen ratio

Reaction time (hour)	**% Increase in rubber mass (R: B=1:6)**											
	V1			**V2**			**V3**			**V4**		
	160^0C	135^0C	Swelling difference	160^0C	150^0C	% Swelling difference	160^0C	165^0C	Swelling difference	160^0C	165^0C	Swelling difference
1	18	8	**-9**	16	11	**-5**	*32*	9	**-23**	4	10	**+6**
4	48	23	**-25**	36	32	**-4**	*52*	27	**-25**	14	23	**+9**
24	94	72	**-22**	84	70	**-14**	*102*	72	**-30**	62	64	**+2**
48	140	83	**-57**	116	92	**-24**	*138*	85	**-53**	86	80	**-6**

CHAPTER 5

Chemical and Mechanical Testing of Residual Bitumen

5.1 INTRODUCTION

Swelling test results from the previous chapter demonstrated that at high temperatures rubber absorbs a percentage of bitumen with the rate of swelling mainly depending on the complex chemical nature and viscosity of the bitumen but also marginally dependent on the penetration grade and crude source. Singleton (2000) indicated that rubber absorbs lighter fractions resulting in a change of the colloidal structure of the residual bitumen. If this were the case, the properties of the bitumen and the performance of the CRM asphalt mixture would also be affected through the interaction due to the hardening of bitumen. Earlier research on bitumen chemistry described the complex chemistry of bitumen as a combination of molecular and intermolecular (microstructure) structures. The molecular structure of bitumen is a mixture of organic molecules with varying molecular weights from several hundred to several thousand that exhibit certain behavioural characteristics (Robertson *et al*, 1991). According to SHRP, the microstructure is a continuous association of polar molecules (asphaltenes) dispersed in non-polar, or relatively low-polarity, molecules (maltenes) which form dipolar intermolecular bonds (Little *et al*, 1994) with the general behaviour of bitumen being ruled by the behavioural characteristics at the intermolecular level. The strength of these intermolecular bonds is weaker than the bonds holding the basic organic hydrocarbon constituents. Therefore, in a specific condition, these structures break first and consequently bitumen's physical characteristics are a direct result of the forming, breaking and reforming of these intermolecular bonds (Little *et al*, 1994). In dry process CRM asphalt mixtures, bitumen hardening is mainly due to the contribution of oxidation and the rubber-bitumen interaction. The physical, chemical and mechanical properties of bitumen are, therefore, needed to give an indication of how these would affect the performance of dry process CRM asphalt mixtures.

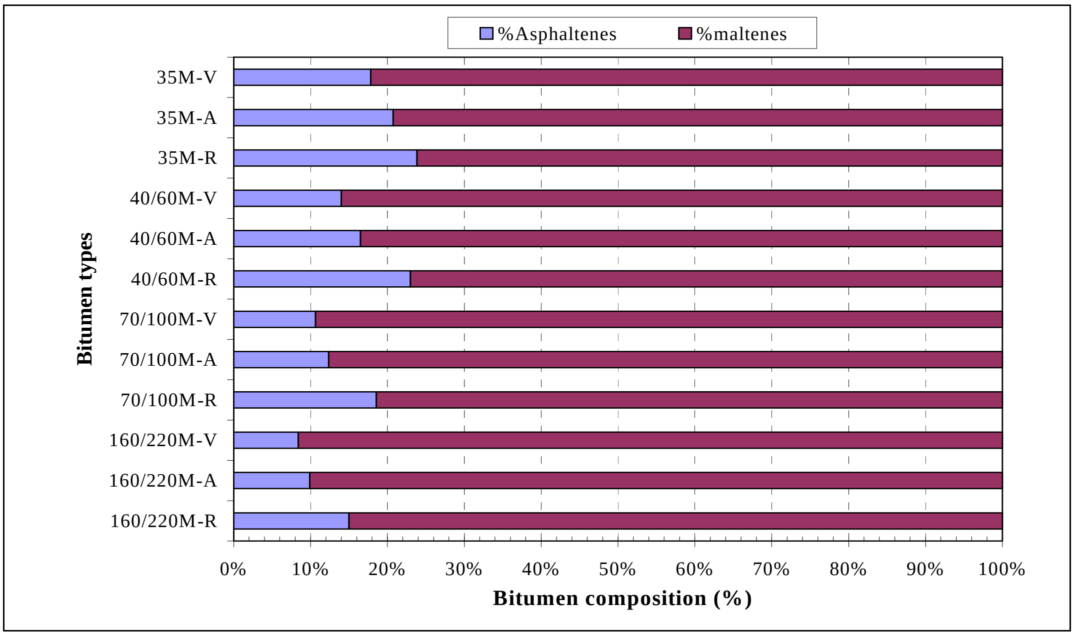
The aim of this chapter is to present the effect of rubber-bitumen interaction on the residual bitumen characteristics. The physical, chemical and rheological properties of neat bitumen were compared to identical bitumen having undergone rubber-bitumen basket drainage tests. This was carried out on all eight penetration grade bitumens used in this investigation. The bitumens were classified as; virgin (i.e unaged), aged (oven cured at the same temperature as the basket drainage tests, but without rubber), and residual (collected after rubber-bitumen absorption test). In the first section of this chapter, the chemical properties of the virgin, residual and aged bitumen are presented. In the next section the rheological properties, in terms of high temperature viscosities, dynamic viscoelastic properties and penetration values are discussed. The final sections present a brief discussion of the results concluding with a summary.

CHEMICAL PROPERTIES

The chemical testing was conducted according to BS 2000: Part 143:1996 to determine asphaltenes content in crude petroleum and petroleum products. In this method the bitumen is mixed with heptane and the mixture is heated. The precipitated asphaltenes, waxy substances and inorganic material are then collected on a filter paper. The waxy and inorganic materials are removed by using hot heptane and hot toluene respectively and the pure asphaltenes are collected and weighted. The results are expressed in terms of percentage asphaltenes content in the bitumen samples.

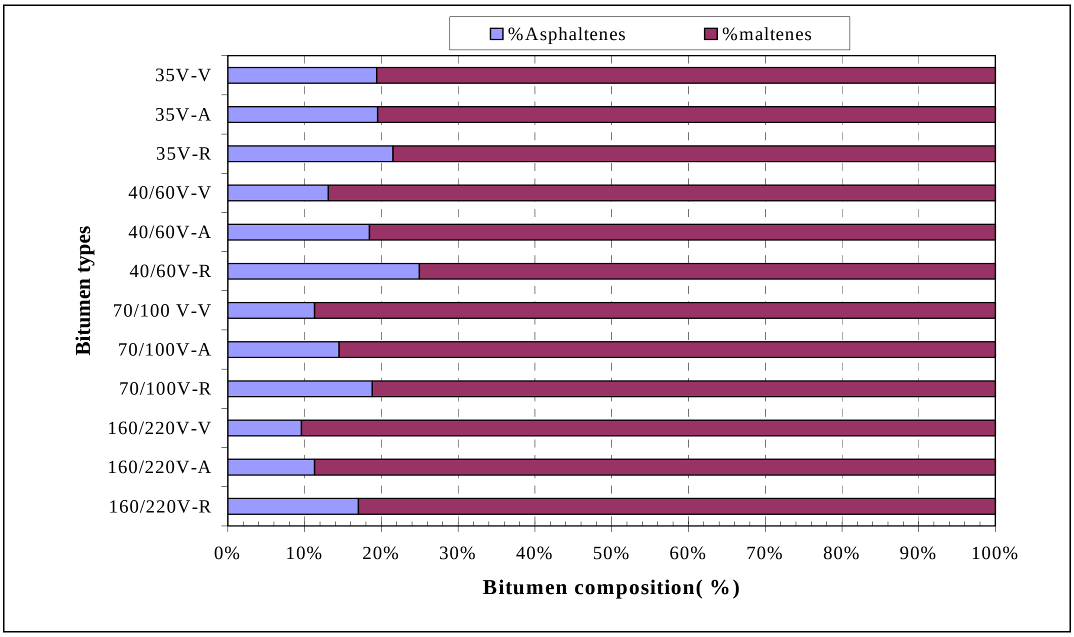
The relative compositions of asphaltenes and maltenes for the unaged, aged and residual binders from the swelling tests conducted at equiviscous temperatures (see Table 4.2) are shown in Figures 5.1 and 5.2. The results are plotted in terms of percentage of asphaltenes and maltenes content before and after the swelling test. It can be seen that in the virgin state the four Middle East binders have slightly lower asphaltenes and therefore higher maltenes compared to the Venezuelan bitumens. The asphaltenes contents of the bitumens as well as their ageing and residual indices (i.e. asphaltenes content for aged or residual binder divided by asphaltene content for the unaged (virgin) bitumen) are presented in Tables 5.1 and 5.2. Table 5.1 shows the results of bitumens subjected to constant viscosity swelling and Table 5.2 concerns the bitumens subjected to constant temperature swelling tests. It can be seen that the relative change in asphaltenes content is not markedly different for the two testing conditions.

Both the Middle East and the Venezuelan binders show a considerable reduction in maltenes and relative increase in asphaltenes after crumb rubber-bitumen interaction compared to the decrease in maltenes and increase in asphaltenes seen after standard high temperature oxidation. This could be attributed to the dipolar molecular bonds breaking and rearranging as explained in the introduction of this chapter, resulting in an increase in the asphaltenes fraction as the lighter fraction is being absorbed by rubber particles.



Note: V: virgin, A: aged, R: residual

Figure 5.1: Chemical composition of unaged, 48hours aged and residual Middle East bitumens tested at equiviscous (0.2 Pa.s) temperature



Note: V: virgin, A: aged, R: residual

Figure 5.2: Chemical composition of unaged, 48 hours aged and residual Venezuelan bitumens tested at equiviscous (0.2 Pa.s) temperature

Table 5.1: Asphaltene content of virgin, aged and residual bitumen subjected to swelling test at equiviscous temperatures for 48 hours

Binder	Asphaltenes			Aged / Unaged	Residual / Unaged
	Unaged (%)	Aged (%)	Residual (%)		
M1	8	10	15	1.18	1.77
M2	11	12	19	1.28	1.66
M3	14	17	23	1.41	1.90
M4	18	21	24	1.01	1.11
V1	10	11	17	1.18	1.79
V2	11	15	19	1.16	1.74
V3	13	18	25	1.18	1.64
V4	19	20	22	1.16	1.34

Table 5.2: Asphaltenes content of virgin, aged and residual bitumen subjected to swelling tests at 160^{0}C for 48 hours

Binder	Asphaltenes			Aged / Unaged	Residual / Unaged
	Unaged (%)	Aged (%)	Residual (%)		
M1	8	12	19	1.48	2.24
M2	11	12	19	1.11	1.80
M3	14	16	22	1.16	1.56
M4	18	20	24	1.11	1.33
V1	10	13	18	1.40	1.88
V2	11	12	19	1.06	1.72
V3	13	14	23	1.04	1.77
V4	19	20	22	1.04	1.13

RHEOLOGICAL PROPERTIES

High Temperature Viscosity

Results from the swelling tests demonstrated that at the initial stage, the rate of swelling depends on the viscosity of the bitumen but the results are more comparable for all bitumens after 24 hours of curing. Both constant temperature (160^{0}C) and equiviscous temperatures (temperature corresponding to 0.2Pa.s viscosity) swelling test results indicated similar trends although slightly more or less absorption was observed depending on the variation of the test temperature (higher or lower than their corresponding equiviscous temperatures). The high temperature viscosities were measured using a rotational viscometer to investigate the effect of the rubber-bitumen interaction and oxidation on the bitumen and are presented in Figures 5.3 to 5.4.

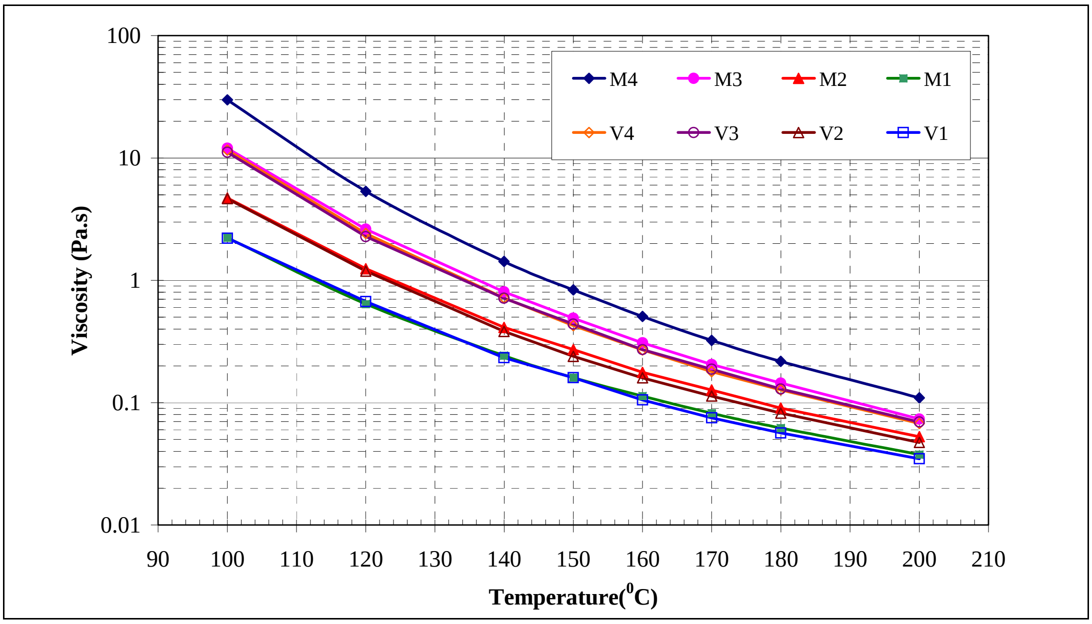


Figure 5.3: High temperature viscosity of Middle East and Venezuelan bitumens aged for 48 hours without rubber

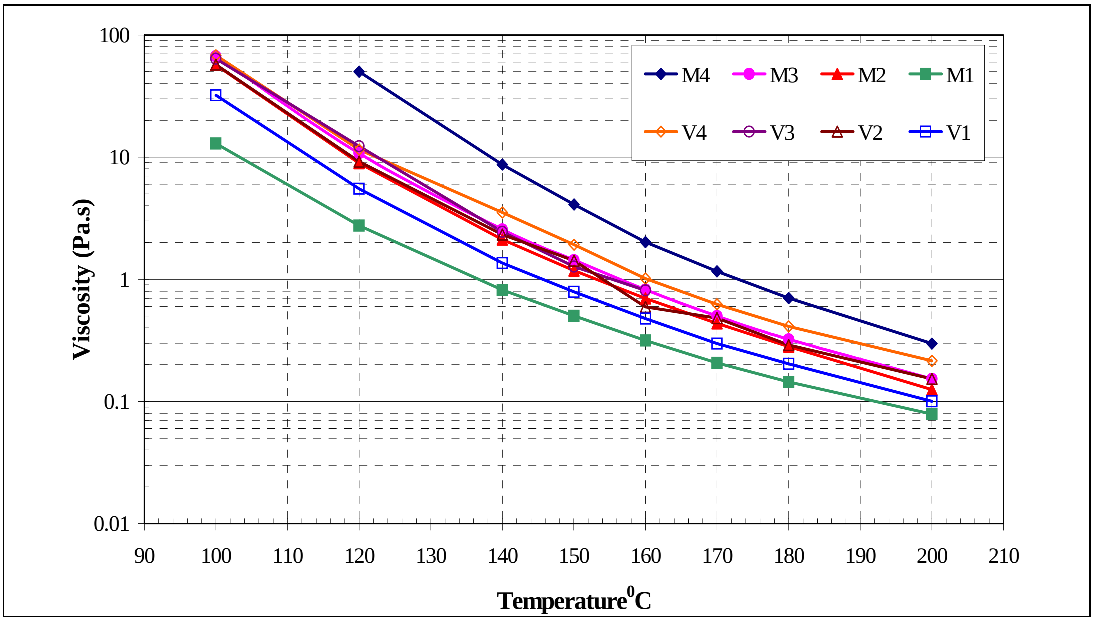


Figure 5.4: High temperature viscosity of Middle East and Venezuelan bitumen residual subjected to 48 hours interaction with rubber

It can be seen from both figures that at any particular temperature, the viscosity of the residual bitumen (Figure 5.4) is much higher than the bitumen aged without rubber (Figure 5.3). All the results were compared with their corresponding unaged viscosities and the results for bitumen M3 (100 to 200°C) in its unaged condition and after curing in air and with crumb rubber at 160^{0}C are shown in Figure 5.5. In addition, rotational viscosities (η) at 100°C and 160°C for all the binders in their unaged, aged and residual conditions are also presented in Tables 5.3 and 5.4 with ageing and residual indices (η for aged or residual binder divided by η for the unaged (virgin) bitumen).

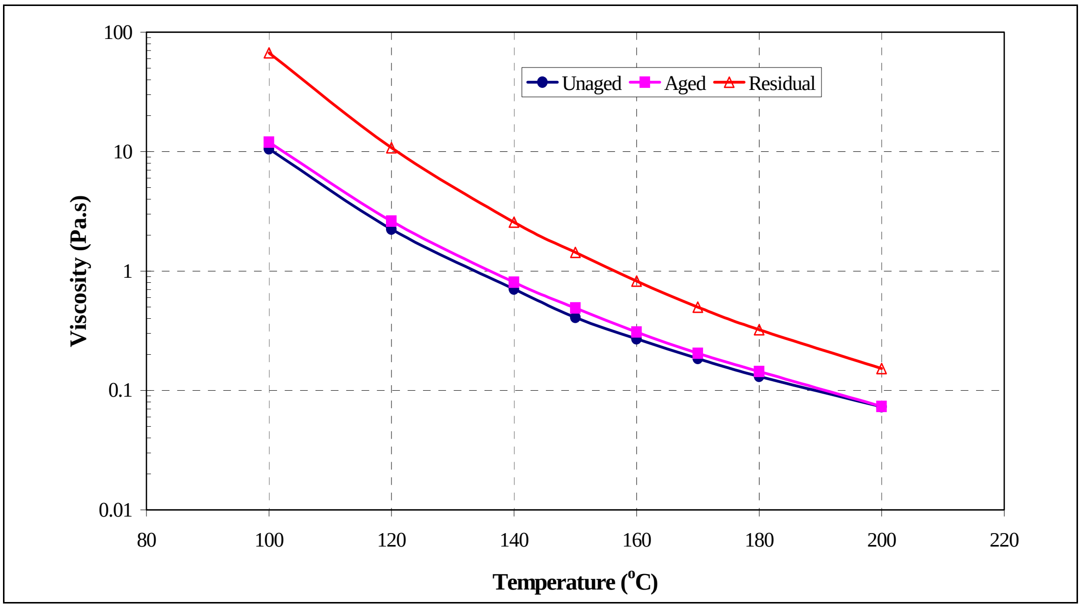


Figure 5.5: High temperature viscosities in unaged (virgin), aged and residual state for M3 bitumen

The results show a consistent increase in viscosity after both oxidative ageing and more significantly after high temperature curing in the presence of crumb rubber. The ageing and residual indices give a clear indication of the increased stiffening effect resulting from the rubber-bitumen interaction compared to that found after standard high temperature oxidative ageing, with the stiffening effect being between 3 and 15 times greater for the residual binders compared to the aged binders. The significant increase in viscosity after the rubber-bitumen interaction can partly be attributed to the additional loss (absorption) of the light oily fractions of the bitumen into the crumb rubber during the high temperature swelling tests. Kandhal and Wenger (1975) reported that an increased relative age hardening of the bitumen could lead to potential ravelling.

***Table 5.3*: Rotational viscosities @ 100^{0}C following 48 hours curing with and without rubber.**

Binder	η_{unaged} (mPa.s)	η_{aged} (mPa.s)	$\eta_{residual}$ (mPa.s)	$\eta_{aged}/\eta_{unaged}$	$\eta_{residual}/\eta_{unaged}$
M1	1,476	2,226	12,906	1.51	8.74
M2	2,972	4,698	56,839	1.58	19.12
M3	10,539	12,003	67,066	1.14	6.36
M4	18,347	29,882	-	1.63	-
V1	1,406	2,208	32,096	1.57	22.83
V2	3,121	4,663	57,200	1.49	18.33
V3	5,869	11,137	63,900	1.90	10.89
V4	7,944	11,602	68,012	1.46	8.56

Table 5.4: Rotational viscosities @ 160^{0}C following 48 hours curing with and without rubber.

Binder	η_{unaged} (mPa.s)	η_{aged} (mPa.s)	$\eta_{residual}$ (mPa.s)	$\eta_{aged}/\eta_{unaged}$	$\eta_{residual}/\eta_{unaged}$
M1	91	113	316	1.24	3.47
M2	138	177	696	1.28	5.04
M3	271	308	822	1.14	3.03
M4	387	507	2,015	1.31	5.21
V1	79	105	477	1.33	6.04
V2	135	160	595	1.19	4.41
V3	210	272	810	1.30	3.86
V4	225	270	1,012	1.20	4.50

Viscoelastic Properties

The intention of this investigation was to measure the changes in the rheological properties of bitumen that has reacted with rubber and compare them with bitumen that has not reacted with rubber. Analysis of the changes of the rheological properties will help to give an indication of what changes occur to the binder in the dry process CRM asphalt mixtures. The bitumen samples were collected after 48 hours swelling and tested in the Dynamic Shear Rheometer (DSR). The working principle of DSR is explained in Chapter 2. The rheological test results for residual bitumen and aged bitumen were compared with those obtained for the virgin (unaged) bitumens to identify changes in the rheological properties following rubber-bitumen interaction and standard oxidative ageing.

Before conducting DSR tests at different temperatures and frequencies, the linear viscoelastic region was determined following the guidelines presented in Section 2.1.3. The tests were conducted at 1Hz and at 25^{0}C and 60^{0}C for all unaged, aged and residual bitumens with strain corresponding to 95% of the initial complex modulus being used as a guideline strain limit for the linear region. The testing protocol including the target strain within the LVE region of the different bitumens, is presented in Table 5.5. The rheological data was transposed between the different frequencies and temperatures using the time-temperature principle of superposition (TTSP) to produce master curves of G* and δ at a reference temperature of 35^{0}C to allow the rheological properties of the virgin, aged and residual binders to be compared.

Table 5.5: Rheology testing protocol

Bitumen	**Frequency (Hz)**	**Bitumen state**	**Test temperature (^{0}C)**	**Geometry (spindle/gap) (mm)**	**Strain level**
M3,V3 M4,V4	0.1 to 10	virgin	10-25	8 /2	0.5%
			25-75	25 / 1	1.5%
M1,V1 M2,V2	0.1 to 10	virgin	10-25	8 /2	0.5%
			25-75	25 / 1	3%
M3,V3 M4,V4	0.1 to 10	aged, residual	10-25	8 /2	0.1%
		aged, residual	25-75	25 / 1	1.5%
M1,V1 M2,V2	0.1 to 10	aged, residual	10-25	8 /2	0.1%
		aged, residual	25-75	25 / 1	1.5%

Master curves of complex modulus and phase angle for the virgin (unaged), aged and residual bitumen M3 at a reference temperature of 35^{0}C are shown in Figures 5.6 and 5.7. Similar plots were also produced for the other seven binders and are presented in the Appendix C. The results clearly demonstrate the increased stiffness (complex modulus) and elastic response (decreased phase angle) of all the aged and particularly the residual binders compared to their virgin state as a result of rubber-bitumen interaction and oxidation at high temperatures for 48 hours.

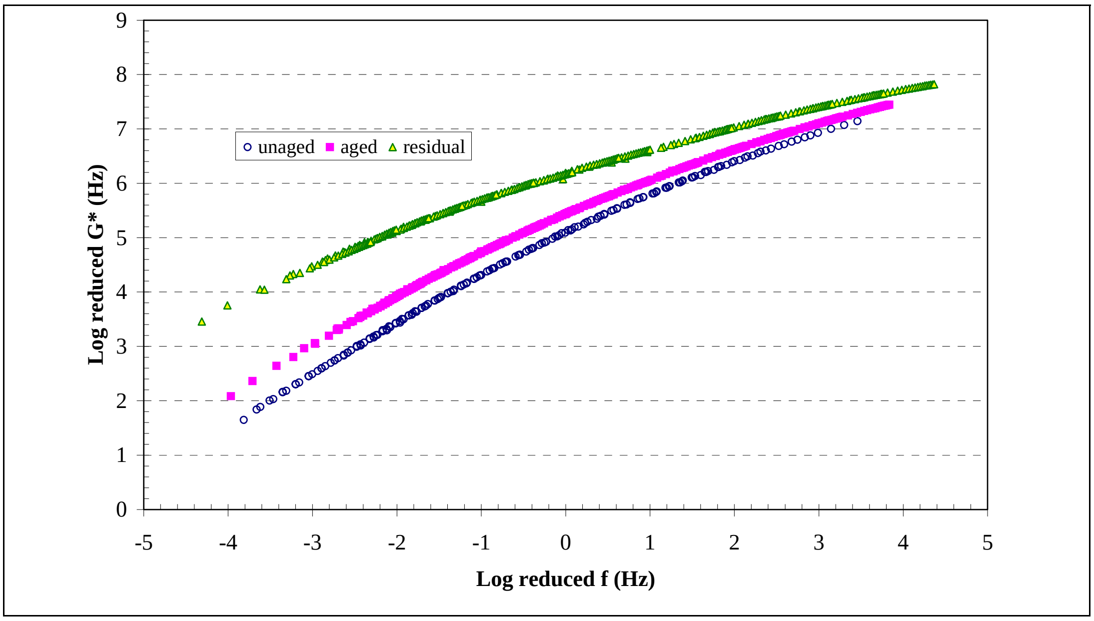


Figure 5.6: Master curves of complex modulus for bitumen M3 at a reference temperature of 35^{0}C for unaged (virgin), 48 hours aged and residual binder tested for 48 hours at a temperature 160^{0}C

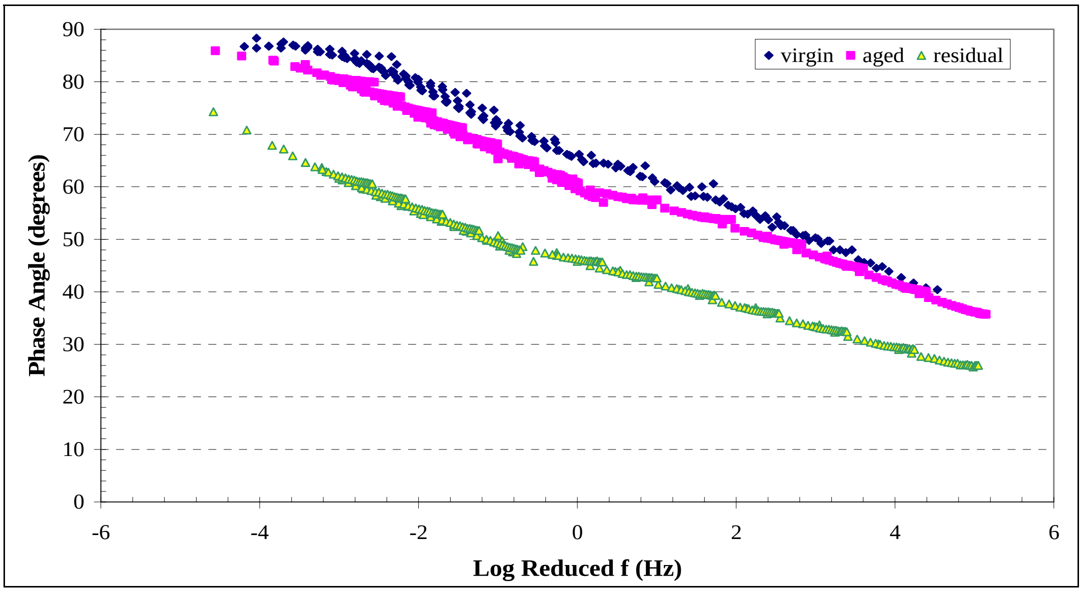


Figure 5.7: Master curves of phase angle for bitumen M3 at a reference temperature of 35^{0}C for unaged (virgin), 48 hours aged and residual binder tested for 48 hours at a temperature 160^{0}C

To compare the relative rheological properties of bitumen obtained from both constant temperature (160^{0}C) and constant viscosity (0.2Pa.s) swelling tests, the complex modulus (G*) and phase angle (δ) at 1 Hz and at temperatures of 25°C and 60°C are presented in Tables 5.6 to 5.8. The relative increase in G* after standard high temperature ageing and after curing in rubber have been included in Tables 5.6 and 5.7 as curing indices (G* aged or G* residual divided by G* unaged).

The aged indices indicate that all eight bitumens undergo a similar increase in stiffness after oxidative ageing irrespective of crude source, penetration grade and testing condition. However, the effect of curing with rubber for 48 hours at 160°C or equiviscous temperature is not as definitive, as the residual indices tend to vary amongst the penetration grades and between the crude sources as well as between the two test temperatures. Overall the residual indices are considerably greater than the aged indices due to the greater loss of the light fractions of the binders following rubber-bitumen interaction.

Table 5.6: Changes in complex modulus at 1 Hz and 25°C following 48 hours curing with and without rubber at 160^{0}C and at 0.02Pa.s viscosity

Binder	G^*_{unaged}	G^*_{aged}		$G^*_{residual}$		G^*_{aged}/G^*_{unaged}		$G^*_{residual}/G^*_{unaged}$	
		T=160^{0}C	η=0.02Pa.s	T=160^{0}C	η=0.02Pa.s	T=160^{0}C	η=0.02Pa.s	T=160^{0}C	η=0.02Pa.s
M1	1.05E+5	1.56E+5	1.55E+05	1.75E+6	6.03E+05	1.49	1.48	16.7	5.74
M2	3.10E+5	6.25E+5	7.85E+05	2.84E+6	4.78E+06	2.02	2.53	9.2	15.42
M3	5.91E+5	1.14E+6	1.34E+06	2.42E+7	6.88E+06	1.94	2.27	41.0	11.64
M4	1.54E+6	2.48E+6	2.13E+06	-	7.97E+06	1.61	1.38	-	5.18
V1	8.17E+4	3.31E+5	1.60E+05	1.92E+6	7.48E+05	4.05	1.96	23.5	9.16
V2	3.64E+5	6.35E+5	4.55E+05	2.69E+6	3.51E+06	1.75	1.25	7.4	9.64
V3	5.59E+5	1.58E+6	1.66E+06	3.31E+6	9.14E+06	2.82	2.97	5.9	16.35
V4	2.02E+6	3.20E+6	2.65E+06	-	1.38E+07	1.58	1.31	-	6.83

Table 5.7: Changes in complex modulus at 1 Hz and 60°C following 48 hours curing with and without rubber at 160^{0}C and at 0.02Pa.s viscosity

Binder	G^*_{unaged}	G^*_{aged}		$G^*_{residual}$		G^*_{aged}/G^*_{unaged}		$G^*_{residual}/G^*_{unaged}$	
		T=160^{0}C	η=0.02Pa.s	T=160^{0}C	η=0.02Pa.s	T=160^{0}C	η=0.02Pa.s	T=160^{0}C	η=0.02Pa.s
M1	5.40E+2	6.53E+2	5.85E+02	1.11E+4	3.43E+03	1.21	1.08	20.6	6.35
M2	9.54E+2	2.24E+3	1.61E+03	5.07E+4	2.24E+04	2.35	1.69	53.1	23.48
M3	3.17E+3	6.13E+3	7.01E+03	8.55E+4	9.81E+04	1.93	2.21	27.0	30.95
M4	8.98E+3	2.88E+4	1.78E+04	-	2.12E+05	3.21	1.98	-	23.61
V1	5.01E+2	1.65E+3	6.73E+02	2.20E+4	2.98E+03	3.30	1.34	44.0	5.95
V2	1.25E+3	2.33E+3	1.71E+03	3.22E+4	1.64E+04	1.87	1.37	25.8	3.12
V3	3.09E+3	1.15E+4	7.34E+03	6.58E+4	8.54E+04	3.73	2.38	21.3	27.64
V4	7.30E+3	1.57E+4	7.93E+03	9.79E+4	1.06E+05	2.15	1.09	13.4	14.2

The phase angles presented in Table 5.8 show that irrespective of bitumen crude source and grade, the rubber-bitumen interaction causes a reduction of phase angle indicating more elastic response under loading. The type of swelling test also shows a significant influence on the elastic response of both aged and residual bitumens. For example, the phase angle for residual bitumen M1 at 160^{0}C shows a decrease from 77 to 50 at 25^{0}C, whereas the same bitumen cured at 140^{0}C changed from 77 to 61. The difference could be attributed to the larger proportion of maltenes fraction absorbed with increasing test temperature.

Table 5.8: Changes in phase angle (degrees) at 1 Hz and two temperatures following 48 hours curing with and without rubber

Binder	δ_{unaged} @25^{0}C	δ_{aged} @ 25^{0}C		$\delta_{residual}$ @ 25^{0}C		δ_{unaged} @60^{0}C	δ_{aged} @ 60^{0}C		$\delta_{residual}$ @ 60^{0}C	
		160^{0}C	0.02 Pa.s	160^{0}C	0.02 Pa.s		160^{0}C	0.02 Pa.s	160^{0}C	0.02 Pa.s
M1	77	59	75	50	61	87	88	88	73	83
M2	69	63	66	26	47	87	63	82	60	71
M3	58	54	53	30	39	82	77	76	58	56
M4	51	33	48	-	34	75	70	71	51	50
V1	78	65	75	32	63	89	85	88	68	83
V2	71	67	70	39	54	85	86	85	68	75
V3	66	46	59	27	39	83	75	78	60	58
V4	55	46	63	21	41	84	72	82	62	61

Calculated Penetration

As the amount of residual bitumen collected after the basket drainage bitumen absorption test was not sufficient to conduct further viscosity tests, the Gershkoff formula, presented in Equation 2.1, was adopted to calculate penetration from the DSR results. In this empirical formula, the complex modulus of a particular bitumen tested at 25^0C and 0.4Hz was directly read off from the isothermal plot and used in Equation 2.1. However, to check the accuracy of the formula, the results from measured and calculated penetration for all the virgin bitumens are also presented in Table 5.9. It can be seen that there are discrepancies between the two methods especially with the very soft bitumens. Therefore, the penetration results were used only in a comparable capacity for the aged and residual bitumens.

Table 5.9: Comparison of measured and calculated penetration

Binder Type	Bitumen source	Penetration grade	Penetration (dmm)	
			Measured	Calculated
M1	Middle East	160/220pen	192	126
M2		70/100pen	88	70
M3		40/60pen	53	48
M4		35pen	32	27
V1	Venezuelan	160/220pen	203	160
V2		70/100pen	90	75
V3		40/60pen	55	56
V4		35pen	36	31

Table 5.10 presents the calculated penetration results for all the virgin, aged and residual bitumens as well as the penetration indices in terms of percentage retained penetration (penetration for aged or residual binder divided by penetration for the unaged bitumen). The results show that compared to aged bitumen, irrespective of crude source and grade, the penetration of the residual bitumen is significantly reduced after rubber-bitumen reaction. In terms of retained penetration, which can be considered as an indication of the remaining life of the bitumen, there is almost a double reduction of penetration due to rubber-bitumen interaction as compared to normal high temperature ageing. In addition, the reduction of penetration below 20dmm raises durability issues as previous research has found that when the

penetration of the bitumen falls below 20, serious pavement cracking might occur (Hubbard and Gollomb, 1937).

Table 5.10: Calculated penetration using Gershkoff formula on bitumen tested at equiviscous temperatures with rubber-bitumen ratio 1:6

Binder	**Stiffness at 25^{0}C and 0.4Hz (Pa)**			**Calculated Pen (mm/10)**			**%Retained Pen**	
	Unaged	**A**ged	**R**esidual	**U**naged	**A**ged	**R**esidual	**A/U**	**R/U**
M1	4.96E+04	6.80E+04	2.90E+05	126	107	50	85%	40%
M2	1.55E+05	3.51E+04	2.52E+06	70	45	16	64%	23%
M3	3.10E+05	6.80E+05	3.82E+06	48	32	13	67%	27%
M4	9.52E+05	1.11E+06	4.60E+06	27	25	12	93%	44%
V1	3.14E+04	7.07E+04	3.53E+05	160	105	45	66%	28%
V2	1.35E+05	2.07E+05	1.76E+06	75	60	20	80%	27%
V3	2.35E+05	8.10E+05	5.11E+06	56	29	11	52%	20%
V4	7.28E+05	1.25E+06	7.58E+06	31	23	9	74%	29%

DISCUSSION

Within the dry process, the reaction between crumb rubber and bitumen is usually reduced by limiting the time at which the two components are maintained at the high mixing and transportation temperatures used to produce the asphalt mixture. The laboratory investigations on different bitumens have determined the absorption of bitumen solvent by crumb rubber when held at high temperatures. As explained in the previous chapter, the interaction of rubber-bitumen is a diffusion process. However, there will always be a finite time when the diffusion reaction takes place to reach equilibrium swelling and the effect of this interaction can be reduced either by using a "hard" bitumen (high viscosity) to reduce the rate of absorption or by using a "soft" bitumen to compensate for the large increases in stiffness and viscosity of the residual binder following the rubber-bitumen interaction.

In addition, instead of using 160^{0}C during mixing, the absorption could be reduced by using a lower equiviscous temperature for softer bitumens resulting in a less stiff and less elastic residual bitumen. On the other hand, for harder bitumen (M4), the high equiviscous temperature is not feasible as rubber molecules start to degrade when the temperature reaches 180^0C. The compositional change of different bitumens indicated that certain fractions of the bitumen are readily absorbed by rubber at high temperature.

In the curing process at high temperatures, the hardening of bitumen accelerates as the reaction with oxygen increases with highly active bitumen components forming micelles due to the molecular attraction forces from high molecular weight bitumen components, and consequently the resultant bitumen become stiffer and harder (Whiteoak, 1991). The hardening can also occur due to the evaporation of the more volatile components, such as aromatics and saturates fractions in the bitumen, from the weaker intermolecular superstructure by breaking under temperature (Little *et al*, 1994). However, in this study the effect of oxidation in terms of the changes in complex modulus and phase angle was not significant. In contrast, high temperature curing with rubber increased the complex modulus and decreased phase angle throughout the viscoelastic region as shown in Figures 5.6 and 5.7. These large increases in the viscosity, stiffness (complex modulus) and elastic behaviour of the residual binders will significantly affect the brittle response of the binders and their ability to resist cracking and fretting, while high percentages of absorption will alter the physical shape and rigidity of the crumb rubber particles.

Finally, the relative increase in asphaltenes for the residual binders compared to the aged binders (Tables 5.1 and 5.2) is not as marked as that seen in Tables 5.3 and 5.4 in terms of high temperature viscosity. It is, therefore, not simply the relative proportions of asphaltenes and maltenes in the aged and residual binders but rather the chemical nature of these two fractions that dictate the relative rheological properties of the binders.

5.5 SUMMARY

The consistency, chemical composition and bitumen rheology tests reported in this chapter have been performed in order to study the chemical and physical interaction between crumb rubber particles and bitumen during dry process CRM asphalt production (mixing, laying and compaction). Fundamental issues such as the diffusion of the lighter fractions of the bitumen into the rubber, the effect on the chemical and physical properties and changes in the rheological properties of the bitumen have been investigated and compared with corresponding unaged and aged bitumen. The following summary can be drawn from the results obtained from the physical, chemical and rheological testing of the residual bitumen:

- The chemical testing on the residual bitumen indicated that in addition to traditional oxidation of bitumen at high temperatures, the residual bitumens experienced further changes in their chemical constitution as a result of the crumb rubber-bitumen interaction and the absorption, by the rubber, of the lighter, more volatile fractions of the bitumen.

- Large increase in viscosity of the residual bitumen following rubber-bitumen interaction leads to a significant ageing of the bitumen and would therefore, adversely affect the serviceability of the mixture. In addition, the physical properties of bitumen were changed due to rubber bitumen interaction. In general penetration values reduced by more than 50% after 48 hours swelling.

- The increase in viscosity, stiffness and elastic response can be considered similar irrespective of crude source or penetration grade, although the absolute values for these parameters will be lower for the “softer” bitumens. In addition, the right production temperature is important as increases in temperature lead to additional maltenes absorption resulting in stiffer and more elastic residual bitumens.

The use of low penetration grade bitumen in a dry process CRM asphalt mixture will reduce the rate and possibly the amount of swelling of the crumb rubber particles. However, any changes in the rheological properties of the binder following rubber-bitumen interaction could result in the binder becoming embrittled with loss of flexibility and the ability to resist cracking and fretting. In contrast, the use of a high penetration grade bitumen will increase both the rate and possibly the amount of rubber swelling and, therefore, the shape and rigidity of the rubber. However, the binder should still have sufficient flexibility following the rubber-bitumen interaction to resist cracking and fretting. It should be noted that rubber-bitumen ratios used in this investigation are far higher than the actual proportion used in dry process asphalt mixtures. Therefore, the mechanical interactions of rubber-bitumen composite samples were investigated where rubber to bitumen ratios were selected to simulate actual material proportions used in dry process CRM asphalt mixtures. In the next chapter an investigation on the dynamic mechanical properties of rubber-bitumen composite mixtures using a triaxial test set up are presented.

CHAPTER 6

Idealised Rubber-Bitumen Composite Mixtures

6.1 INTRODUCTION

Dry Process CRM asphalt mixtures are composed of aggregate, large sized crumb rubber particles, filler and bitumen in various proportions. The mixture configuration is slightly different from traditional asphalt mixtures as the incorporation of crumb rubber particles allows the mixture to accommodate large strain. Literature review revealed that field performance of the dry process CRM material is not consistent where premature cracking, fretting are observed (Epps, 1994; Amirkhanian, 2001). Various factors probably contribute to this variable performance of the mixture, such as the rubber-bitumen interaction during production, rubber-bitumen adhesion, variability of crumb rubber and construction practices.

The swelling test results reported in Chapters four and five confirmed that at high temperatures, rubber absorbs the lighter fractions of bitumen through a diffusion process and as a result residual bitumen becomes more elastic and stiffer. These changed residual properties of the bitumen may well affect the performance of the mixture. In addition, it is important to understand the effect of the interaction on the rubber component. Singleton (2000) and Bouchet (1999) performed limited investigations on the performance of individual crumb rubber particles after rubber-bitumen interaction. The rubber specimens were collected from the walls of truck tyres using a coring drill to get the thickest possible cylindrical specimen and tested using an in-house purpose-built test rig to measure dynamic uni-axial stiffness and phase angle under compressive loading. The specimen diameter and thickness were 20mm and 5-6mm respectively. The experiments were conducted on rubber soaked in bitumen at 180^0C for 1 hour and 24 hours and tested in a strain-controlled mode at 20^0C using 2 different frequencies, 2 and 10Hz. Based on the results obtained from the uni-axial testing, it was concluded that the rubber following rubber-bitumen interaction is less stiff and more elastic than the virgin rubber. Similar findings were observed by Flory *et al* (1943), who concluded that diffusion of solvents would soften the rubber but increase its elasticity. In addition, Singleton (2000) and Bouchet (1999) found that the influence of loading frequency is less significant on the mechanical properties compared to the effect of ageing in bitumen for longer periods.

However, there were a number of limitations to the above research. The testing on rubber alone does not reflect the actual material behaviour in the whole mixture matrix and also the mixture proportion of rubber-bitumen did not reflect actual material proportions in the CRM asphalt mixture. In addition, the specimen geometry was not realistically identical to the crumb rubber used in the asphalt mixture (nominal size varies from 2-8mm) and was not ideal for uni-axial compressive testing as the possibility of transverse friction between the loading plates and the rubber would affect the results. Finally, the crumb rubber used in the asphalt mixtures is produced from the whole tyre not only from the apex of the tyre which contains the largest proportion of natural rubber which is known to deteriorate faster at high temperature than synthetic rubber (Blow, 1971).

In the dry process, crumb rubber is used as a partial aggregate substitute and, therefore, carries stresses induced by the traffic. Both rubber and aggregate are "glued" by the bitumen and the performance of the mixture depends on the performance of each component as well as the composite. By considering all the factors, fundamental investigations on the mechanical properties of rubber-bitumen as a composite are essential to understand the effect of the rubber-bitumen interaction. The main objective of this rubber-bitumen composite study is to investigate the change in mechanical properties due to short-term interaction between rubber and bitumen. In addition to this primary objective, the influence of other variables such as bitumen type and grade, stress level, loading frequency and test temperatures are also evaluated.

The first part of this chapter describes the mixture design, compaction, curing and dimensional stability of the rubber-bitumen composite samples produced for later triaxial testing. The later part of the chapter then presents the triaxial testing results on different composite mixtures as a function of different loading times and temperatures.

6.2 SAMPLE PRODUCTION

6.2.1 Materials

Four different types of bitumen and one single source of crumb rubber from recycled truck tyres were used for this rubber-bitumen composite study. The bitumen was chosen to cover low penetration (40/60 pen) to high penetration (160/220 pen) grades with two sources, Middle East and Venezuelan. The following bitumen was used in this study:

- M1 = 160/220pen Middle East,
- V1 = 160/220pen Venezuelan,
- M3 = 40/60pen Middle East, and
- V3 = 40/60pen Venezuelan.

The physical and chemical properties of the bitumens are presented in Table 4.2 in Chapter 4. The crumb rubber gradation is shown in Table 4.1 (Chapter 4). The crumb rubber used in this study was 2-8 mm in size and was sieved to obtain two single size fractions; passing 6.3 mm and retained on 3.35 mm, and passing 3.35 mm and retained on 0.3 mm. As the majority of the granulated crumb rubber was less than

3.35 mm, the two fraction were not placed in equal amounts but consisted of 20% < 6.3 mm and > 3.35 mm and 80% < 3.35 mm and >0.3 mm.

6.2.2 Rubber-Bitumen Mixture Proportions

As there are no standard mixture design guidelines, several trial-and-error techniques were investigated to produce composite samples that simulate realistic proportions of rubber-bitumen in dry process CRM asphalt mixtures. After several trials, it was found that calculating bitumen film thickness from the whole mixture matrix could be used to calculate rubber-bitumen proportions by assuming that:

- Bitumen film thickness is constant throughout the asphalt mixture matrix. In other words, there is enough bitumen in the system to cover all particles to an equal depth.

- Binder film thickness (BFT) for aggregate is equal to BFT of rubber (Figure 6.1a)

- BFT is constant for all sizes of rubber particles (Figure 6.1b)

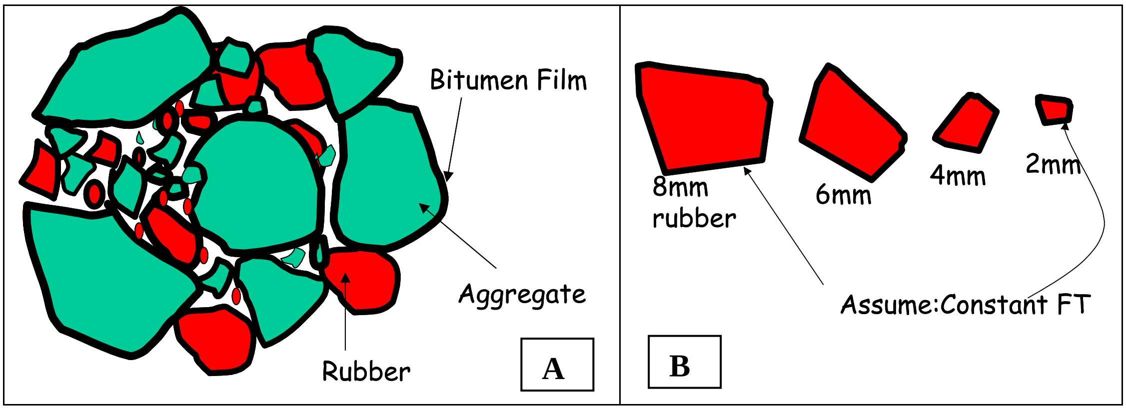


Figure 6.1: Schematic of bitumen film thickness in the mixture and rubber particles

Based on the above-mentioned assumptions, bitumen film thickness was calculated using the Shell formula (Whiteoak, 1991) presented in Equation 6.1. In order to determine the isolated rubber-bitumen proportion compared to the actual mixture, binder film thickness (T) was calculated for the whole asphalt mixture matrix.

$$T = \frac{b}{100 - b} \times \frac{1}{SG_b} \times \frac{1}{SAF} \qquad \textbf{(6.1)}$$

$$SAF = \frac{A}{M} \qquad \textbf{(6.2)}$$

Where,

T	=	bitumen film thickness (μm),
SG_b	=	specific gravity of bitumen,
SAF	=	surface area factor (m^2 / kg). The factor was developed by Hveem, assuming spherical particle of aggregate and specific gravity 2.65 (ASTM, 1992),
A	=	area of the spherical particle (m^2),
M	=	mass of the particle (kg), and
b	=	bitumen content, % mass

Calculation of bitumen film thickness (BFT) using Equation 6.1 is dependent on the percentage of bitumen content in the mixture, surface area of the aggregate and the specific gravity of the bitumen. The bitumen content was chosen as 6.5 % by mass of the mixture as literature, reported in Chapter 3, suggests that bitumen content for CRM mixtures is typically 10-20% more than conventional hot mix asphalt mixtures. The surface area factor was calculated assuming that the aggregate and rubber particles are spherical with specific gravities of 2.65 and 1.1 respectively. The specific gravity of bitumen was supplied by the manufacturer as 1.01. After calculating bitumen film thickness for the whole mixture matrix, a new SAF was calculated for the rubber alone using the specific gradation of rubber presented in Table 4.1. Finally, Equation 6.1 was rearranged to calculate the bitumen content (by mass) for the rubber-bitumen composite mixture. A detailed description of the calculation procedure with an example is outlined in Appendix A-II.

6.2.3 Mould Selection and Specimen Geometry

It was observed that coring from slabs significantly damaged the surface of the specimens due to the difficulty in cutting the rubber component of the mixtures making the specimens unsuitable for testing. To avoid this difficulty, a cylindrical split mould was used to cast the specimens (Figure 6.2) as it was easier to place the mixture into the mould, and after compaction it was easier to remove the specimen without disturbing the actual mixture matrix.

Figure 6.2: Split mould to produce rubber-bitumen composite specimens

A non-evaporating, heat resistant grease was used to lubricate the inner surface of the mould to prevent sticking of the mixture component during compaction. The sample geometry was chosen as 150 mm by 75 mm as the height to diameter ratio for triaxial testing should be at least 2:1 to eliminate the instability and shear plane effect from the boundaries (Punmia, 1996).

6.2.4 Mixture Preparation

The required quantity of bitumen was heated at 160 to 170^{0}C depending on the bitumen type for at least two hours prior to mixing with rubber. A temperature controlled sun-and-planet type mixer (Figure 6.3) was used to mix the rubber and bitumen.

Figure 6.3: Sun-and-planet mixture

Three different batches of rubber were prepared with the appropriate grading and one sample was produced at each time. The required quantity of rubber was placed in the mixer and the preheated bitumen was then added to the rubber and mixed for approximately six minutes at 160^0C. The fully coated mixture was then placed inside a thermostatically controlled oven before compaction. The other two mixtures were produced using the same method and all three mixtures were kept inside a thermostatically oven for 0, 2 and 6 hours to simulate typical durations that the mixture would be held at high temperatures during mixing, transportation and laying prior to compaction. Initially, the curing temperature was chosen at 135^0C to simulate typical short-term ageing for asphalt mixtures (Scholz, 1996), but it was found that the rubber particles stuck together and it was difficult to compact. To minimise this problem, short-term ageing was conducted at 160^0C. However, ageing at 160^0C would increase the rate of reaction especially for the composite with softer bitumen. Therefore, to reduce the effect of excessive ageing, the calculated bitumen film thickness was increased by 20% during the design process. For example, assuming 6.5% bitumen by mass in the asphalt mixtures, the calculated bitumen film thickness using Equation 6.1 is 0.112 mm. Increasing the bitumen film thickness by 20% and rearranging Equation 6.1 and using the new surface area factor for rubber, the calculated proportion of bitumen in the rubber-bitumen composite is 15% by mass and the corresponding rubber proportion is 85% by mass.

6.2.5 Compaction

Specimen manufacture was achieved by compacting the rubber-bitumen composite in a preheated (160^0C) cylindrical split mould. The mixture was placed into the mould in three layers each compacted by tamping using a Kango Hammer. A wooden platen was placed on top of the mixture during the compaction process to avoid bitumen and rubber sticking to the hammer foot and loading head. Several trial compaction methods were undertaken using vibratory and gyratory compaction. None of the methods were found to be suitable for manufacturing samples with the required geometrical configuration set for triaxial testing. Finally, a Denison T60 (Figure 6.4) was used to compact the specimen using a dead load of 15 kN to achieve maximum compaction.

A special loading head (Figure 6.5a) was placed on top of the wooden platen and load was applied directly to that head to ensure that there was no eccentricity during compaction. In addition, the loading head was marked (Figure 6.5b) to ensure that the sample reached a target height of 150 mm at the required compaction effort. The mould was also covered by insulation to minimise heat lose during compaction.

The load was applied in increments of 0.5kN to reach a maximum 15kN compaction load. The specimen was kept for a further 10 minutes under 15kN load to allow the mixture to cool down and bitumen to gain stiffness to reduce rubber rebounding. The process was found inadequate as non-uniform compaction was observed between the top and bottom parts of the specimen and consequently the specimens were unsuitable for instrumentation and testing. After several trials, two-way compaction, similar to Marshall compaction, was found to be adequate with 10 minutes compaction on each side (total compaction duration 20 minutes). The procedure consists of applying 15kN load for the first 10 minutes and then turning the mould up side down to apply 15kN for a further 10 minutes. In both cases, the loading head was positioned accurately to ensure that the sample reached its target height.

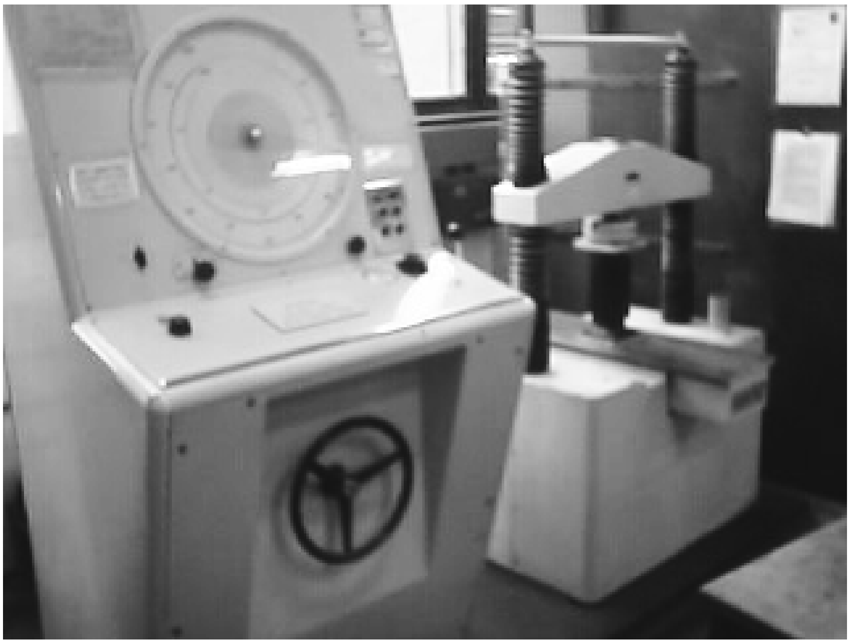


Figure 6.4: Denison T60 machine for dead load compaction

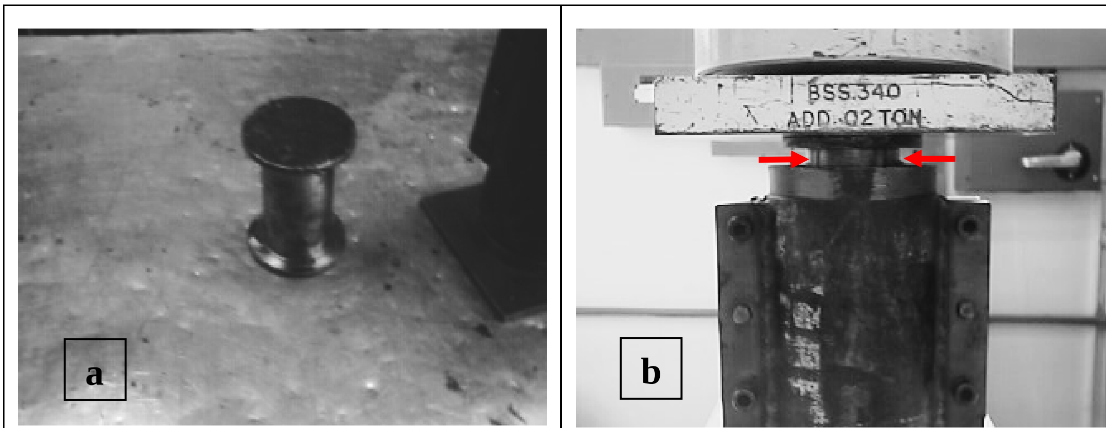


Figure 6.5: (a) Loading head (b) Loading head with specific marked position for target height

This procedure allowed uniform compaction between the top and bottom parts of the specimen and an extra 10 minutes for the bitumen to gain more strength. In addition, immediately after compaction, a dead load of 5kg was applied for a further 24 hours to provide enough time for the sample to reach room temperature. The samples were removed from the split moulds after 24 hours curing and stored at 5^0C for future testing. The compacted samples are presented in Figure 6.6.

Figure 6.6: Composite specimens

6.2.6 Homogeneity of the Composite Samples

In order to ensure homogeneity of the specimens, the density of the mixture was calculated as a theoretical maximum density ($\rho_{\max}$) using Equation 6.3. The bulk density (ρ) of the compacted sample was calculated by measuring the dimensions of the specimens as the primary concern was the voided surface of the specimen and the difficulty in sealing the sample with self-adhesive aluminium foil to perform the conventional method of measuring weight in air and in water. The average of five measurements (height and diameter) was used to calculate the volume of the specimen and a high accuracy balance was used to measure the mass of the compacted specimen. The specific gravity of the crumb rubber was determined using a specific gas jar protocol based on BS1377: part 2:1990:8:2 and was found to be 1.10. The specific gravity of the bitumen was used as 1.01. The percentage volume of voids (V_v) was calculated using Equation 6.4.

$$\rho_{\max} = \frac{M_T}{\frac{M_R}{SG_R} + \frac{M_B}{SG_b}} \qquad \textbf{\textit{(6.3)}}$$

$$V_v = (1 - \frac{\rho}{\rho_{max}}) \times 100 \qquad \textit{(6.4)}$$

Where;

M_T = mass of total rubber plus mass of bitumen

M_R = mass of rubber

M_B = mass of bitumen

SG_R = specific gravity of rubber

SG_B = specific gravity of bitumen

V_v = voids in total mixture

The effect of high temperature curing on the loose composite mixture was investigated in terms of the voids profile using the maximum compaction effort. Fifteen samples were manufactured to study the consistency of the sample production method using the mixture design procedure, compaction and curing techniques outlined in the previous sections. Three samples were produced in each series after 0,1,2,4 and 6 hours curing of the loose mixture at 160^{0}C. Figure 6.7 presents the voids profile of composite samples produced using M3 bitumen against curing period.

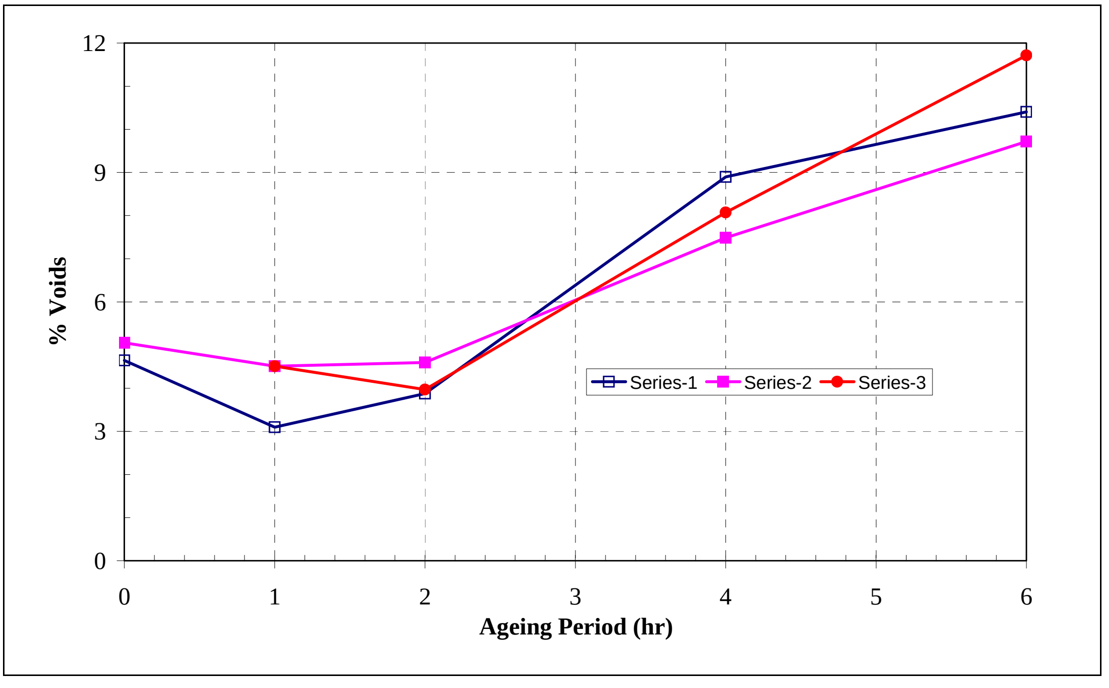


Figure 6.7: Voids profile for composite samples produced using M3 bitumen and compacted for 20 minutes under 15kN load and cured for 24 hours inside the mould with 5 kg dead load imposed on top

It can be seen that the percentage air voids increased due to ageing of the mixture. The results show that the volumetrics of the sample do not change significantly with only two hours of curing but change rapidly with more curing time. This may have an effect on the mixture performance as the conventional transportation and laying period can be up to six hours. The literature review revealed that rubber-bitumen interaction is a diffusion process, where the bitumen first enters the outer shell of the particle and eventually enters the centre until equilibrium swelling is achieved but the shape of the cross-linked network remains the same. In addition, the rate of swelling depends on the particle size of the rubber and the concentration of the solvent. All these factors cause rubber particles to swell and ultimately for the mixture to loose bitumen from the system and become less workable. However, the voids profile for all three series of tests show similar trends ensuring the consistency of the sample production procedure.

Figure 6.8 shows the average air voids of different composite mixtures against curing time. It can be seen that samples with harder bitumen (M3, V3) become more voided due to curing in the loose stage compared to samples produced using softer bitumen (M1, V1). The result indicates that the absorption of the lighter fractions of bitumen has more effect on harder bitumen (contains more asphaltenes) compared to the softer one (contains more aromatics) and, therefore, overall workability of the mixture is reduced.

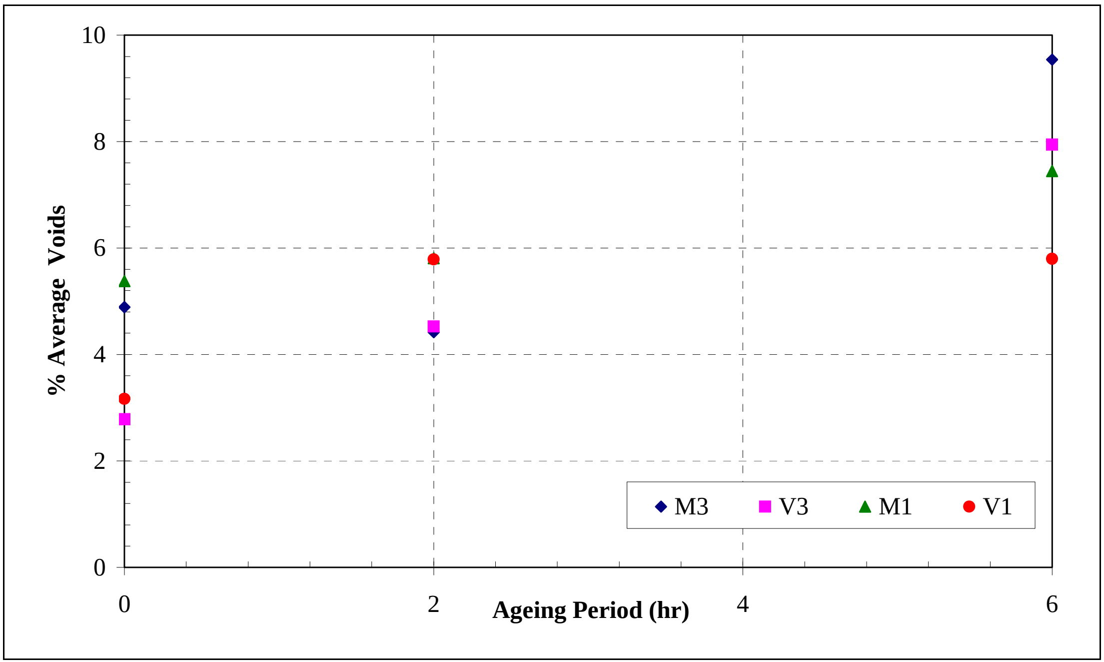


Figure 6.8: Voids profile for rubber-bitumen composite samples (rubber = 85%, bitumen = 15%) with 15kN compaction effort with 20 minutes duration and cured for 24 hours inside the mould with 5kg dead load imposed on top

6.2.7 Dimensional Stability

The expansion (vertical and radial) of the sample was monitored continuously soon after the compaction was completed. As most of the dimensional distortions (expansion in vertical and radial directions) of the sample occur at an early age when the sample is at an elevated temperature, the expansion was monitored at each hour for the first six hours. However, at this stage, only vertical expansion was recorded as the mould prevents any radial movement. After 24 hours, the sample was removed from the mould and two LVDTs were mounted (Figure 6.9), one on top and one in a radial direction, to monitor the expansion of the sample at room temperature and over a longer period of time.

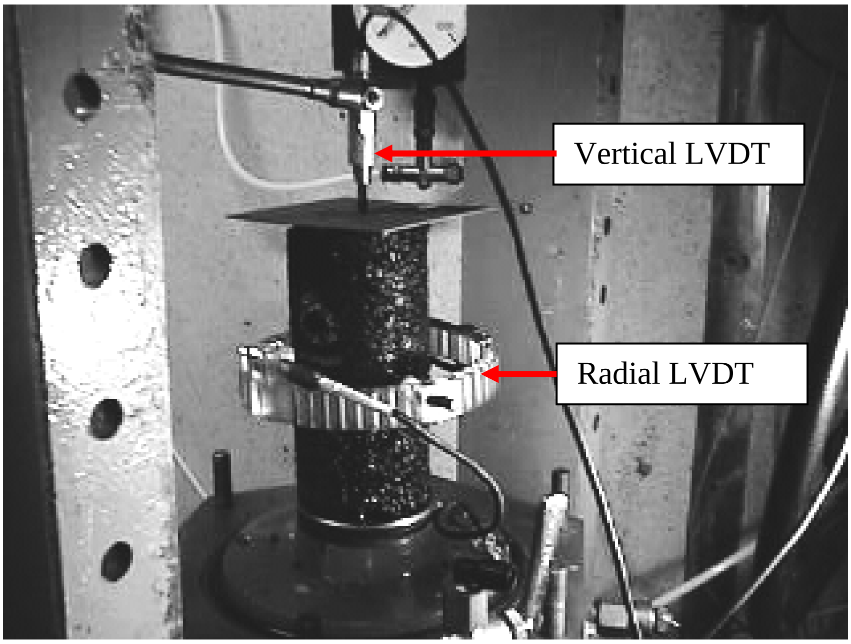


Figure 6.9: Dimensional stability testing arrangement

The radial transducer was mounted at the mid height of the sample to ensure that the collar frame was diametrically opposite to each mounting device. Two curved plastic-mounting pieces, each with a base width of 20 mm were directly glued to the specimen to ensure proper seating without imparting any load to the specimen. The expansion was monitored without application of external load for 72 hours. The test was conducted on specimens produced using the same bitumen and same rubber to bitumen ratio but with different compaction and post compaction curing techniques. The dimensional stability test results are listed in Table 6.1 including a brief description of the three compaction and curing techniques. The results are measured in terms of vertical and radial expansion in millimetres compared to the original height of the specimen. Both vertical and radial expansions are also presented graphically in Figures 6.10 and 6.11.

Table 6.1: Summary of the dimensional stability as a function of different compaction and curing techniques for rubber bitumen composite mixtures

Compaction techniques		**Post compaction curing**	**Total expansion after 72 hours (mm)**	
Method	Description		Vertical	Radial
1	Load: 15kN Duration: 10 minutes Compaction from top	Specimen inside the mould for 24 hours	14	0.63
2	Load: 15kN Duration: 20 minutes Compaction from top	Specimen inside the mould for 24 hours 5kg dead load for 24 hours	6	0.41
3	Load: 15kN Duration: 20 minutes Compaction from top and bottom	5kg dead load for 24 hours	2	0.01

It can be seen from Figures 6.10 and 6.11 that both vertical and radial expansion reduced significantly when the sample was compacted at both ends and cured for 24 hours inside the mould with 5kg deal load imposed on it. The expansion of the sample is negligible when the specimen was in the mould, which suggests that the 5kg dead load was enough to stop any vertical movement. The maximum expansion happened soon after the samples were removed from the mould and became virtually constant within a few hours. Method 3 showed the least distortion with less than 1.5% expansion in the vertical direction and only less than 0.1% in the radial direction indicating adequate compaction and good integrity of the sample.

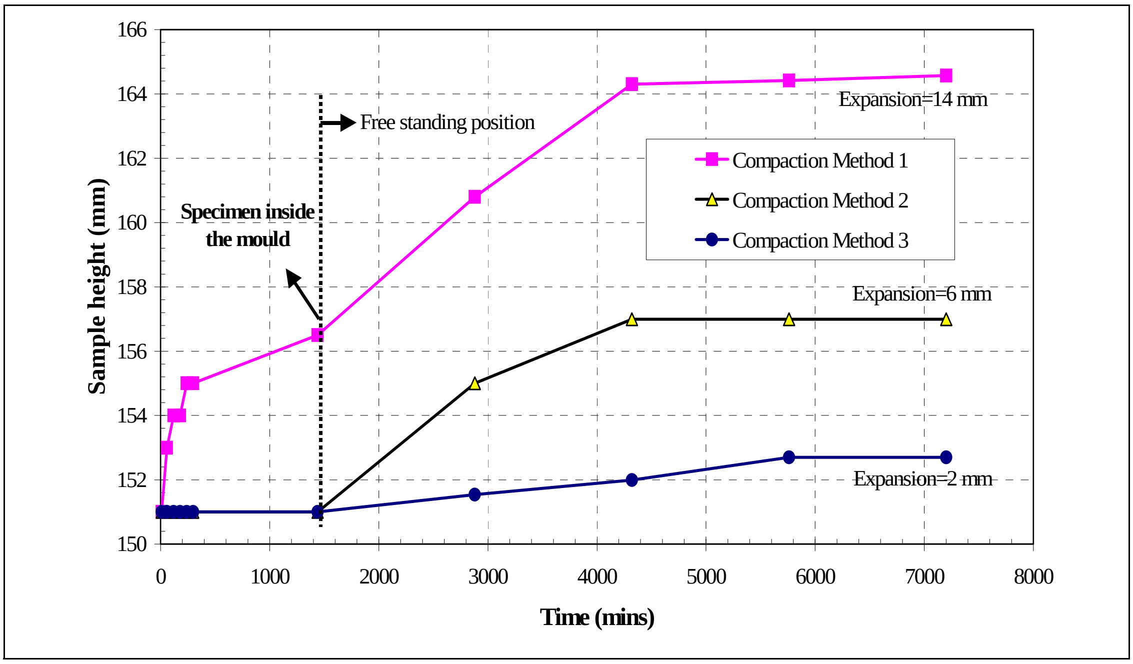


Figure 6.10: Vertical expansion of composites samples using 85% rubber and 15% bitumen by mass with different compaction techniques

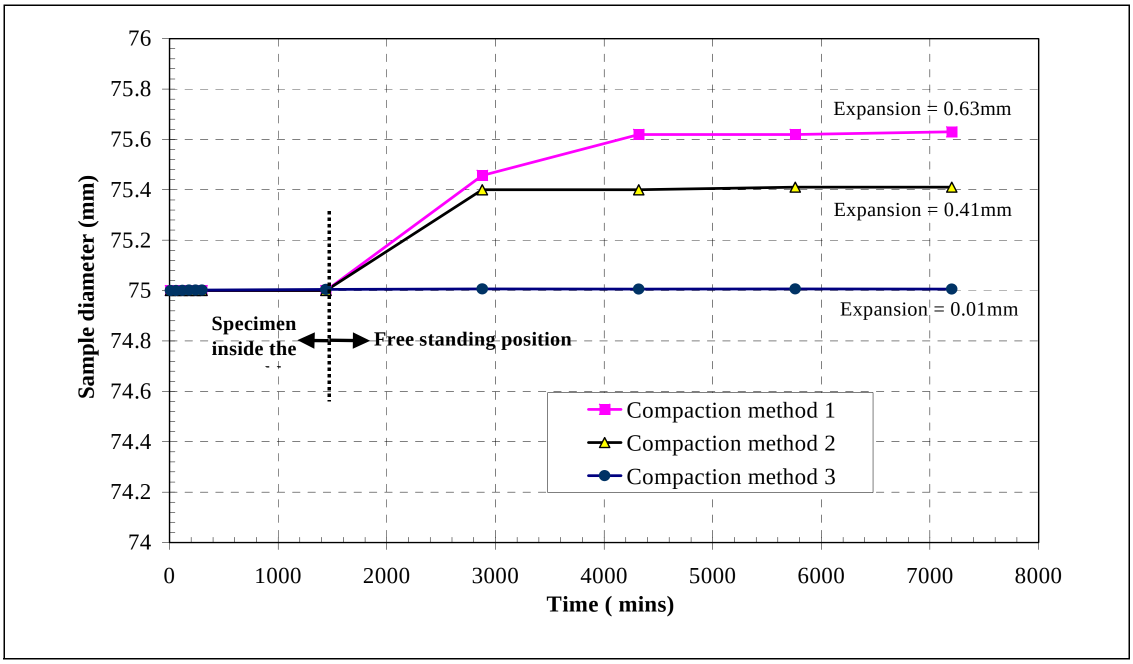


Figure 6.11: Radial expansion of composites samples using 85% rubber and 15% bitumen by mass with different compaction techniques

6.3 TRIAXIAL TESTING OF RUBBER-BITUMEN COMPOSITES

Dynamic triaxial compression testing was initially chosen for the rubber-bitumen composites investigation as it was suspected that the specimen would require some level of confinement to withstand dynamic loading as well as to simulate field conditions. However, it was found that the rubber-bitumen samples could be tested repeatedly at a low stress-strain level without any confinement and the objectives were therefore changed to calculate complex modulus and phase angle in the linear stress-strain region under various conditioning regimes (short-term ageing, bitumen types, etc.). Finally, it was decided that the experimental programme would be limited to using the triaxial set-up and covering the sample with a latex membrane to provide some extra lateral support but without applying any level of confinement. Thom (1988) measured the effect of a membrane on the lateral support of the sample and reported that the effect is only 4-5 kPa.

In the following sections, a brief overview of the testing arrangement, sample preparation, testing procedure and protocol will be presented, including test results for the various tests (frequency, stress level, temperature) and mixture conditioning (ageing, bitumen types and grades) regimes. At the end, a brief discussion on the mechanical properties of rubber-bitumen composites will be presented together with an overall summary.

6.3.1 Testing Arrangement

The triaxial testing equipment was located inside a temperature-controlled (-5^{0}C to 70^{0}C) room and consisted of a loading frame, an axially mounted hydraulic actuator, a triaxial cell and instrumentation as shown in Figure 6.12. The triaxial cell incorporated an epoxy resin base, perspex sidewalls and an aluminium cell top. The base was made of resin to eliminate any bi-metallic corrosion caused by different metals in contact with water (the original cell fluid). The cell was designed to test specimens of 150 mm height and 75 mm diameter. It was fitted with castors, to eliminate the need to carry the triaxial cell and limit sample disturbance. Finally, an impervious latex membrane 78 mm in diameter and 267 mm in length was used to seal the specimens to provide lateral support during axial loading. Measurements were marked on the membrane without stretching it to locate its centre.

A controller with data acquisition system was linked with the equipment from outside the temperature-controlled room. Hydraulic power was supplied by a pump at a normal operating pressure of 14 MPa. The control system compared the output of the relevant feedback transducer with the input command signal, and the difference was amplified and fed to the servo-valve which adjusted the oil flow to the actuator to reduce the difference between the input and output signals (error signal), so that the actuator responded precisely to the command signal. A high speed (2μs) data acquisition system was used to capture the voltage output from the control unit.

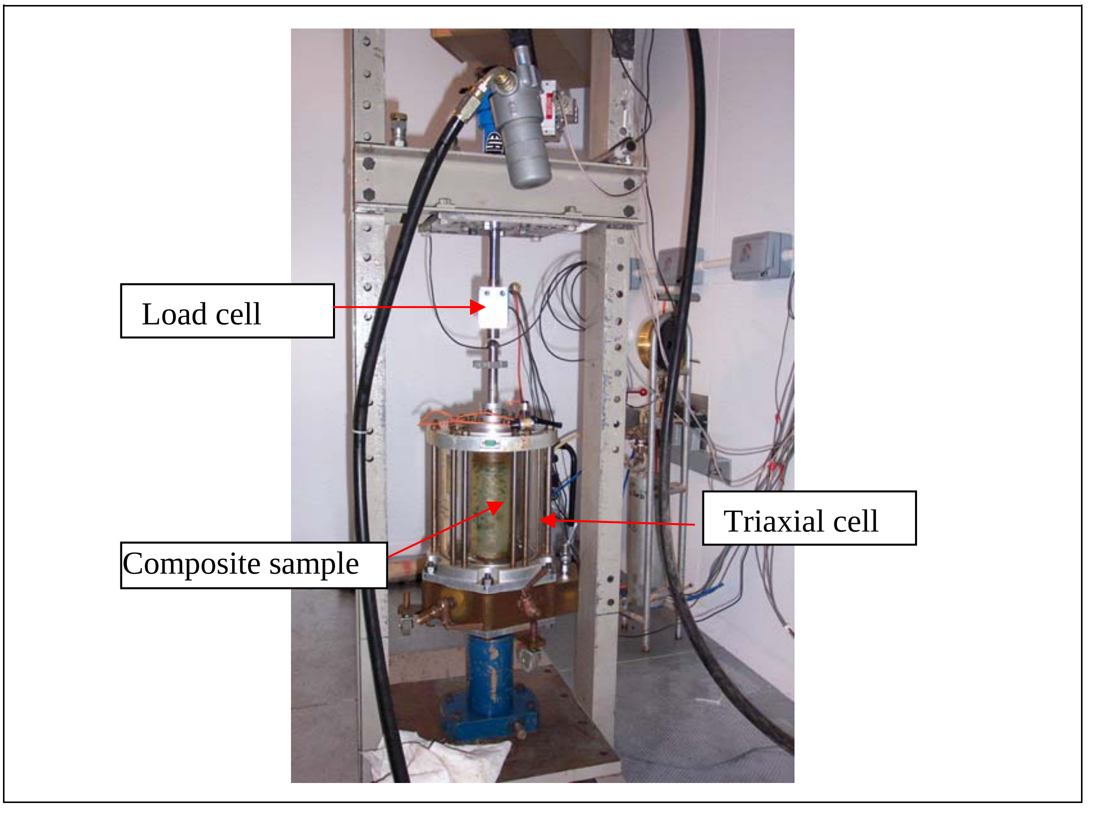


Figure 6.12: Triaxial testing arrangement

The axial load was applied to the sample through a submersible piston, connecting the triaxial cell directly to the hydraulic actuator by sliding smoothly through the end plate of the triaxial cell whilst still providing an airtight seal. The load cell was located on the piston above the triaxial cell. The load cell provided the feedback signal required for the control system. There was an option for position control, which used a long range LVDT, connected to the load ram to provide the feedback signal.

It is important to note that rubber is a highly elastic material with low stiffness modulus and can produce large strains under small load applications. Therefore, the testing required a very low range of loading (0.05kN to 0.5kN) to measure strain accurately without producing excessive strain. Most testing equipment in civil engineering contains larger loads cells (minimum 5kN). Consequently, the testing equipment was modified and a 1kN load cell was used to apply the low loading range. However, to ensure that the piston moved freely inside the triaxial cell, a sinusoidal displacement was applied to the piston, which was free to move in the triaxial cell without any specimen. The load cell recorded zero loads, which confirmed that there was minimal friction between the cell and the piston.

6.3.2 Calibration

A 1kN load cell and 10 mm range LVDT were calibrated in order to convert voltage output to load and displacement units. A number of checks were also performed to ensure that no incorrect or biased results were obtained. The load cell was calibrated by placing a proving ring under the load cell and applying 100N load increments while monitoring the voltage output through the data acquisition system. A linear calibration curve was then plotted to convert voltage to load as shown in Figure 6.13.

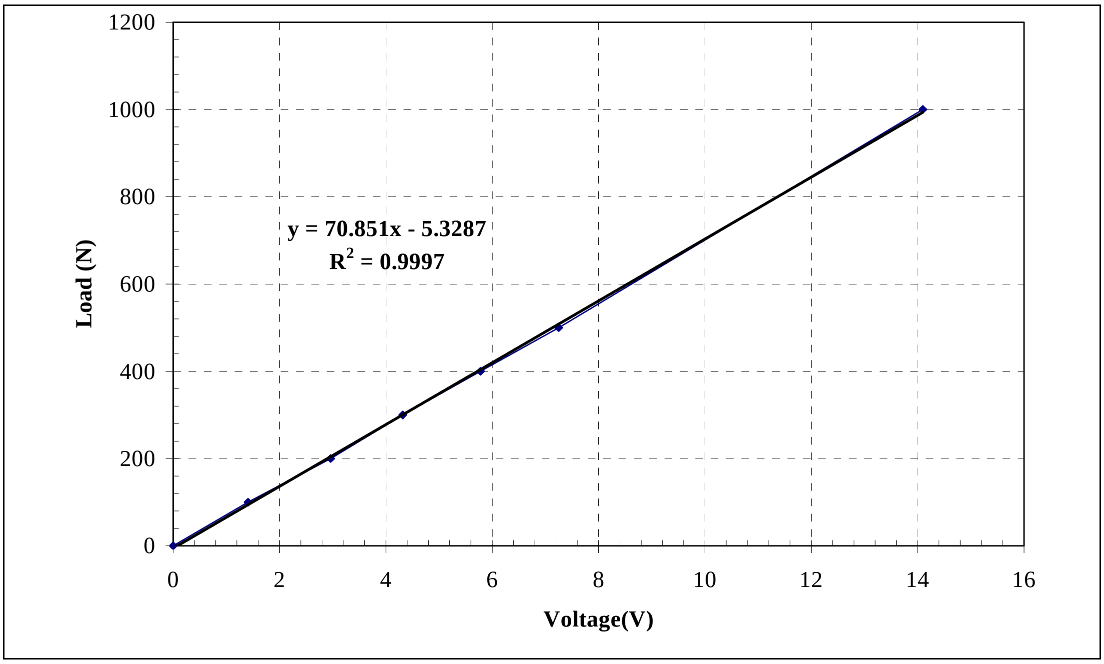


Figure 6.13: Calibration curve for load cell

The LVDT was calibrated using a micrometer fixed into a calibration frame. Increments of 20µm were used and the calibration equations are plotted in Figure 6.14. The displacements were therefore simply calculated by inputting the voltage output of the LVDTs into the equation.

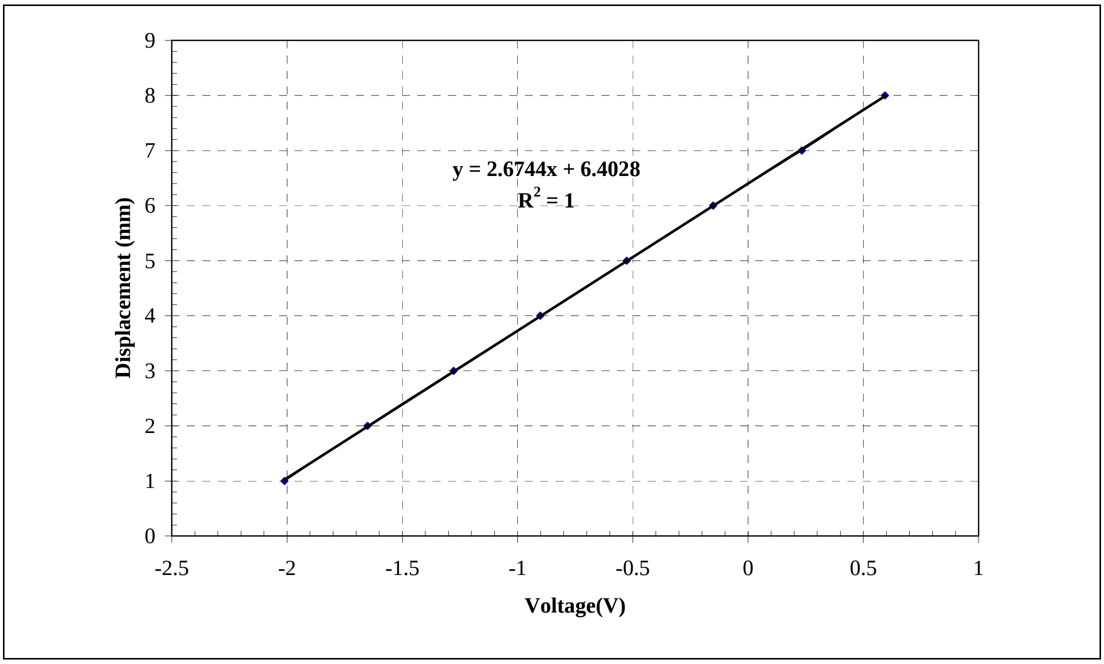


Figure 6.14: Calibration curve for vertical LVDT

It is important to note that load cell and LVDT signal conditioning should not affect the phase difference from stress and strain response. To verify this, a series of dynamic tests at different frequencies were carried out using a purely elastic steel spring with known stiffness and the phase difference was checked using the best-fit curve for load and displacement (Figure 6.15). The phase angle difference was calculated at approximately 0.41^0, which is near enough to zero to serve the purpose of this project.

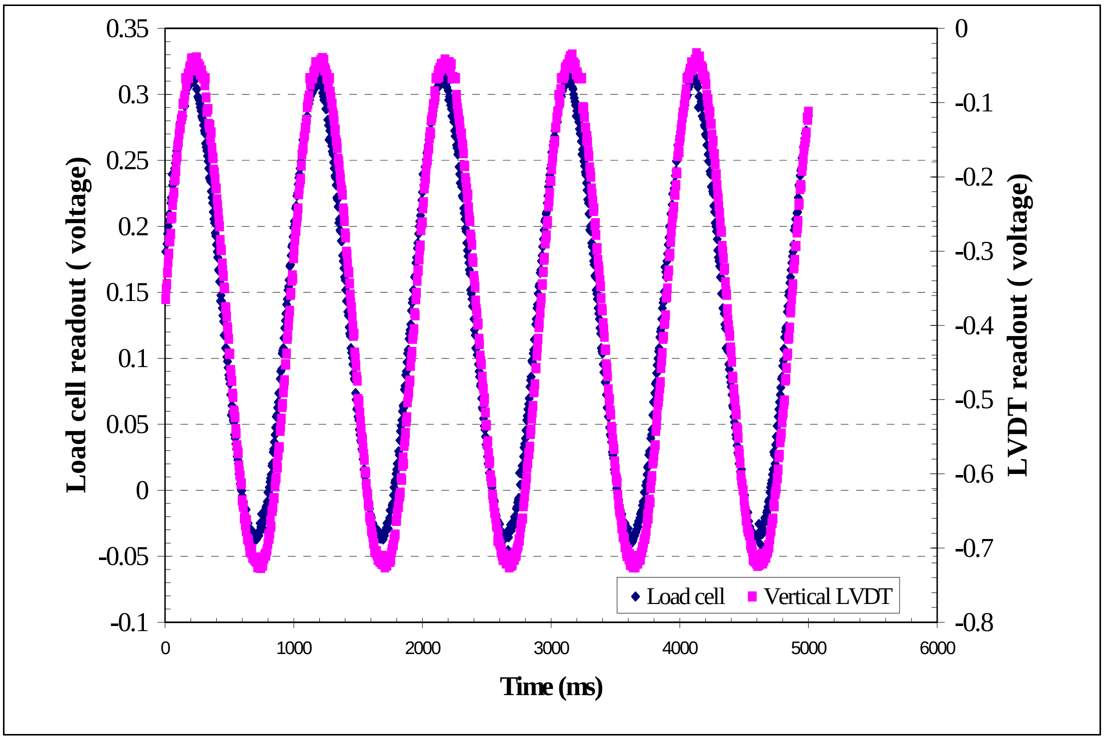


Figure 6.15: Load and LVDT readout for a perfectly elastic spring tested at 1Hz

6.3.3 Data Analysis

The voltage readout of the load cell and LVDT from the data acquisition system was converted to load (N) and displacement (μm) using the appropriate calibration factors. Load and deformation measurements were collected for the last five cycles of the first thirty recorded cycles with 100 data points per cycle. After each test, the load cell and LVDT were set to zero before starting the next test. A data extraction programme using MATLAB was modified (Rahimzadeh, 2002) to fit the sinusoidal curves of stress and strain and complex modulus and phase angle were calculated using the following formulas:

$$\phi = \frac{t_i}{t_p} \times (360^0) \qquad \textbf{\textit{(6.5)}}$$

Where:

ϕ = phase angles

t_i = time lag between the peak sinusoidal stress and strain

t_p = time for a complete stress cycles

$$|E| = \left[\frac{stress\ \ amplitude}{strain\ \ amplitude} \right] \qquad \textbf{\textit{(6.6)}}$$

6.3.4 Testing Protocol

The rubber-bitumen composite samples produced under different conditioning regimes were subjected to repeated load triaxial compression testing at different stresses, frequencies and temperatures. Four different types of bitumen, two hard (V3, M3) and two soft (V1, M1) with one rubber type and a fixed rubber to bitumen ratio were used to manufacture and test composite specimens to investigate the effect of bitumen crude sources and grades on the mechanical interaction. Each mixture was subjected to conditioning for 0, 2 and 6 hours at 160^0C prior to compaction in a forced draft oven to simulate the effect of short-term ageing on the composite mixtures. Each sample was conditioned for at least four hours at test temperature prior to testing and then tested under sinusoidal loads (stresses) over a range of frequencies. The resultant vertical displacement (strain) was measured using a LVDT in order to calculate complex modulus and phase angle. The detailed testing protocols are listed in Table 6.2.

Table 6.2: Material and testing protocol for triaxial testing

Mixture	**Bitumen type in the mixture** (*Rubber=85%, Bitumen=15%)*	**Short-term ageing (hours)**	**Test temperature (^{0}C)**	**Stress (kPa)**	**Frequency (Hz)**
CM1	160/220p Middle East	0,2,6	5,20,35	10, 20,30, 40, 50	0.2,0.5 1,2,5
CV1	160/220p Venezuelan				
CM3	40/60p Middle East				
CV3	40/6p Venezuelan				

The testing programme was divided into two main series. In the first series, the stress-strain relation of the composite mixtures was investigated. It is important to note that rubber-like material behaves as a linearly elastic substance at very small strains although there is no agreement as to how small the "small" strains may be (Sommer and Yeoh, 1992). Sommer and Yeoh also reported that fitting the best straight line to experimental stress-strain data could be used to calculate stiffness for practical purposes. Therefore, stress-strain curves were plotted at each temperature and frequency level to calculate dynamic stiffness modulus and phase angle. In addition the results were also interpreted to perform sensitivity analysis in terms of the effect of stress levels, frequencies, temperatures and bitumen types on the mechanical response (stiffness and phase angle) of the mixtures. Finally data was compared to understand the effect of bitumen types, grades and the effect of short term ageing at high temperature prior to compaction on mechanical performance in terms of phase angle and complex modulus.

6.4 TEST RESULTS

6.4.1 Stress-Strain

Selected stress-strain plots of the composite mixtures are presented in Figures 6.16 to 6.19 for three ageing conditions tested at different temperatures. In general, it can be seen that even at low stress levels, the response of the mixture is not linear with the very high degree of elasticity generating high strains. The results also indicate that the composite mixture is not significantly affected by the frequencies but affected by the increase in test temperature. In addition, the stress-strain behaviour is similar for all three ageing conditions including mixtures produced using both hard and soft bitumen.

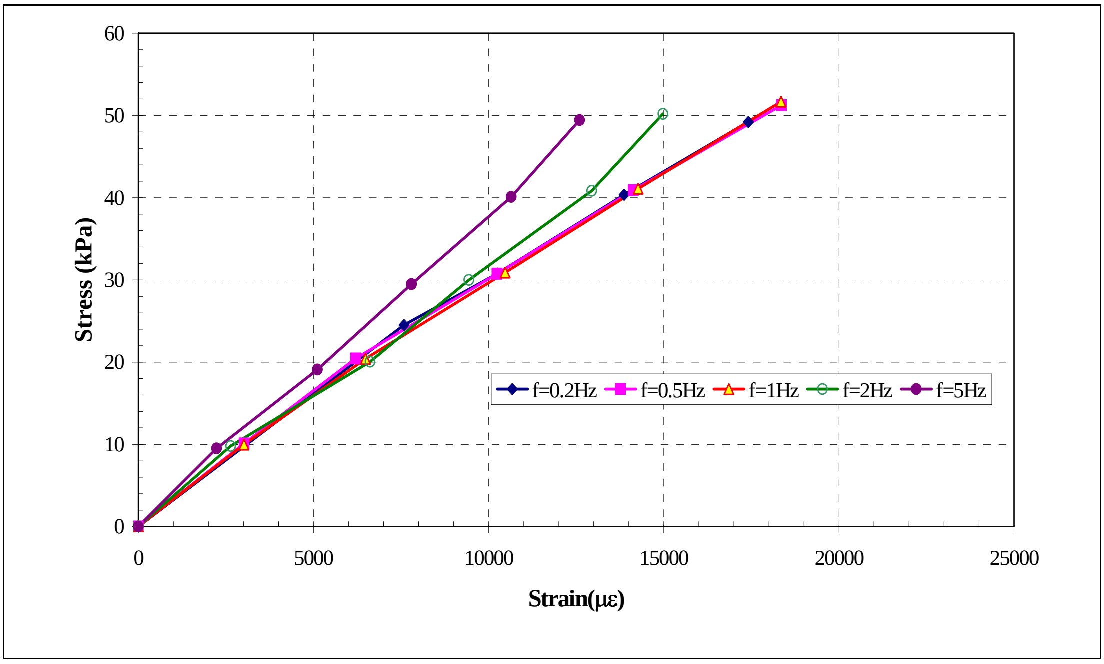


Figure 6.16: Stress-strain curve of composite produced using M3 bitumen, 0hr ageing, and tested at 5^{0}C

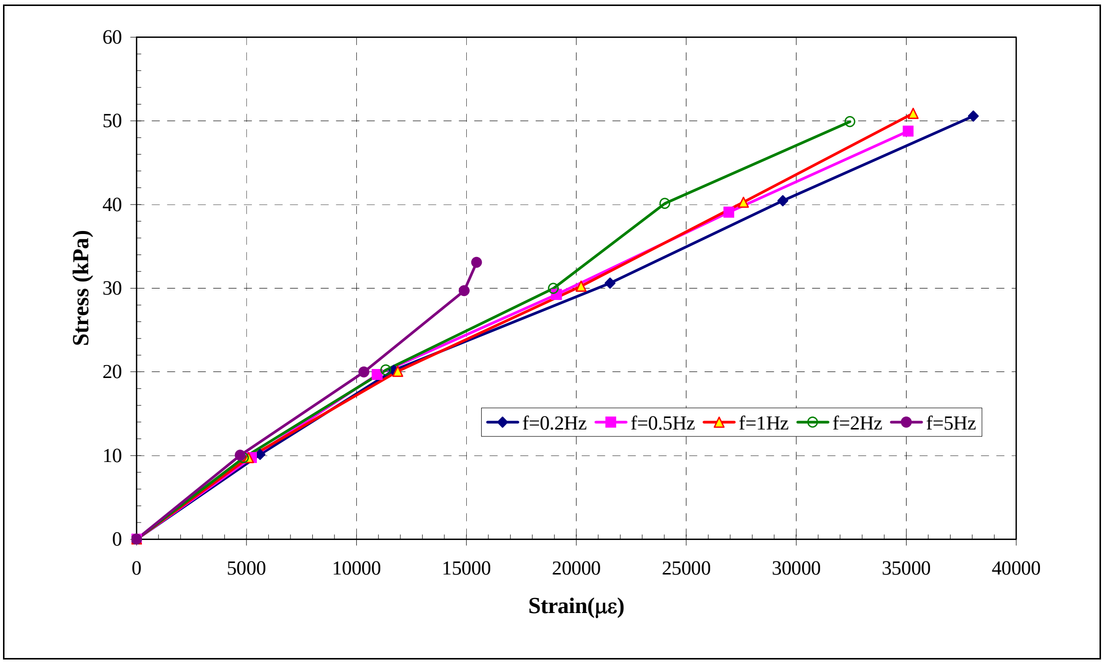


Figure 6.17: Stress-strain curve of composite produced using M1 bitumen, 2hrs ageing, and tested at 20^{0}C

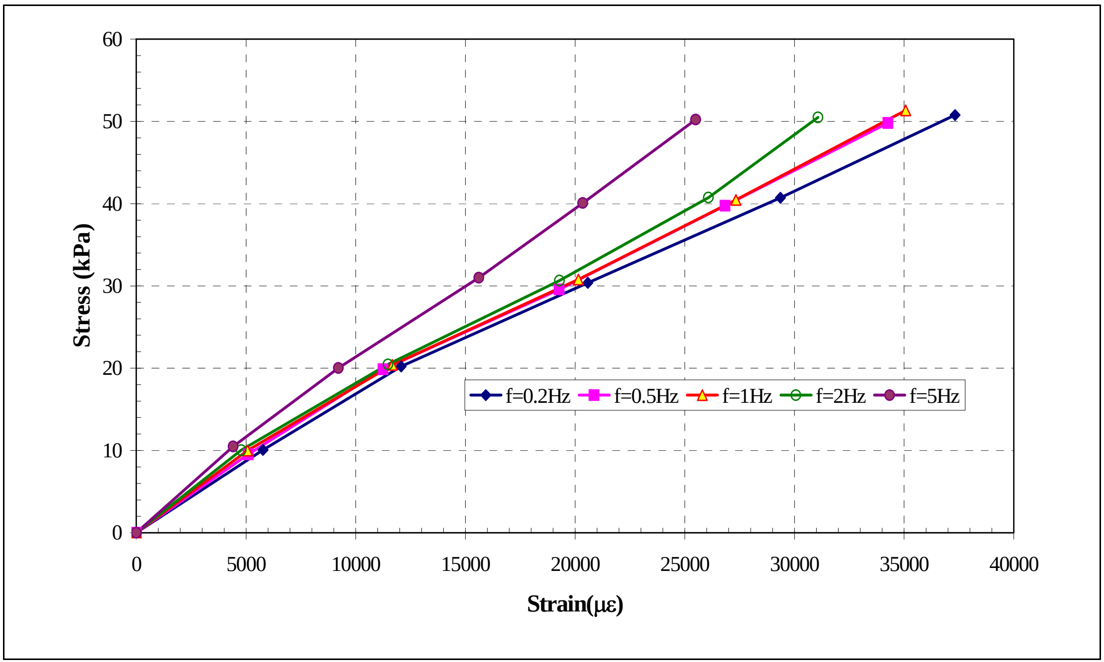


Figure 6.18: Stress-strain curve of composite produced using V3 bitumen, 6hrs ageing, and tested at 20^{0}C

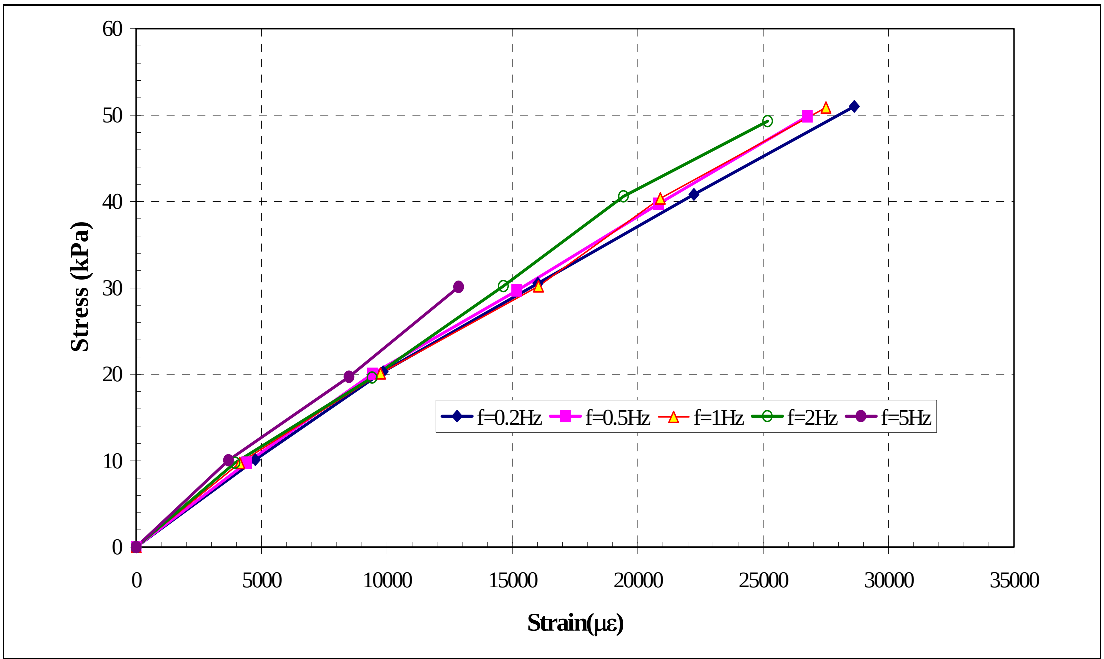


Figure 6.19: Stress-strain curve of composite produced using M3 bitumen, 0hr ageing, and tested at 35^{0}C

However, it is difficult to identify the influence of all the variables using this form of data presentation. Therefore, a sensitivity analysis was performed for all composite mixtures using one set of independent variables to predict the influence of the other variables.

6.4.2 Sensitivity Analysis

The effect of the independent variables were used to predict the composite stiffness and phase angle by means of a number of sensitivity plots using one set of original parameters. The following variables were kept constant to analyse the effect of the other variables.

- Loading frequency - 1Hz
- Temperature - 20^0C
- Stress - 30kPa

Figures 6.20 and 6.21 present sensitivity plots in terms of complex modulus for CM1 and CV3 mixtures produced straight after mixing (0 hour conditioning) and Figure 6.22 presents a sensitivity plot for CM1 mixture subjected to 6 hours short-term conditioning. Similar plots were also produced for the other mixtures and in general, showed that the overall response at different temperatures, frequencies and stress levels is similar for all mixtures irrespective of bitumen type, grades and conditioning regimes. It also showed that increases in testing temperature significantly reduced mixture stiffness which was only marginally influenced by loading frequency and stress level. The change in stiffness with variations in temperature is probably due to the combined influence of the more elastic response of the bitumen at low temperatures and elastic response of rubber particles at high temperatures. In addition, the results also demonstrate that the stiffness of the mixture is significantly lower for composites produced using soft bitumen (Figure 6.20) compared to a mixture produced using a harder bitumen (Figure 6.21). As all mixtures were produced using the same percentage of rubber and bitumen using similar compaction effort and curing techniques, the softer bitumens are probably the main contributing factor for low stiffness. Another possibility is the higher percentage of bitumen absorption in soft bitumen (M1) during short-term ageing at 160^0C as swelling test results indicated that the bitumen absorption could be as high as 20% for M1 bitumen when the test temperature increased from 140^0C to 160^0C. Consequently, the rubber particles become softer and as the percentage of rubber particles is higher in the mixture, it is the rubber's properties that dominate the overall response of the composite mixtures.

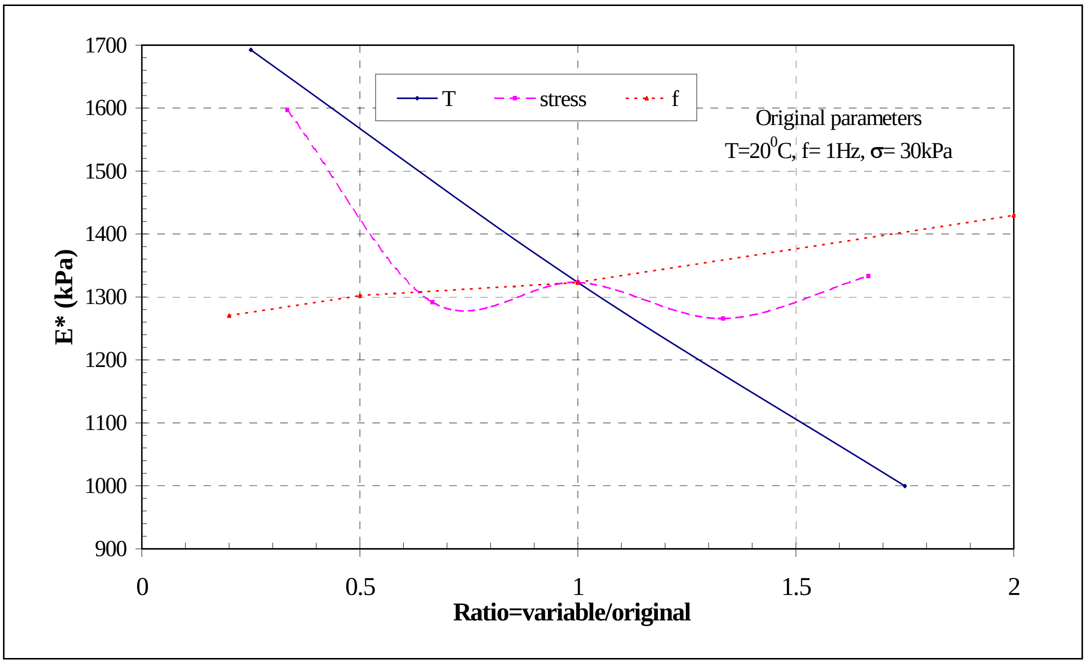


Figure 6.20: Sensitivity plot of complex modulus of CM1 mixture produced using after 0hr conditioning in loose stage

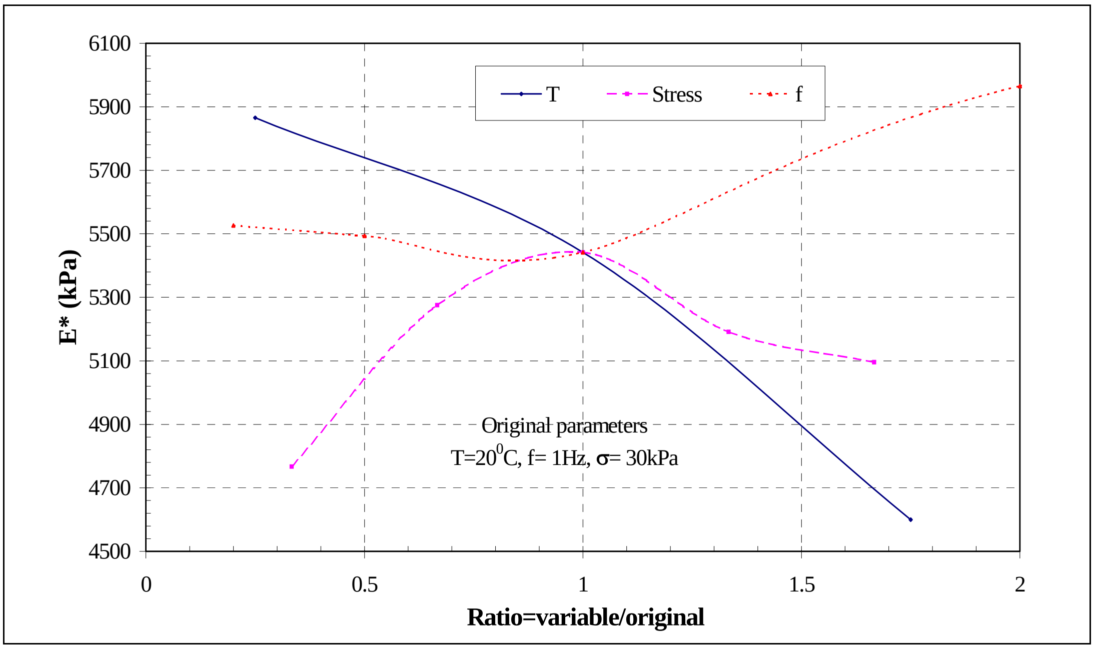


Figure 6.21: Sensitivity plot of complex modulus of composite mixture produced using CV3 mixture and aged 0hr

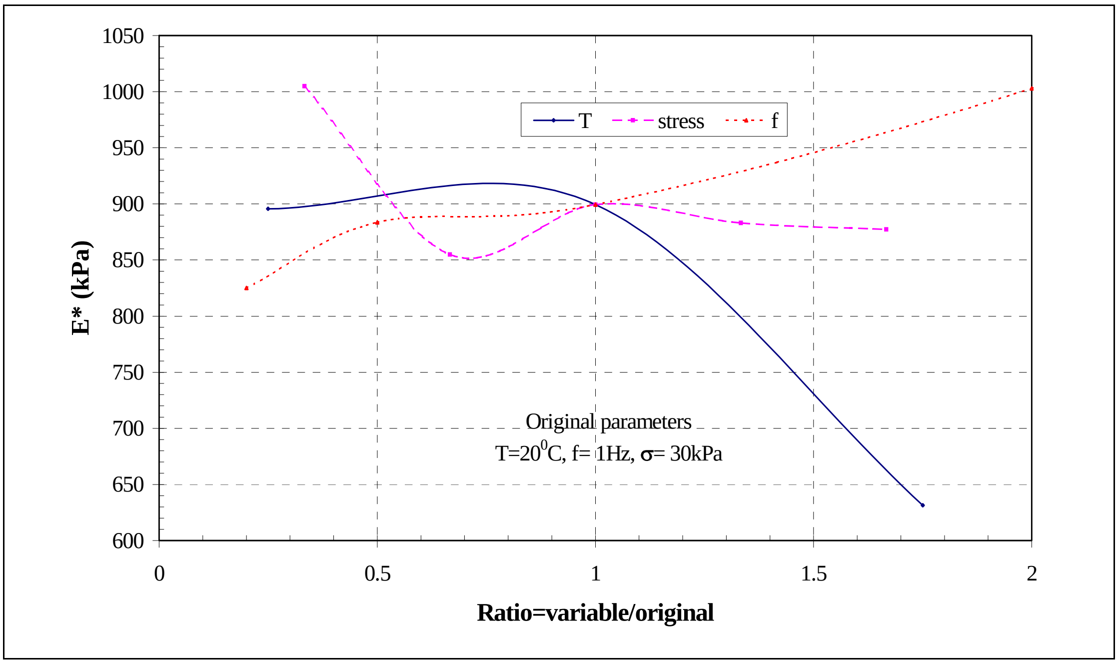


Figure 6.22: Sensitivity plot of complex modulus of composite mixture produced using CM1 mixture and aged 6hrs

In terms of phase angle, Figures 6.23 and 6.24 show sensitivity plots of two composite mixtures (CM3 and CV1) produced using soft and hard bitumen. Similar plots were drawn for the other mixtures in all ageing combinations and in general, the results indicate that the influence of frequency on phase angle is minimal, but that the mixtures are marginally influenced by testing temperature and stress level. This may be due to the dominance of the rubber component, as it comprises a higher proportion in the mixture.

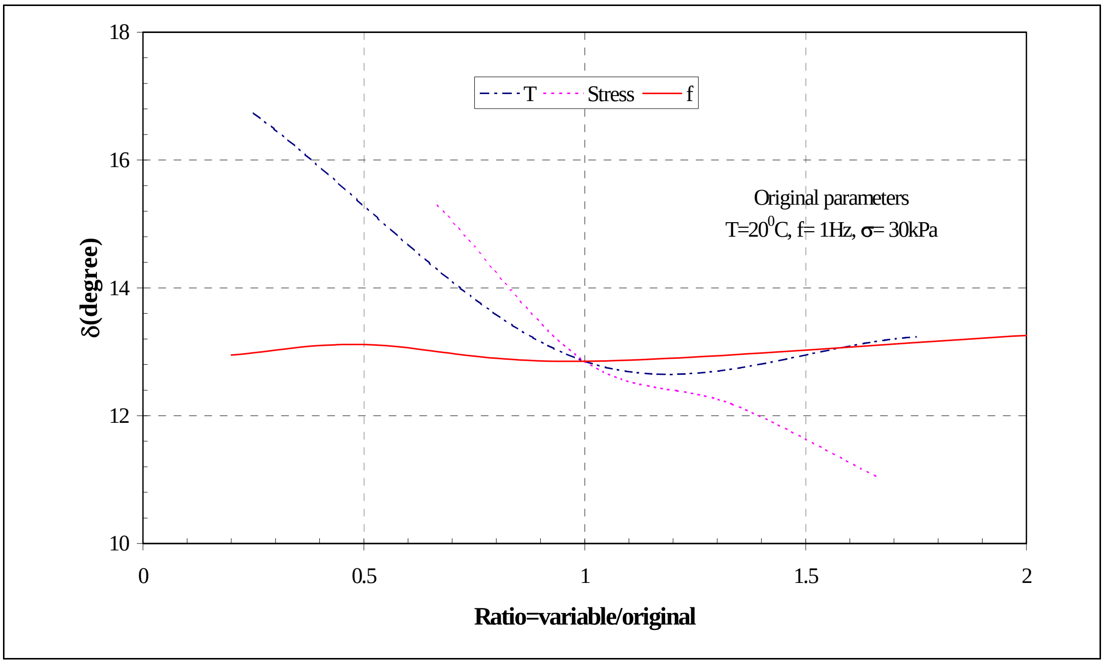


Figure 6.23: Sensitivity plot of phase angle of composite mixture produced using CM3 mixture and aged 0hr

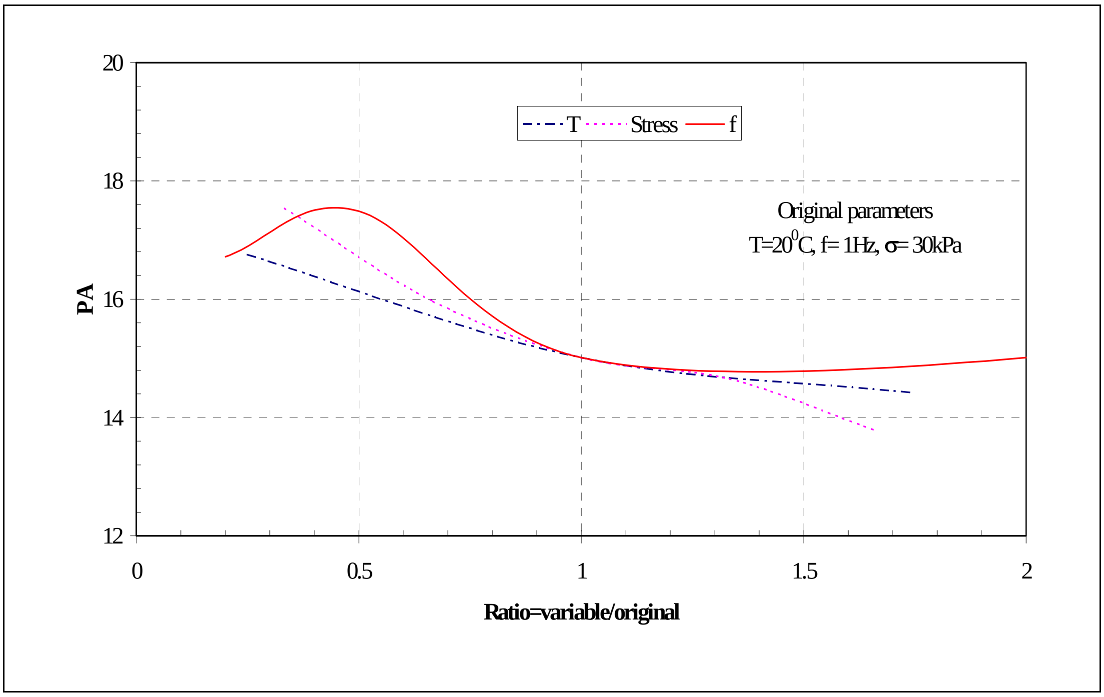


Figure 6.24: Sensitivity plot of phase angle of composite mixture produced using CV1 mixture and aged 6hrs

6.4.3 Rubber-Bitumen Interaction

Figures 6.25 and 6.26 show the influence of short term ageing and testing temperature on the stiffness for CM3 and CV1 composite mixtures tested at f=1Hz and at stress level 30kPa. The results clearly demonstrate that the stiffness is dependent on the test temperature. It can also be seen that the high temperature ageing of the mixture prior to compaction reduced the mixture stiffness.

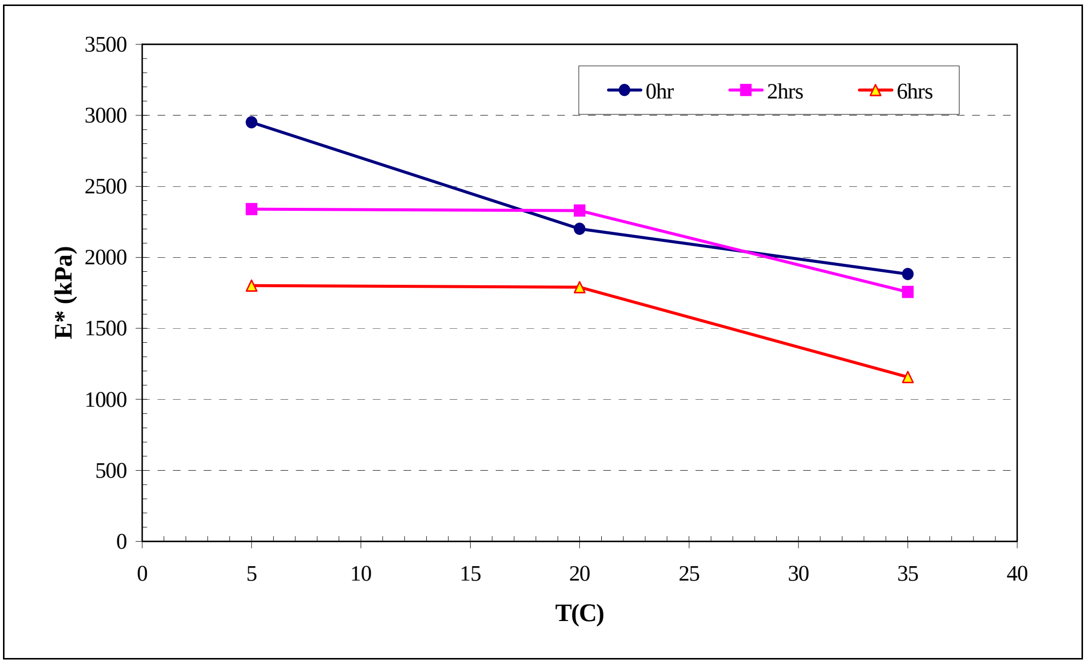


Figure 6.25: Complex modulus versus temperature for CM3 mixture tested at f=1Hz, Stress level =30kPa

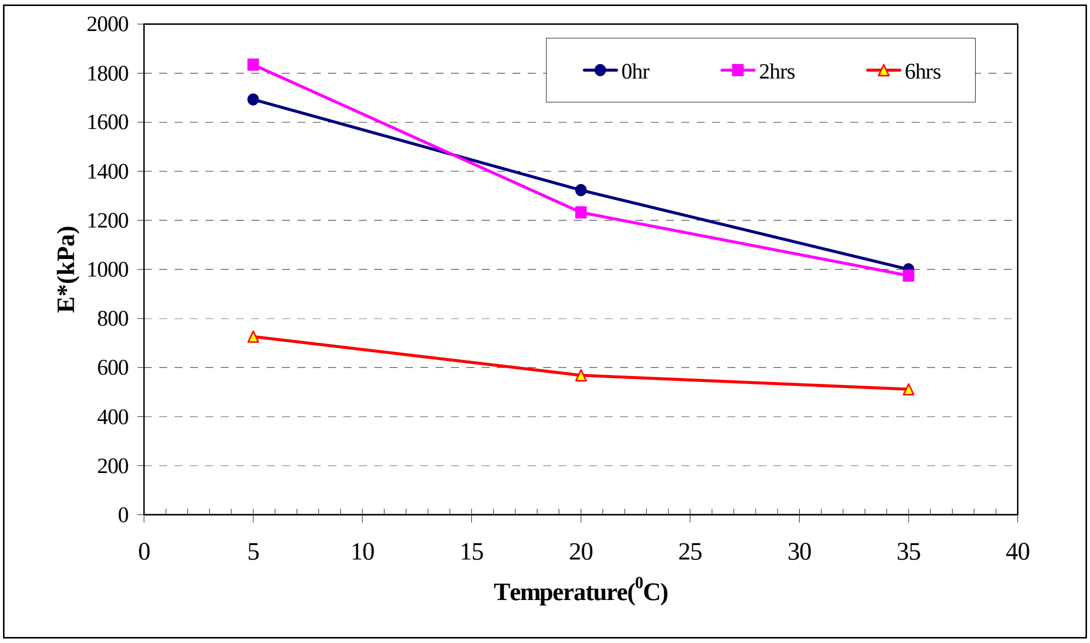


Figure 6.26: Complex modulus versus temperature for CV1 mixture tested at f=1Hz, Stress level =30kPa

However, the reduction in stiffness could also be attributed to the results from Figure 6.8 which showed that voids content of the samples also increased with ageing and an increase in voids contents has an effect on mixture stiffness. To identify the influence of void content on mixture performance, complex modulus versus voids content at three ageing conditions for four different mixtures tested at 20^{0}C and 1Hz were analysed and presented in Figure 6.27. The results show a high degree of scatter with only a 58% correlation although the stiffness of the mixture generally decreased with the increase in voids content. To review the influence of void content, the results obtained from the 20^{0}C and 1 Hz test were normalised to 6% voids and plotted in Figure 6.28 in terms of stiffness and conditioning periods.

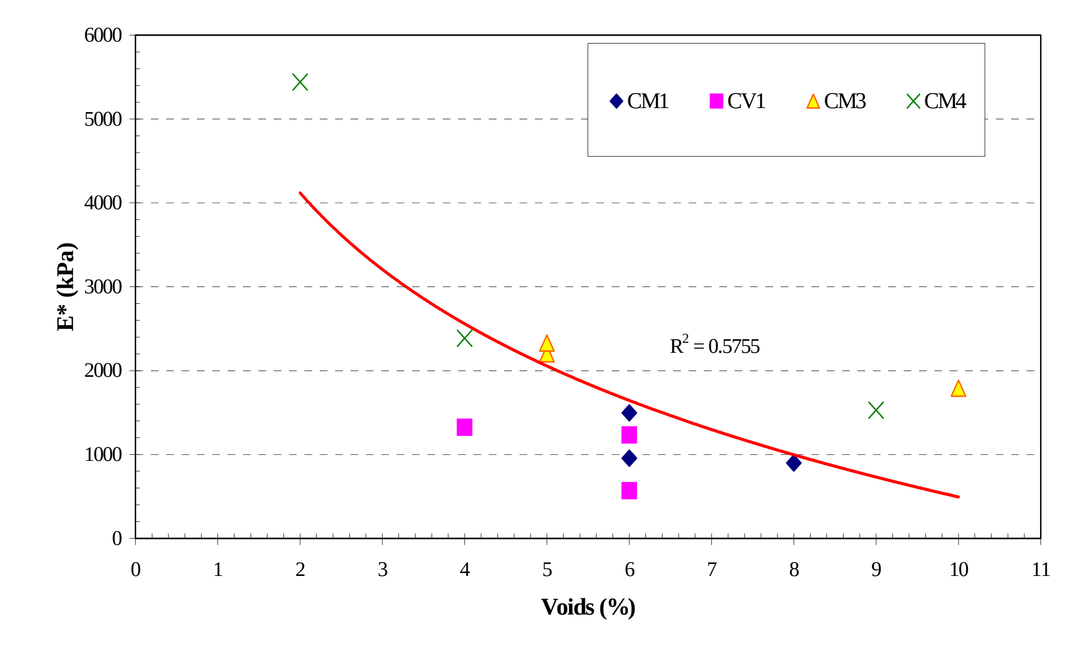


Figure 6.27: Complex modulus versus percentage of air voids of all composite mixtures produced after ageing 0,2 & 6 hours and at f=1Hz, T=20^{0}C and Stress level= 30kPa

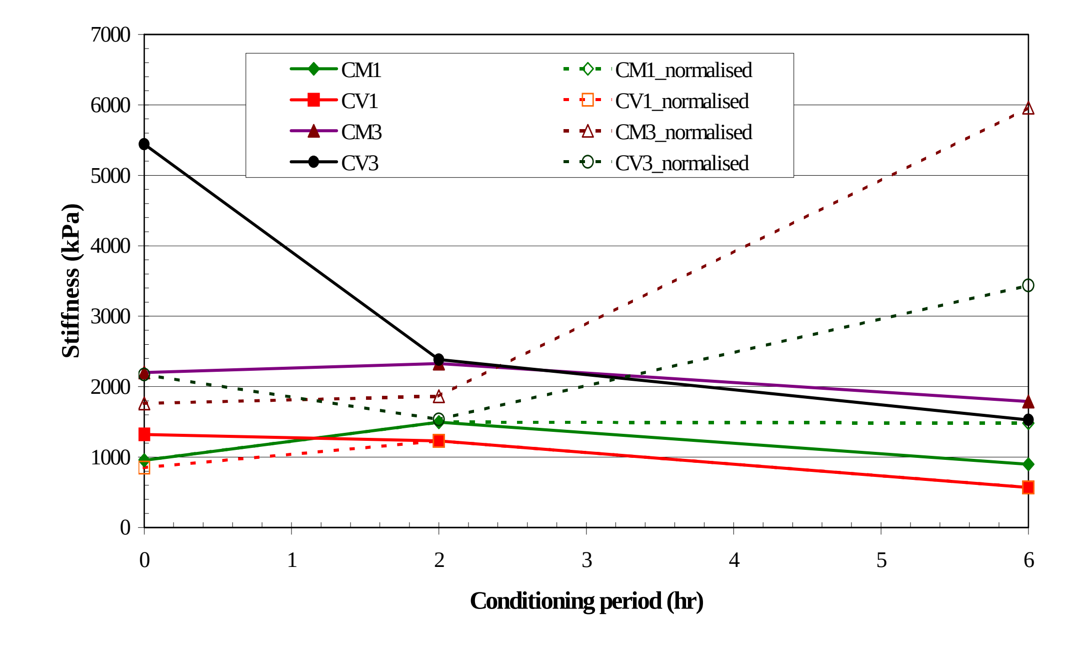


Figure 6.28: Complex modulus versus conditioning period of all composite mixtures produced at 0,2 & 6 hours short term ageing and tested at f=1Hz, T=20^{0}C and stress level= 30kPa

The results show that the normalised stiffness for the softer bitumens (CM1 and CV1) decreased due to ageing although the normalised stiffness increased for both mixtures produced using the harder bitumens (CM3, CV3). This might be due to the higher percentage of asphaltenes in M3 and V3 bitumen contributing to the increase in the overall stiffness of the mixture following conditioning for 6 hours at 160^{0}C. In contrast, the ageing at 160^{0}C for CM1 and CV1 may have less effect on the bitumen because the bitumen contains higher percentages of the maltenes fractions.

In terms of phase angle, Figure 6.29 shows results obtained from different composite mixtures aged for 0, 2 and 6 hours and tested at 20^{0}C, stress level 30kPa and frequency 1Hz. The result suggests that the composite mixtures are highly elastic and not influenced by bitumen type and grades, voids content and ageing of the mixture. The results also demonstrate that although the material has become more voided and less stiff with ageing, the highly elastic property of the rubber is a dominant feature in the composite mixtures with low phase angles (14^0-18^0).

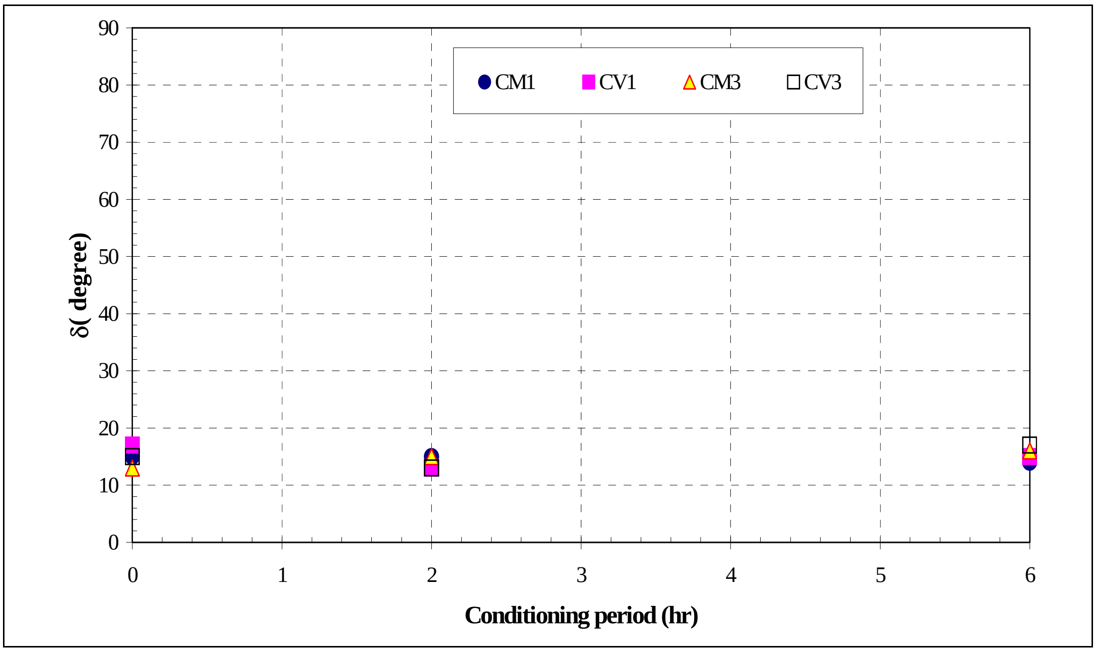


Figure 6.29: Phase angle of different composite mixtures tested at 20^{0}C, stress level =30kPa and f=1Hz

6.5 SUMMARY

An idealised rubber-bitumen composite mixture using four different types of bitumen and one single source of rubber has been studied in this chapter. This chapter illustrates the development of a novel technique to test rubber-bitumen composites specimens that can simulate actual rubber-bitumen proportions in dry process CRM asphalt mixtures. In addition, dynamic mechanical properties under different conditioning and testing regimes were also investigated to measure the complex modulus and phase angle of the different composite mixtures.

The novel technique consisted of using the bitumen film thickness concept to find a realistic mixture proportion. Ranges of composite mixtures were produced using hard and soft bitumen and subjected to short-term ageing up to six hours at 160^{0}C prior to compaction. Several trial compaction techniques were investigated and finally compaction similar to the Marshall method was found to be suitable to produce repeatable samples. A dimensional stability check was also performed on all samples to check the stability of the specimen geometry during the storage period and the sample's suitability for triaxial compression testing.

Cylindrical specimens were produced, tested in a triaxial set-up with the sample covered with a latex membrane and subjected to dynamic sinusoidal loading. All the mixtures were tested at five stress levels (10kPa to 50kPa), five frequencies (0.1, 0.2, 1, 2, 5Hz) and three temperatures (5, 20, 35^{0}C). The resultant axial strains were measured and used to calculate the dynamic mechanical properties such as complex modulus and phase angle. As the mixture is highly non-linear elastic in nature due to the high proportion of the rubber content, each test was conducted at very low stress levels to ensure that the material remains within the linear region and that the stress condition does not influence the test results.

The stress-strain behaviour was plotted for different mixtures under different testing conditions. The response was found to be non-linear and highly elastic in nature and the strain generated under load was significantly high indicating rubber dominance in the response. A sensitivity analysis was performed to investigate the influence of the different testing variables and it was found that the mixture stiffness is mostly influenced by the testing temperature, but marginally dependent on test frequencies.

In terms of bitumen types, the stiffness reduction is significantly higher for mixtures produced using softer bitumens (M1, V1) than for mixtures produced using harder bitumens (M3, V3). This is probably due to the combined effect of lower bitumen stiffness and higher bitumen absorption during short-term ageing at 160^{0}C. Similar observations were found in the swelling tests (Chapter 4, Section 4.4.3) with the initial absorption being considerably greater for softer bitumen when the test is conducted at 160^{0}C, which is approximately 20^{0}C higher than the equiviscous temperature of the M1 bitumen. In general, stiffness for all composite mixtures decreased due to the combined effect of short-term ageing and voids content. Phase angle on the other hand is not predominantly influenced by bitumen type and grades used, short-term ageing of the mixture, mixture volumetric, test temperature, stress level and frequency. The overall phase angle of the mixture was within the range of 14^0-18^0 and mainly influenced by the rubber component in the mixture indicating the viscoelastic nature of the CRM asphalt mixture could be compensated by using more flexible rubber particles.

CHAPTER 7

Mixture Design

7.1 INTRODUCTION

Mixture composition, preparation and curing are significant elements in the production phase that affect mixture performance in service. Currently, no widely accepted mixture design method has been developed for rubber-modified asphalt mixtures in the UK. PlusRide® and Generic (Epps, 1994) are the two most widely used dry process technologies in North America for wearing course applications (See Chapter 3). However, their field and laboratory performance are inconsistent (Epps, 1994, Fager, 2001) with limited fundamental research to understand the mixture's mechanical properties. Consequently, the dry process has become less popular, although it has a high potential to consume larger quantities of scrap tyres and is also logistically easier compared to the wet process.

The literature review revealed that irrespective of mixture gradation (gap or dense), ravelling and early life cracking are the two main distress mechanisms that occur in dry process CRM asphalt surface courses. It is also suspected that the mixture is susceptible to weathering and mechanical wear. Therefore, in this project, the mixture was designed as a binder course to avoid direct impact of mechanical wear and weathering. In addition, the mixture was designed using an existing UK gradation to minimise extra effort required in the material design stage. As the material gradation and mixture design used in this study were different from other types of CRM mixtures currently available, the term "CRM" was adopted as an acronym to represent the mixture throughout this research.

In this research project, laboratory investigations were performed to understand, predict, and explain the performance of CRM mixtures prior to full-scale field applications in the UK. Elemental stiffness modulus, fatigue and permanent deformation resistance of the CRM mixtures and durability in terms of moisture susceptibility and ageing properties were studied using the NAT and are presented in Chapters 8 and 9. In addition, an analytical modelling exercise was also performed to predict the performance of CRM mixtures in the field.

The aim of this chapter is to present a mixture design procedure for the dry process CRM asphalt mixture by substituting a percentage of the mineral aggregate from BS 4987 for a continuously graded Dense Bitumen Macadam (DBM) binder course with rubber. In the first section, a brief description of materials, CRM mixture design methodology, sample preparation protocol, compaction methodology and a brief comparison between presently available dry process mixtures versus the CRM mixture designed for this research are presented. The next section presents the mixture's volumetric calculations with a brief summary at the end.

7.2 SAMPLE PRODUCTION

7.2.1 Materials

The coarse, fine and filler mineral aggregate fractions used in this investigation consisted of limestone obtained from Foster Yeoman (Torr works quarry, Somerset, UK). The source and grade of the bitumen was Middle East 100/150Pen as specified in BS EN 12591. The crumb rubber used in this elemental mixture testing study was obtained from Charles Lawrence Recycling Ltd, produced from scrap truck tyres using the granulation method. The specific gravity for the coarse, fine, filler and bitumen fractions of the mixtures were made available by the producers. The specific gravity of the crumb rubber was determined using the gas jar method outlined in BS1377: Part 2: 1990:8.2. The grading and specifications of the aggregate, filler, bitumen and rubber are listed in Table 7.1.

Table 7.1: Aggregate, rubber and bitumen specification

Component	Type	Specifications	Specific gravity
Coarse aggregate	Limestone	Maximum 20 mm and all > 2.36 mm	2.70
Fine aggregate	Limestone	<2.36 mm and > 0.075 mm	2.65
Filler	Limestone	<0.075 mm	2.70
Bitumen	Middle East	100/150 Pen	1.01
Crumb rubber	Truck tyre crumb	2 to 8 mm	1.10

7.2.2 Mixture Design Methodology

The dry process CRM mixtures were produced by adjusting the grading curve to incorporate different percentages of crumb rubber. In practice, 1 to 3% of crumb rubber by mass of total mixture has been used depending on the type of dry process (Heitzman, 1992, Epps, 1994). In this project, 3% and 5% of crumb rubber by mass of total aggregate was used. A continuously graded, 20 mm maximum aggregate size DBM asphalt mixture, as specified in BS 4987, was modified to manufacture a range of control and dry process CRM asphalt mixtures. According to BS 4987 the bitumen content for 20 mm DBM macadam should be in the range of 4.8±0.5%. Previous studies on dry process modification suggested that the bitumen content should be 10 to 20% more than conventional mixtures (Epps, 1994, FHWA, 1997). Therefore, all the CRM mixtures were produced with a binder content of 5.25% by mass of total mixture, determined from compactability trials, to allow some extra bitumen in the mixture for initial rubber-bitumen reaction. Furthermore, the mixtures were produced in different short-term conditioning regimes to reflect the effect of the rubber-bitumen interaction during the production period (1 to 6 hours) and were also compacted to achieve two target void contents to simulate good and poor compaction. In addition, control mixtures were produced under the same conditioning regimes with the same binder and compacted to achieve the same voids content as the CRM mixtures. The mass and volumetric compositions of the control and CRM mixtures are presented in Figures 7.1 to 7.3 and detail calculations are included in Appendix D. It can be seen that the volumetric proportions of the aggregate and bitumen decrease with increasing rubber content and consequently, the densities of the mixtures will be less due to the lower density of the rubber particles. One would expect that the stiffness modulus of the mixture would also be reduced as the mineral aggregate skeleton becomes more affected by the highly elastic rubber particles.

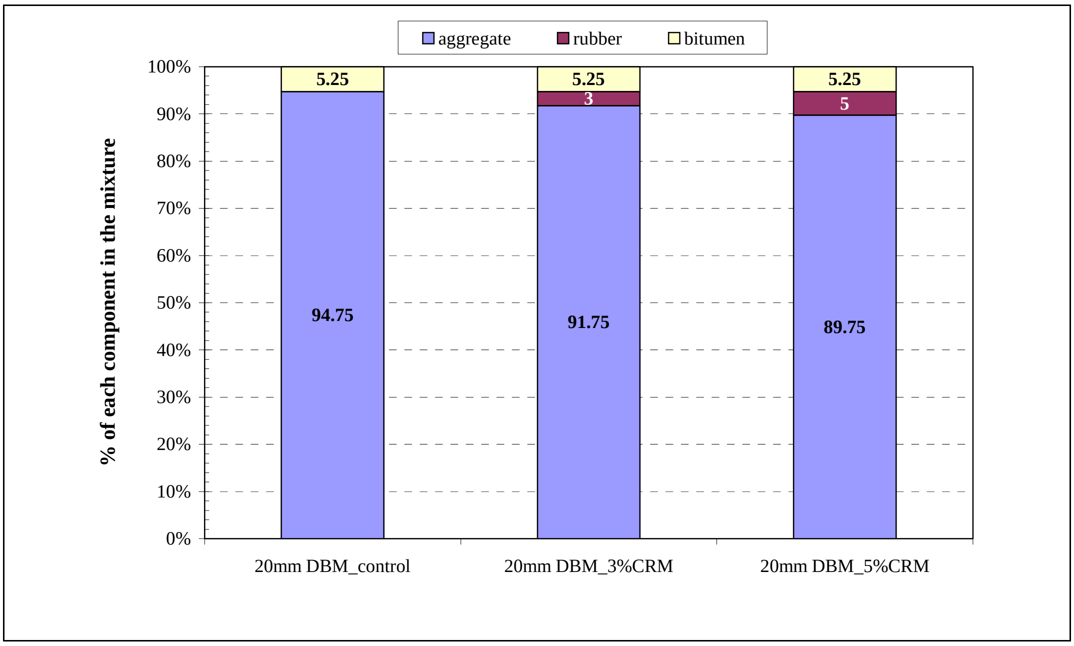


Figure 7.1: Percentage of each component by mass in the 20 mm DBM mixture

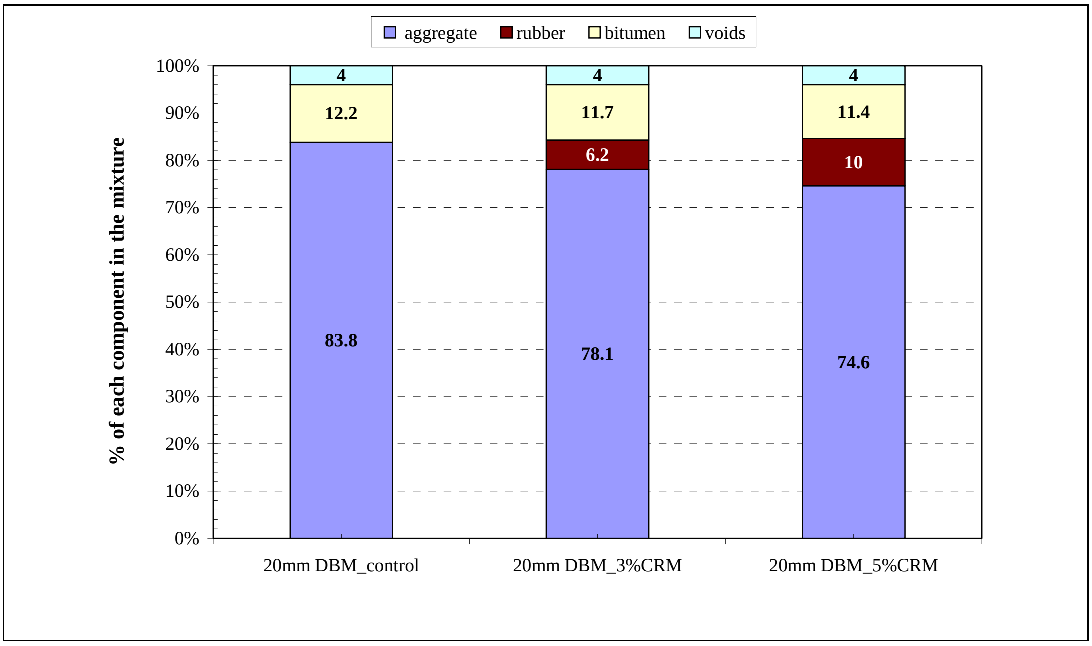


Figure 7.2: Volumetric proportion of the mixtures with designed void content of 4%

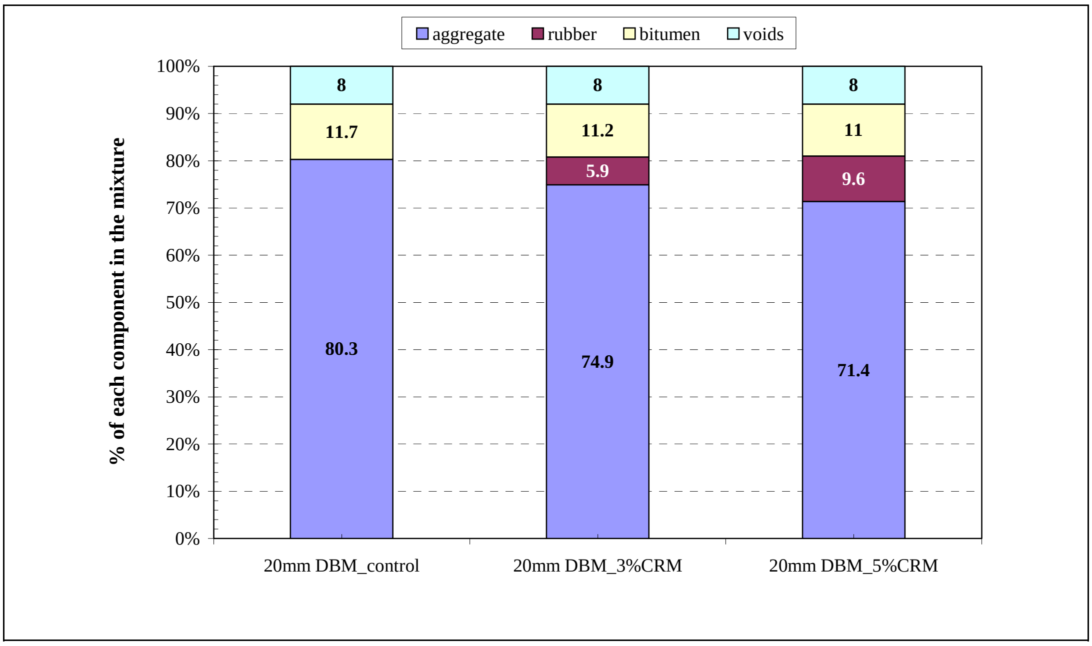


Figure 7.3: Volumetric proportion of the mixtures with designed void content of 8%

As the grading curve was modified to replace 3% and 5% aggregate by mass with crumb rubber, the grading of rubber particles was also taken into consideration when designing the mixture. Although the size of the crumb rubber varied from 2 to 8 mm, two single size fractions were chosen to ensure consistency and simplicity of the mixture design. Crumb rubber was divided into the following two fractions: passing 6.3 mm and retained on 3.35 mm, and passing 3.35 mm and retained on 0.3 mm. These two fractions were then substituted into the design DBM gradation. As the majority of the granulated crumb rubber was less than 3.35 mm, the two fractions were not replaced in equal amounts but consisting of 20% < 6.3 mm and > 3.35 mm and 80% < 3.35 mm and > 0.3 mm. The mixture design and grading curves for the 20mm DBM mixtures are shown in Table 7.2 and Figure 7.4.

Table 7.2: Material gradation (individual percentage retained) of control and CRM mixtures for 20 mm DBM binder course

Sieve size	**20DBM_Control**	**20DBM_3%CRM**	**20DBM_5%CRM**
20	2.5	2.5	2.5
14	22.5	22.5	22.5
10	13	13	13
6.3	15	14.4	14
6.3(20% CR)	**-**	**0.6**	**1**
3.35	8	5.6	4
3.35(80% CR)	**-**	**2.4**	**4**
0.3	25	25	25
0.075	8.5	8.5	8.5
Pan	5.5	5.5	5.5

It is important to note that the CRM asphalt mixtures were batched gravimetrically and the gravimetric gradings were then converted to volumetric gradings to check that they were still within the grading envelopes of the 20 mm DBM asphalt mixture.

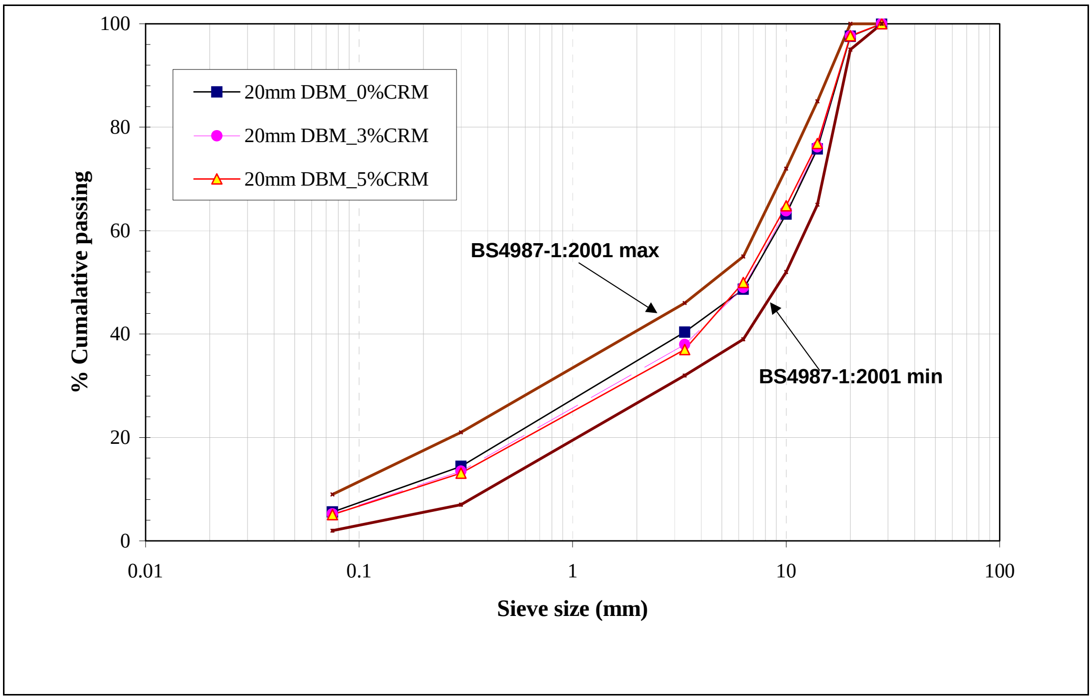


Figure 7.4: Modified BS 4987-1:2001 grading for 20 mm DBM control and CRM mixtures

7.2.3 Mixture Preparation

The required quantities of aggregate and bitumen for mixing were heated at 160^{0}C for a maximum four hours prior to mixing to distribute heat uniformly, whilst the rubber particles were kept at room temperature and added directly into the mixer. The aggregate and rubber were then mixed in a Sun-and-Planet (Figure 7.5) mixer at 160^{0}C for approximately two minutes. Heated bitumen was then added to the mixture and the mixing continued for a further five minutes. Immediately after each batch was mixed, it was placed in a preheated gyratory mould (Figure 7.6) and subjected to gyratory compaction. As rubber absorbs bitumen at high temperatures, the prime objective was to ensure consistency during production by mixing all samples in a fixed time sequence so that temperature and pre-compaction ageing duration remained the same. In addition, the mixtures were subjected to short term ageing (2 and 6 hours) in their loose state at the mixing temperature to investigate the effect of rubber-bitumen interaction on the mechanical properties. The conditioning procedure consisted of placing the un-compacted, mixed material in shallow trays in a forced draft oven at 155^{0}C for 2 and 6 hours to simulate transportation and laying periods.

Figure 7.5: Sun-and-Planet asphalt mixer

Figure 7.6: Gyratory mould

7.2.4 Compaction

The gyratory compactor produces asphalt mixture specimens to densities achieved under actual pavement loading conditions. A gyratory compactor consists of a rigid reaction frame, loading system, specimen height measurement and computer controlled interface unit (Figure 7.7). In this method, a split mould is mounted in the cage, clamped at both ends and rotated on an axis eccentric to the vertical with an angle of 1.26^0, a static compressive vertical load, controlled by a pneumatic actuator, is applied to the material through parallel end plates. The actual load can be maintained at the required level through a voltage/pressure (V/P) converter regulating the pressure of the supplied air. This action generates horizontal shear stresses within the material and reorients the aggregates. The height of the specimen is monitored during the compaction process using a deformation transducer and the applied load is measured using a load cell. Thus, the specimen density and number of gyrations are recorded during compaction for compactability analysis.

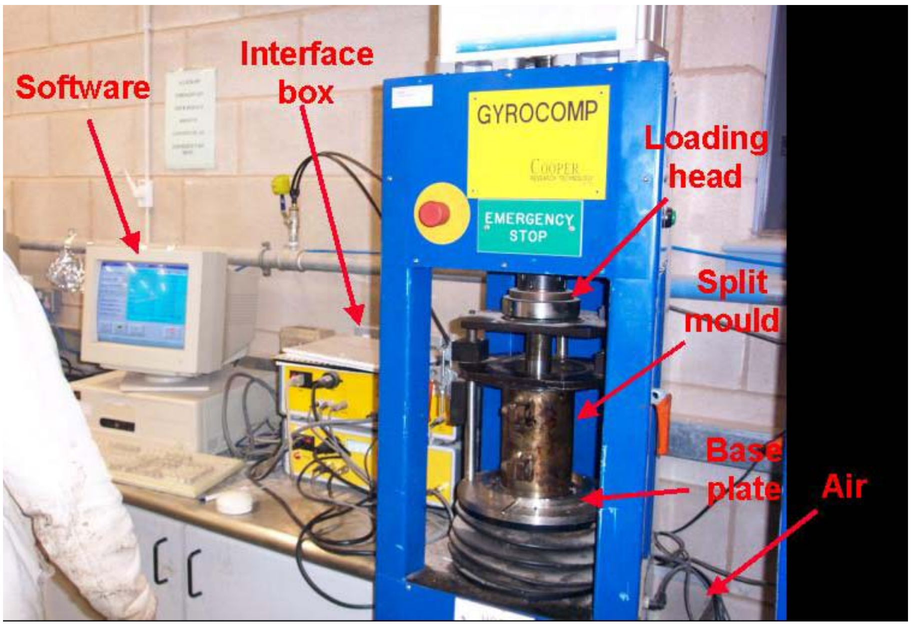


Figure 7.7: Gyratory compactor

The compaction of the mixtures was performed by applying a static vertical stress of 600kPa with 1.25^0 gyratory angle at a rate of 30 gyrations per minute. The compacted sample height was targeted at 90mm with a diameter of 100mm. One of the main problems observed during the trial compaction of CRM mixtures was that the compacted mixture bounces back during the curing period. The sample height was always 4-5mm higher than the target height, which resulted in inconsistent densities. This problem was predominantly higher for the mixtures with higher rubber content. To overcome this problem, several trial samples were prepared with different durations of compaction and different mixture densities. Finally, it was found that extra compaction effort, at least 600 gyrations, was required to minimise the rubber rebounding effect. All specimens (control and CRM) were compacted to a design density (target air voids content) or 600 (maximum number) gyrations whichever occurred first. In addition, after compaction the specimens were kept inside the mould for a further 24 hours at room temperature. This was important to allow the bitumen to gain sufficient stiffness to hold the rubber particles and ensures homogeneity of the specimen. Finally, the samples were removed from the mould and were trimmed from both sides using a saw to achieve 60 mm specimen thickness suitable for NAT testing. Figure 7.8 shows a picture of a compacted specimen and Figure 7.9 shows a trimmed specimen for NAT testing.

It is important to note that the distribution of lower density rubber particles is not uniform in the mixtures and they form clusters as shown in Figure 7.10. These clusters tend to fall out from the edge during trimming, during high temperature conditioning prior to permanent deformation testing (Chapter 8), and also during moisture conditioning (Chapter 9). This material dislodging will have an influence on consistent mechanical properties such as stiffness, fatigue and resistance to permanent deformation.

(A) (B)

Figure 7.8: Compacted specimen, A = CRM specimen with 5% rubber by mass of total aggregate, B = conventional 20 mm DBM specimen (control)

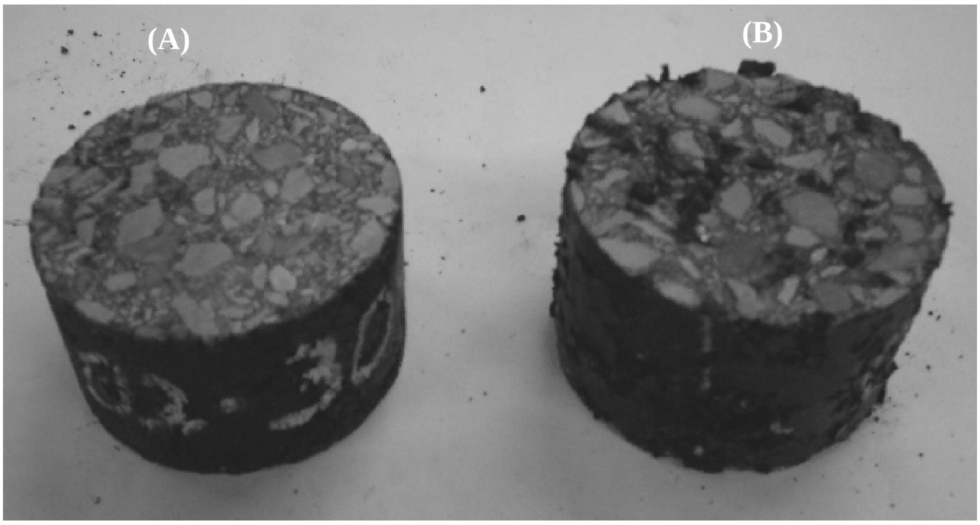


Figure 7.9: NAT specimen, A = Control, B = CRM sample with 3% rubber by mass of total aggregate

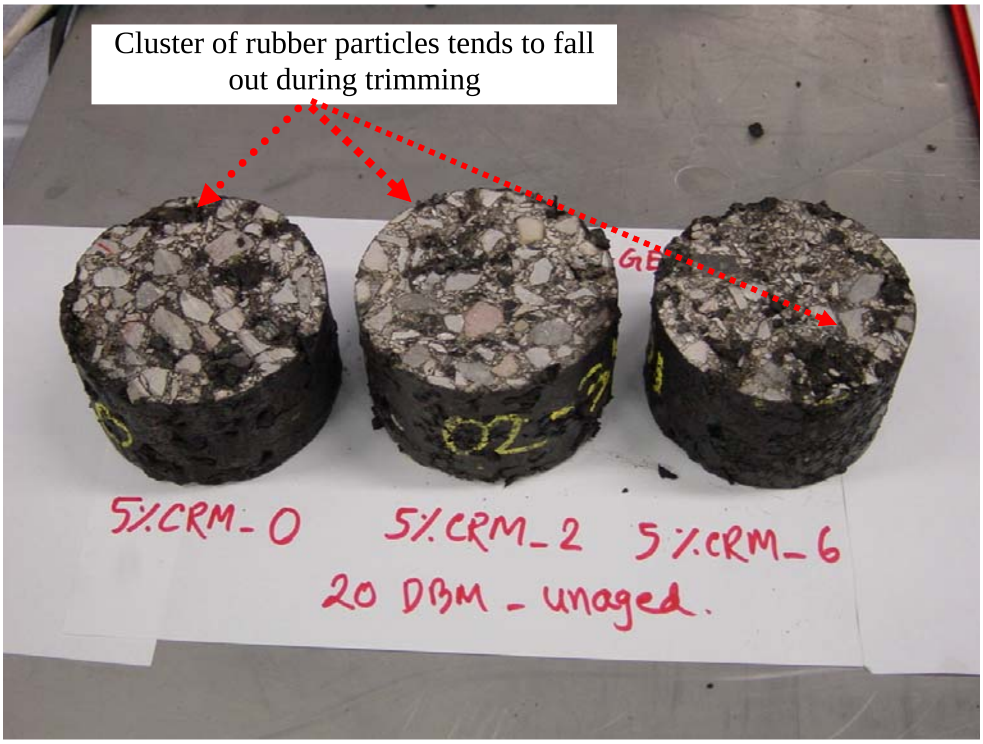


Figure 7.10: Specimens with 5% rubber content and subjected to 0, 2 and 6 hours short-term conditioning prior to compaction

7.2.5 Patented Dry Process Mixtures versus CRM Mixtures

A literature review on dry process CRM asphalt mixtures revealed that the primary design property is the percentage of air voids instead of stability. The target air void content for the CRM mixture should be 2-4% (Heitzman, 1992, Epps, 1994, FHWA, 1997). In addition, CRM mixtures should be designed volumetrically to compensate for the lower specific gravity of the crumb rubber particles.

As described in the literature review, there are two widely used methods of dry process CRM asphalt mixtures, namely, PlusRide® and Generic. A comparative study of the design considerations of both methods and the method used in this project are listed in Table 7.4. One significant difference is that the CRM mixture used in this project is suitable for a binder course whereas the other two mixtures are mainly for surfacing applications. In addition, the CRM mixtures were designed to accommodate a larger percentage of rubber content and the particle size is larger than both the PlusRide® and Generic Dry Processes. The bitumen content is also lower for the CRM mixtures compared to other dry process mixtures. However, all the mixtures use similar mixing temperatures, long periods of compaction and only the generic dry process uses a catalyst to allow a faster reaction between the rubber particles and bitumen.

Table 7.3: Comparative study of different dry process CRM asphalt mixtures

Specification		**Plus Ride**	**Generic**	**CRM Project**
Mixture type		Gap graded	Dense graded	Dense graded macadam
Mixture Volumetric		To compensate low density rubber	To compensate low density rubber	To compensate low density rubber
Applications		Surface course	Surface course	Binder course
Rubber content		1-3% by weight of total mixture	<2% by weight of total mixtures	3% & 5% by mass of total aggregate content
Bitumen content		7.5 to 9% by mass	7% by mass	5.25% by mass
Bitumen grade		Same as conventional	Same as conventional	100/150Pen
Target voids		2-4%	3%	4% & 8%
Rubber size		4.2 mm to 2.0 mm	Combination of larger (4.75 mm) and finer particles (>1 mm)	2.0 mm to 8.0 mm
Gradation		Patented gradation	Generic gradation	Adjusted to BS4987
Mixing temperature		149-177^{0}C	149-177^{0}C	155-160^{0}C
Rubber pre-treatment		No pre-treatment	Rubber pre-treated with catalyst	No pre-treatment
Compaction	*Method*	Similar to conventional hot mix asphalt	Similar to conventional hot mix asphalt	Lab compaction similar to traditional hot mix asphalt mixture using gyratory compactor
	Lay down Temperature	> 120^{0}C in the field	129^{0}C in the field	150-160^{0}C compaction temperature in laboratory
	Conditions	< 60^{0}C to maintain mat density until the binder cools and gains strength to counteract the expansion tendencies of the compressed CRM	< 60^{0}C to maintain mat density until the binder cools and gains strength to counteract the expansion tendencies of the compressed CRM	Target density. Specimens were kept inside the mould for a further 24 hours for post compaction curing.

7.3 VOLUMETRIC PROPORTIONS

The determination of specimen bulk density was carried out in accordance with BS 598 part 104: 1989 (although aluminium foil, as opposed to wax, was used to seal the specimens). The maximum density, compacted mixture density and porosity were calculated using the following procedure.

- The maximum density of each mixture was obtained by performing Rice density measurement according to BS DD 228:1996.

$$S_{\max} = \frac{A * 997.1}{\{A - (B - C)\}} \qquad \textbf{(7.1)}$$

Where;

S_{max}	= Maximum density (g/cm^3)
A	= Mass of dry test portion in air (g)
B	= Mass of Rice pot, sample and water (g)
C	= Mass of Rice Pot Filled with water at 25^{0}C (g)

- Compacted mixture density (D)

$$D = \frac{S_s * W_1}{S_s(W_2 - W_4) - (W_2 - W_1)} \qquad \textbf{(7.2)}$$

Where;

D	= Density of compacted specimen (g/cm^3)
W_1	= Mass of uncoated specimen in air (g)
W_2	= Mass of coated (aluminium foil) specimen in air (g)
W_4	= Mass of coated specimen in water (g)
S_s	= Relative density of foil =1.65

- Calculation of air voids of compacted sample

$$V_v = \frac{100(S_{max} - D)}{S_{max}} \qquad \textbf{\textit{(7.3)}}$$

Where;

V_v = Void content (%)

S_{max} = maximum density of the mixture (g/cm^3)

D = Density of compacted specimen (g/cm^3)

Using the above equations, the maximum density and bulk densities of the mixtures with 0% rubber (control), 3% and 5% crumb rubber by mass of total aggregate are presented in Table 7.4. It can be seen that the density of the mixtures reduced as a result of increasing rubber content as well as voids contents.

Table 7.4: Mixture density

Mixture types	**Maximum density (kg/m^3)**	**Bulk density of compacted sample (kg/m^3)**	
		4% voids	8% voids
20mm DBM_ control	2473	2374	2275
20mm DBM_3%CRM	2383	2288	2192
20mm DBM_5%CRM	2332	2239	2145

7.4 SUMMARY

In this chapter a brief description of the mixture design procedure and compaction technique of CRM samples produced by modifying BS 4987 for the 20 mm DBM mixture are presented. Specimens were produced in three short-term age conditioning regimes 0, 2 and 6 hours with two target air void contents of 4% and 8%. In the next chapter a detailed investigation of the fundamental mechanical properties of the primary and secondary aggregate asphalt mixtures using the NAT will be presented. Three main performance indicators will be assessed, namely stiffness modulus to investigate load bearing capacity and the two main pavement distress mechanisms in terms of fatigue cracking and permanent deformation.

CHAPTER 8

Asphalt Mixture Mechanical Properties

8.1 INTRODUCTION

This chapter describes the investigation of the effects on the mechanical properties of asphalt mixtures due to the incorporation of recycled crumb rubber as partial replacement of aggregate. An assumption that is generally made with the use of the dry process is that the rubber crumb is solely part of the aggregate and that the reaction between bitumen and crumb rubber is negligible. This is usually achieved by limiting the time at which the bitumen and crumb rubber are maintained at high mixing (reaction) temperatures and specifying a coarse granulated crumb rubber with a low surface area and smooth (less reactive) sheared surfaces. However, swelling test results (Chapter 4) and recent research on rubber-bitumen (Singleton *et al,* 2000, Airey *et al*, 2002) have shown that during the mixing period as well as during transportation and laying, rubber crumb does swell and react with bitumen changing the properties of the residual bitumen, the shape and rigidity of the rubber and consequently the performance of the asphalt mixture. To investigate the effect of this rubber-bitumen reaction, the asphalt mixtures were subjected to short-term ageing (conditioning) prior to compaction of the material. The conditioning procedure consisted of placing the uncompacted, mixed asphalt material in shallow trays in a forced draft oven at 155°C for 2 and 6 hours.

Another concern with the dry process is the difficulty in achieving field compaction of CRM material that is inherently resilient under loading. The effect of compaction on mechanical performance was investigated by subjecting half of the asphalt mixtures to a lower degree of compaction than would normally be prescribed in a laboratory environment. A total of sixteen control and CRM asphalt mixture combinations were therefore produced in the laboratory with the following variables:

- Crumb rubber content by mass of total aggregate: 0, 3 and 5%,
- Short-term age conditioning: 0, 2 and 6 hours at 155°C,
- Compaction effort (air void content): 4 and 8%.

Sixteen combinations were produced with the control mixtures being produced at two short-term age conditions (0 and 6 hours) whereas three short-term conditions (0, 2 and 6 hours) were used for the production of the CRM mixtures as greater bitumen modification was expected through rubber-bitumen interaction.

The different materials were coded as follows; crumb rubber content as R0, R3 and R5, short-term conditioning as C0, C2 and C6, and air void content (compaction effort) as L for low air void content of 4% and H for high air void content of 8%. For example, R3-C6-H refers to the CRM asphalt mixture with 3% rubber content by mass, short-term conditioned for 6 hours with an air voids content of 8%.

The mechanical properties, such as stiffness modulus, fatigue and resistance to permanent deformation of the asphalt mixtures were evaluated using the NAT. A brief description of NAT testing procedures is presented in Section 2.3.

Stiffness modulus, as an indicator of the load spreading ability of the mixtures and routinely used as an input for elastic layer analysis in mechanistic pavement design, was investigated for all the CRM mixtures subjected to the three short-term age conditioning regimes and two compaction efforts. On the other hand, the two main performance indicators, fatigue and resistance to permanent deformation were only investigated for mixtures produced at 0 and 6 hours short-term age conditioning. All the test results were compared with the control mixtures produced at similar conditioning regimes and compaction efforts.

In the first section of the chapter, the results obtained from the stiffness, fatigue and permanent deformation tests on the control (conventional DBM) and CRM mixtures are presented. The results are analysed in terms of the effect of rubber content, short-term ageing and compaction effort. In the final section, the overall findings are summarised.

8.2 STIFFNESS MODULUS

8.2.1 ITSM Testing Protocol

The stiffness moduli of the primary and secondary asphalt mixtures were measured using the Indirect Tensile Stiffness Modulus (ITSM) test on 100 mm diameter specimens with an average height of 60 mm. The test was performed in accordance with the British Standard DD213 using the following test parameters:

- Test temperature: 20°C,
- Loading rise-time: 124 milliseconds, and
- Peak transient horizontal deformation: 5 μm.

Each sample was tested twice by rotating $90^0 \pm 10^0$ about its horizontal axis with results within 10% of each other being used for the analysis. During the test a load pulse is applied along the vertical diameter of a cylindrical specimen and the resultant peak transient deformation measured along the horizontal diameter. The stiffness modulus is then a function of load, deformation, specimen dimensions and an assumed Poisson's ratio of 0.35 and calculated using equation 2.20 (Chapter 2).

8.2.2 ITSM Test Results

The volumetric proportions and stiffness modulus results for the control and CRM asphalt mixtures are presented in Tables 8.1 and 8.2. The mean values together with their minimum, maximum and standard deviation have been quoted for each asphalt mixture with target voids of 4% and 8% respectively. The results are compared with the control mixtures with the same voids range. The stiffness results for the individual samples for all the mixtures are included in Appendix E.

It can be seen from the table that there is a wide variation between minimum and maximum stiffness for all the mixtures. Although it has been possible to produce CRM mixtures at the two target air void contents (4 and 8%) in terms of their average results, difficulties associated with compacting the CRM mixtures, particularly the higher crumb rubber content (5%) mixtures, has meant that there has been a considerable amount of scatter in terms of both air void content and stiffness. This scatter is shown in Figure 8.1 where the specimens produced at high and low levels of compaction have been grouped together for each combination of crumb rubber content and short-term age conditioning with specimens below 6% being classified as low void content and above 6% as high.

In addition, after ITSM testing on all samples, two thirds of the total samples from each mixture were stored at 5^0C for durability testing as presented in Chapter 9. The remaining one third was divided in two parts for ITFT and CRLAT testing in their unconditioned state and the results are presented in Sections 8.3 and 8.4 of this chapter.

Table 8.1: Volumetric and stiffness results for highly compacted (4% target voids) control and CRM mixtures

Mixture	No of Sample	Air void content (%)				Stiffness (MPa)			
		avg	max	min	std	avg	max	min	std
R0_C0_L	27	**3.3**	5.4	2.3	0.8	**3642**	4439	2586	542
R0_C6_L	30	**3.1**	5.5	2.1	0.8	**4937**	6523	3627	827
R3_C0_L	32	**4.0**	5.9	2.1	1.2	**2708**	3733	2117	308
R3_C2_L	33	**4.1**	6.0	2.0	1.3	**2863**	3384	2147	319
R3_C6_L	36	**4.5**	6.0	2.7	1.0	**3703**	4664	2873	470
R5_C0_L	25	**4.0**	6.0	1.5	1.4	**2032**	3430	1442	423
R5_C2_L	23	**3.6**	5.8	2.1	1.2	**2540**	3867	1632	584
R5_C6_L	26	**3.8**	5.8	2.2	1.0	**3393**	4963	2487	618

Table 8.2: Volumetric and stiffness results for poorly compacted (8% target voids) control and CRM mixtures

Mixture	Sample No	Air void content (%)				Stiffness (MPa)			
		avg	max	min	std	avg	max	min	std
R0_C0_H	31	**7.7**	9.9	6.0	0.9	**2245**	2768	1853	231
R0_C6_H	30	**7.8**	9.5	6.1	0.9	**3928**	4582	3016	384
R3_C0_H	32	**7.5**	10.3	6.0	1.1	**2112**	2500	1576	256
R3_C2_H	26	**8.0**	10.0	6.2	1.1	**2401**	3261	1750	365
R3_C6_H	27	**7.9**	10.3	6.0	1.3	**2899**	3740	2456	344
R5_C0_H	29	**8.4**	10.0	6.1	1.3	**1502**	1939	1036	218
R5_C2_H	21	**8.2**	10.4	6.1	1.5	**1723**	2200	1140	248
R5_C6_H	27	**8.0**	10.4	6.3	1.2	**2297**	3110	1777	320

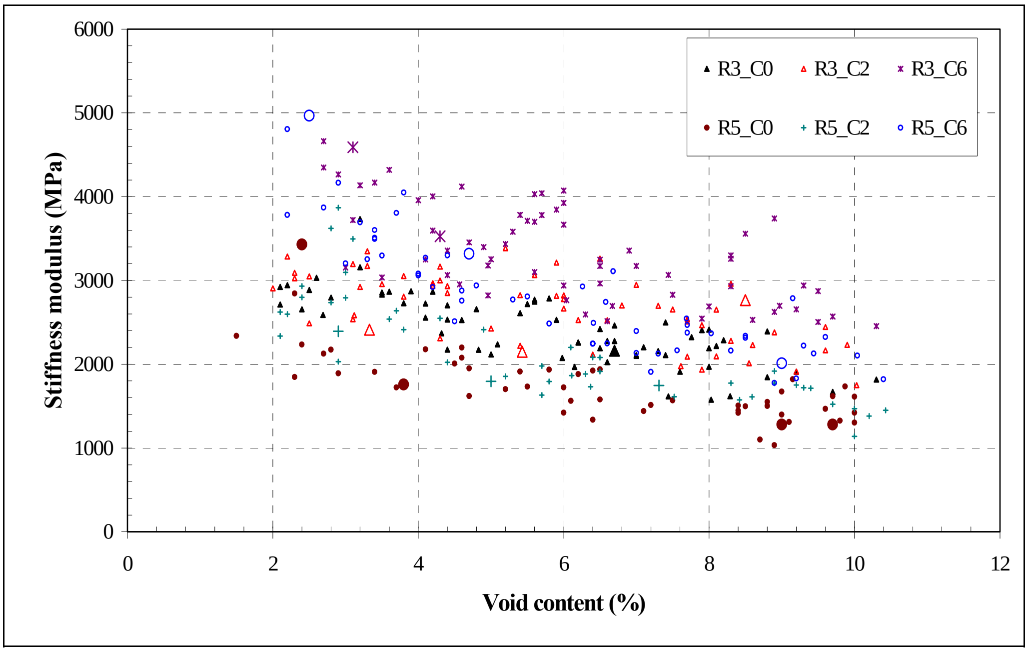


Figure 8.1: Stiffness versus void contents of CRM mixtures

8.2.3 Rubber Contents

The effect of crumb rubber modification on the stiffness of the DBM asphalt mixtures can be seen in Figure 8.2. The graphs are plotted in terms of average stiffness for the different mixtures. In general, for all four combinations of high and low void contents and 0 and 6 hours age conditioning, the replacement of aggregate sizes between 2 and 8 mm with crumb rubber leads to a reduction in the load spreading ability (stiffness) of the asphalt mixture although a general increase in stiffness modulus was observed in all mixtures due to short-term pre-compaction conditioning.

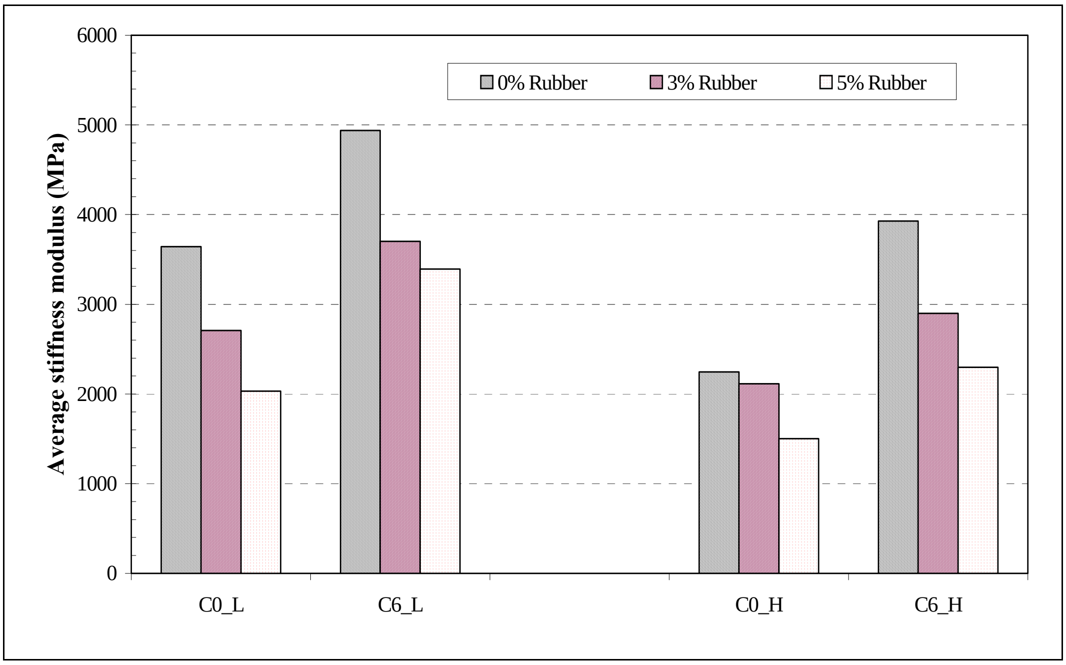


Figure 8.2: Average stiffness modulus for high and low compacted control and CRM mixtures

Table 8.3 presents the percentage reduction of stiffness modulus for CRM mixtures compared to their corresponding control mixtures. The results show that the reduction of stiffness for the highly compacted R3-C0 mixture is around 26% whereas the reduction is as high as 44% for the R5-C0 mixture.

On the other hand, the stiffness reduction for poorly compacted R3-C0 mixtures is only 6% whereas the stiffness reduced up to 26% for R5-C0 mixtures. The short-term ageing on the other hand does not appear to have much influence on the stiffness reduction because apart from the R3-C0-H, all the other CRM mixtures are showing a reduction in stiffness in the range of 25% to 44% depending on compaction effort and the amount of rubber content in the mixtures.

Table 8.3: Percentage of stiffness reduction as a function of rubber content compared with the control mixtures

Mixture	**Percent of stiffness reduction compared to control mixtures**	
	3% CRM	**5%CRM**
C0-L	26	44
C6-L	25	31
C0-H	6	26
C6-H	33	42

8.2.4 Short-Term Ageing

In terms of the effect of short-term age conditioning on stiffness, stiffness values of individual samples of R3 mixtures are plotted in Figure 8.3. The results demonstrate that irrespective of compaction effort, stiffness increases due to short-term conditioning of the loose mixtures. The average stiffness values for all highly and poorly compacted mixtures are presented in Table 8.4. The percentage changes in stiffness with short-term ageing are also included in Table 8.4 and plotted in Figure 8.4. It can be seen that the stiffness increased 36%, 37% and 67% for highly compacted R0-L, R3-L and R5-L mixtures due to conditioning for six hours prior to compaction. Irrespective of the amount of rubber content in the mixtures, similar increases in stiffness can also be seen for the poorly compacted CRM mixtures.

Comparing R3 to R5 mixtures, Figure 8.4 shows that irrespective of compaction effort, short-term ageing has more effect on R5 mixtures results with greater increases in stiffness for 0 to 2 hours and 0 to 6 hours conditioning regimes. The results from the 2 hours conditioned mixtures show that average stiffness values increased by 6% for R3-L and 25% for R5-L mixture whereas similar increases in stiffness, around 15%, can be seen for both poorly compacted CRM mixtures. On the other hand, the increase in stiffness for the 6 hours conditioned mixtures is more than 50% (52% for poorly compacted mixtures and 67% for highly compacted mixtures) for the 5% CRM mixtures indicating more bitumen stiffening is happening compared to the 3% CRM mixtures, where 37% increase in stiffness was observed in both high and low compaction states.

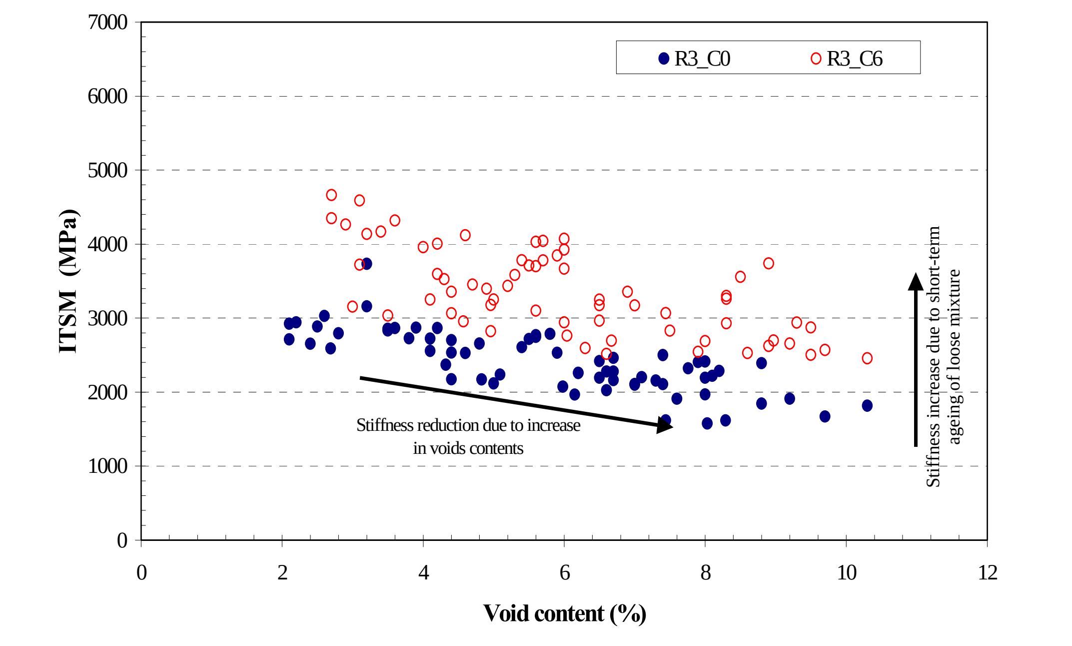


Figure 8.3: Increase in ITSM values of 3%CRM mixture with respect to air voids following short time oven ageing of the loose mixtures

The increase in stiffness is probably due to extra bitumen absorption with increasing rubber content in the mixtures as presented in Chapters 4 and 5 where the extra binder stiffening effect can be as much as 20 times greater than that normally found by simply ageing the binder. However, these rubber-bitumen curing tests were performed at far higher binder to rubber ratios than that experienced in normal asphalt mixtures and ignored the softening effect that the reacted rubber would have on the overall mixture stiffness.

Table 8.4: Stiffness modulus results as a function of short-term conditioning for control and CRM asphalt mixtures.

Mixture	Average stiffness modulus (MPa)			Percentage change (%)	
	0 hours age conditioning	2 hours age conditioning	6 hours age conditioning	0-2	0-6
R0-L	3642	not tested	4937	-	36
R0-H	2245	not tested	3928	-	75
R3-L	2708	2863	3703	6	37
R3-H	2112	2401	2899	14	37
R5-L	2032	2540	3393	25	67
R5-H	1502	1723	2297	15	52

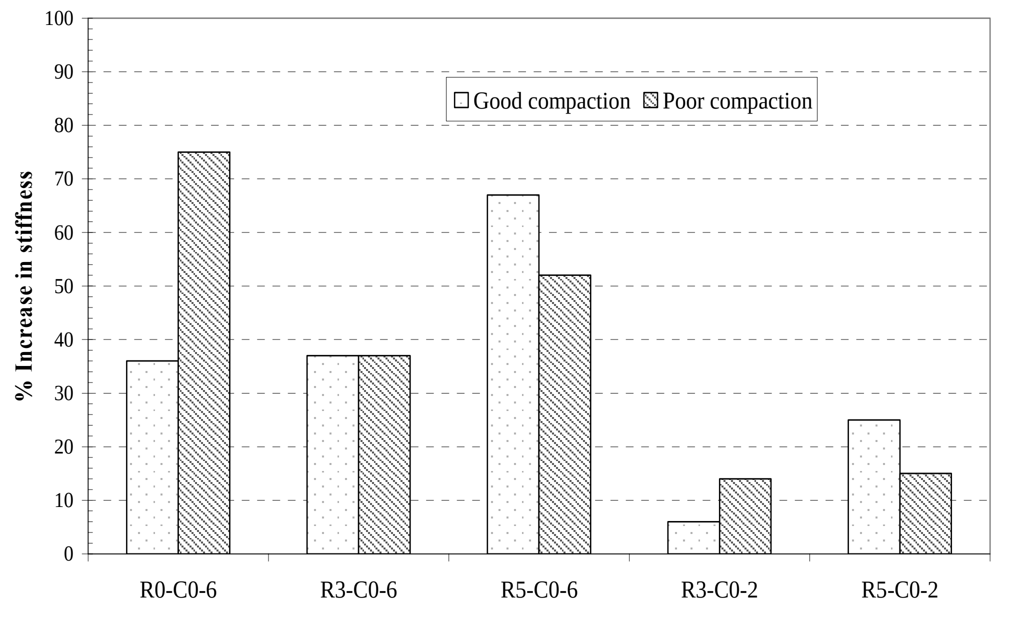


Figure 8.4: Percent change in stiffness due to short-time ageing

8.2.5 Compaction Effort

The stiffness results for the high compaction (low void content: 4%) and low compaction (high void content: 8%) mixtures are presented in Table 8.5 including the average void contents of each mixture. It can be seen that there is a fairly uniform decrease in stiffness for all the asphalt mixtures (control and CRM) with increasing air void content. The results also indicate that although the voids profile for the CRM mixtures were considerably scattered (Figure 8.1) and the range for high compaction and low compaction was chosen within ±2% of the target voids, the relative influence of average compaction effort (6% voids) might not significantly alter the stiffness of the mixture. Therefore, in terms of asphalt mixture stiffness modulus, the two issues of short-term curing (conditioning) and under compaction (high air void content) do not appear to have any more significance for the CRM mixtures than they have for the control mixtures.

Table 8.5: Stiffness modulus results of control and CRM asphalt mixtures

Mixture	Compaction effort				Percentage stiffness decrease (%)
	High compaction		Low compaction		
	Avg. Void contents (%)	Avg. stiffness (MPa)	Avg. void contents (%)	Avg. stiffness (MPa)	
R0-C0	3.3	3642	7.7	2245	38
R0-C6	3.1	4937	7.8	3928	20
R3-C0	4.0	2708	7.5	2112	22
R3-C2	4.1	2863	8.0	2401	16
R3-C6	4.5	3703	7.9	2899	22
R5-C0	4.0	2032	8.4	1502	26
R5-C2	3.6	2540	8.2	1723	32
R5-C6	3.8	3393	8.0	2297	32

8.3 FATIGUE PROPERTIES OF CRM ASPHALT MIXTURES

8.3.1 ITFT Testing Protocol

The fatigue resistance of the asphalt mixtures was determined by means of the Indirect Tensile Fatigue Test (ITFT) on all the C0 and C6 mixtures with an experimental arrangement similar to that used for the ITSM but under repeated loading and with slight modifications to the testing modulus crosshead. The ITFT tests were performed in accordance to DD ABF: 2002 using the following test parameters:

- Test temperature: 20°C,
- Loading condition: Controlled-stress,
- Loading rise-time: 120 milliseconds, and
- Failure condition: 9 mm vertical deformation

The results were interpreted in terms of initial tensile strain versus number of load repetitions to produce 9 mm vertical deformation. The tensile strain was calculated using Equation 2.26 in Chapter 2.

8.3.2 ITFT Test Results

Figures 8.5 to 8.8 present fatigue lives of different mixtures with design voids of 4% and 8%. The results are plotted in terms of initial tensile strain and number of cycles to failure. In addition, ITFT test data for all the mixtures are included in Appendix F.

The fatigue line equation for highly compacted mixtures (Figures 8.5 and 8.6) demonstrated that at high stress levels CRM mixtures could exhibit larger strains in both unconditioned and short-term aged conditioned states. In addition, at low strain levels all C0 and C6 CRM mixtures demonstrate improved fatigue lives at both low and high void contents compared to the control mixture. The results are similar for poorly compacted mixtures (Figures 8.7 and 8.8) although the overall fatigue life is reduced due to lower stiffness modulus and poor compaction.

However, it can be seen that in the region of tensile strain that is of interest to pavement engineers, 30 to 200 $\mu\varepsilon$ (Read, 1996), CRM mixtures generally show better fatigue life compared to the control irrespective of compaction effort. This is despite the fact that the mixture stiffness reduced due to the incorporation of crumb rubber, which would have an effect in the control stress mode of loading as lower stiffness generates higher tensile strains, consequently, reducing the fatigue life. The flexible rubber particles are believed to be the main contributory element to accommodate more load applications although higher tensile strains were generated at a particular stress level.

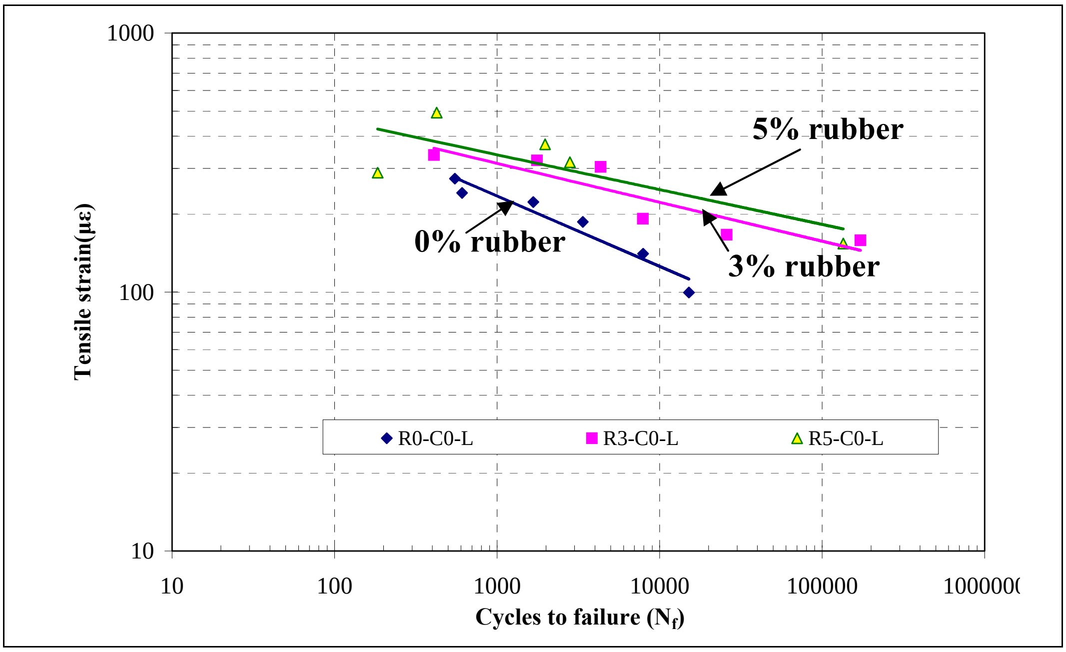


Figure 8.5: ITFT fatigue line for highly compacted mixtures conditioned in loose stage for 0 hr

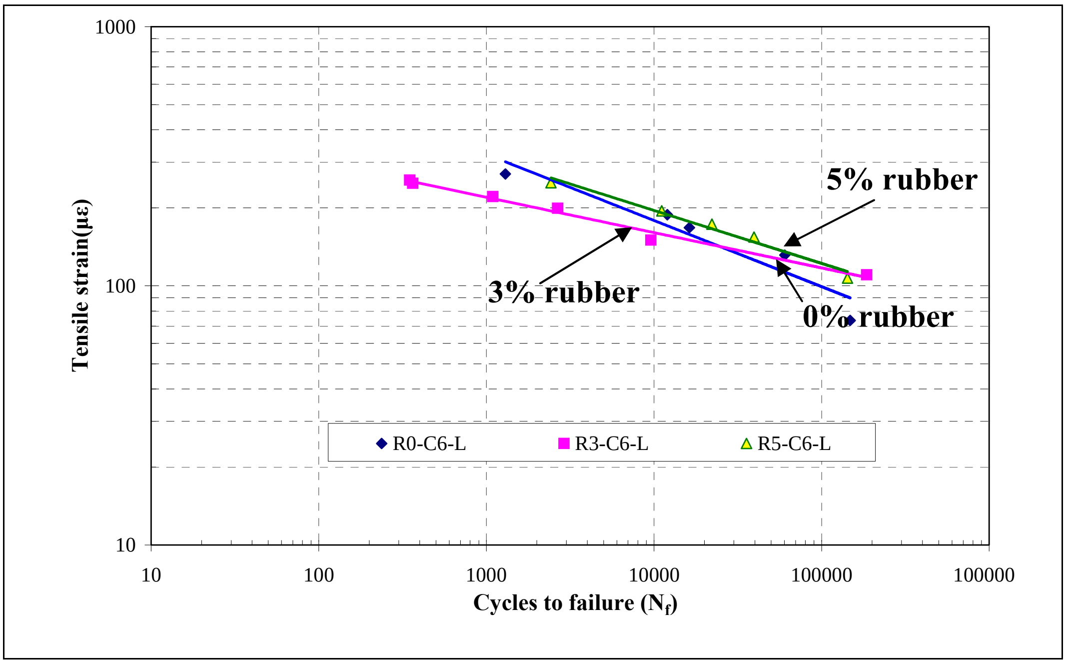


Figure 8.6: ITFT fatigue line for highly compacted mixtures conditioned in loose stage for 6 hours

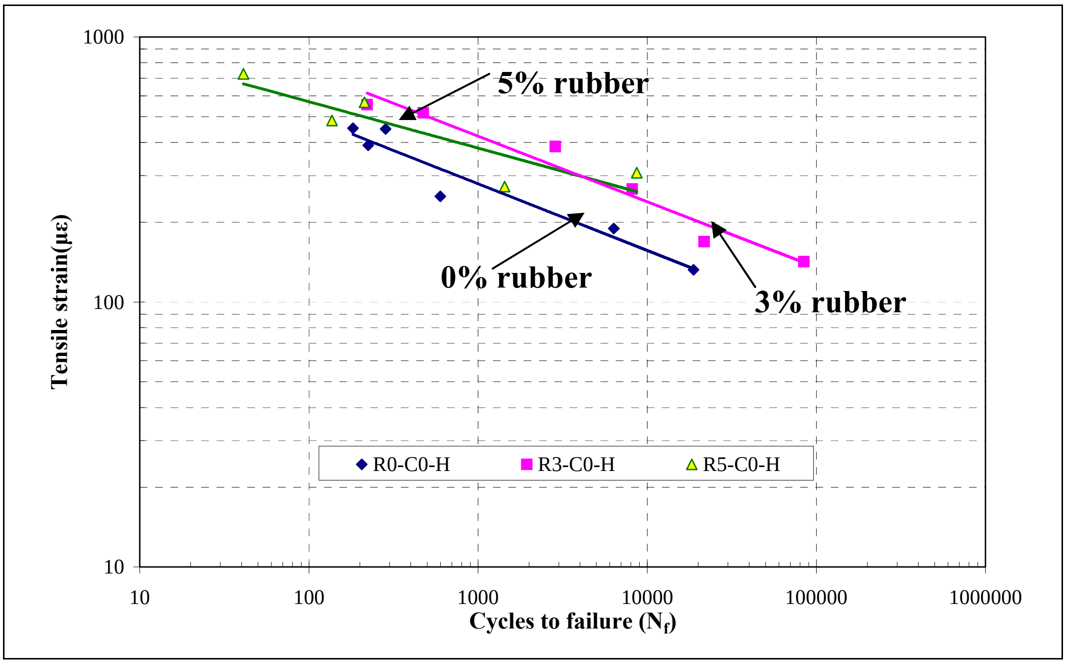


Figure 8.7: ITFT fatigue line for poorly compacted mixtures conditioned in loose stage for 0 hr

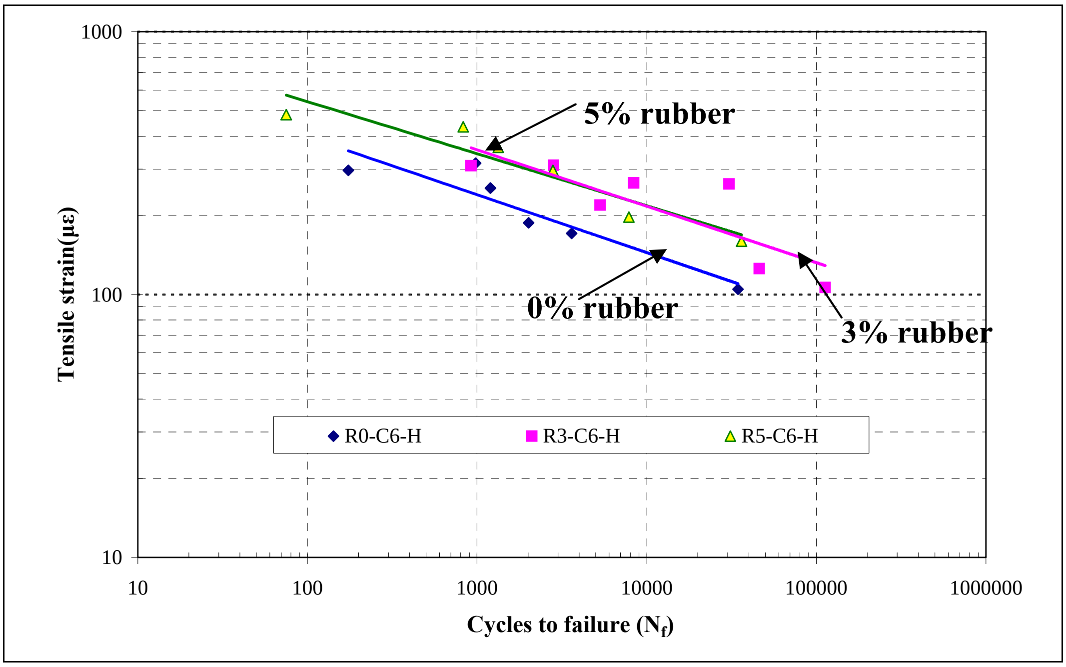


Figure 8.8: ITFT fatigue line for poorly compacted mixture conditioned in loose stage for 6 hours

To calculate the predicted strain and fatigue life, the fatigue regression line for each mixture is presented in Table 8.6. The equation of the regression line is based on the initial strain (ε) and the number of cycles to failure (N_f). The test data for the CRM mixtures have shown considerable scatter in some cases with slightly lower co-efficients of correlation. The scatter in the indirect tensile stiffness modulus results in this investigation may have contributed to the scatter for the initial strain calculations. With the exception of mixtures R5-C0-L and R3-C6-H, the fatigue and strain equations have all been established with a fairly high degree of confidence.

Using equations presented in Table 8.6, the predicted fatigue life for 100με and predicted strains for a million load cycles were calculated to study the effect of rubber content, short-term conditioning and compaction effort on fatigue performance. The predicted strain and fatigue life for highly and poorly compacted mixtures are presented in Figures 8.9 and 8.10. Figure 8.9 shows that irrespective of compaction effort, the predicted strain for a million cycles is generally higher in the CRM mixtures compared to their corresponding control mixtures. The number of load pulses to produce 100με is also higher for CRM mixtures at both void contents (Figure 8.10).

Table 8.6: Fatigue relationship for control and CRM asphalt mixtures

Mixture	**Fatigue equation**	**Strain equation**	**Fit (R^2)**
R0-C0-L	$N_f = 1.57 \times 10^{11}\ \varepsilon^{-3.447}$	$\varepsilon = 1534 \times N_f^{-0.272}$	0.94
R0-C6-L	$N_f = 1.02 \times 10^{12}\ \varepsilon^{-3.541}$	$\varepsilon = 1879 \times N_f^{-0.255}$	0.90
R3-C0-L	$N_f = 4.80 \times 10^{16}\ \varepsilon^{-5.416}$	$\varepsilon = 887 \times N_f^{-0.150}$	0.81
R3-C6-L	$N_f = 8.97 \times 10^{19}\ \varepsilon^{-7.237}$	$\varepsilon = 563 \times N_f^{-0.136}$	0.99
R5-C0-L	$N_f = 1.02 \times 10^{15}\ \varepsilon^{-4.694}$	$\varepsilon = 858 \times N_f^{-0.134}$	0.63
R5-C6-L	$N_f = 9.03 \times 10^{14}\ \varepsilon^{-4.779}$	$\varepsilon = 1284 \times N_f^{-0.204}$	0.98
R0-C0-H	$N_f = 1.06 \times 10^{12}\ \varepsilon^{-3.691}$	$\varepsilon = 1587 \times N_f^{-0.252}$	0.93
R0-C6-H	$N_f = 1.87 \times 10^{12}\ \varepsilon^{-3.879}$	$\varepsilon = 1096 \times N_f^{-0.220}$	0.85
R3-C0-H	$N_f = 1.58 \times 10^{13}\ \varepsilon^{-3.875}$	$\varepsilon = 2336 \times N_f^{-0.248}$	0.96
R3-C6-H	$N_f = 3.58 \times 10^{11}\ \varepsilon^{-3.229}$	$\varepsilon = 1564 \times N_f^{-0.215}$	0.69
R5-C0-H	$N_f = 5.45 \times 10^{14}\ \varepsilon^{-4.576}$	$\varepsilon = 1277 \times N_f^{-0.175}$	0.80
R5-C6-H	$N_f = 2.52 \times 10^{14}\ \varepsilon^{-4.483}$	$\varepsilon = 1349 \times N_f^{-0.198}$	0.89

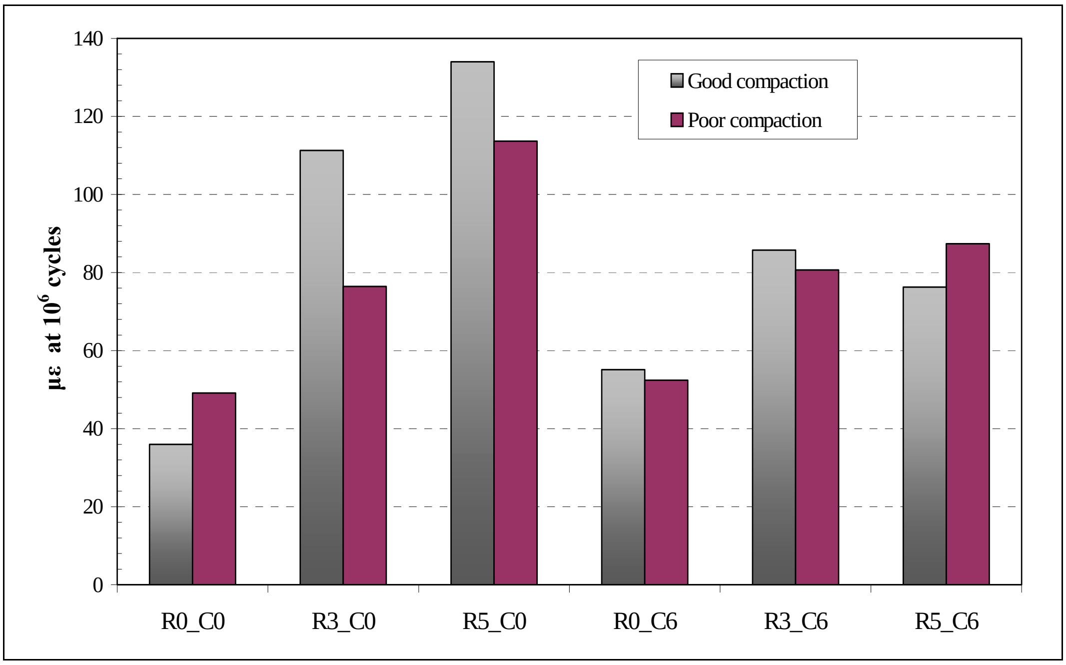


Figure 8.9: Comparison of strain for million cycles on control and CRM asphalt mixtures

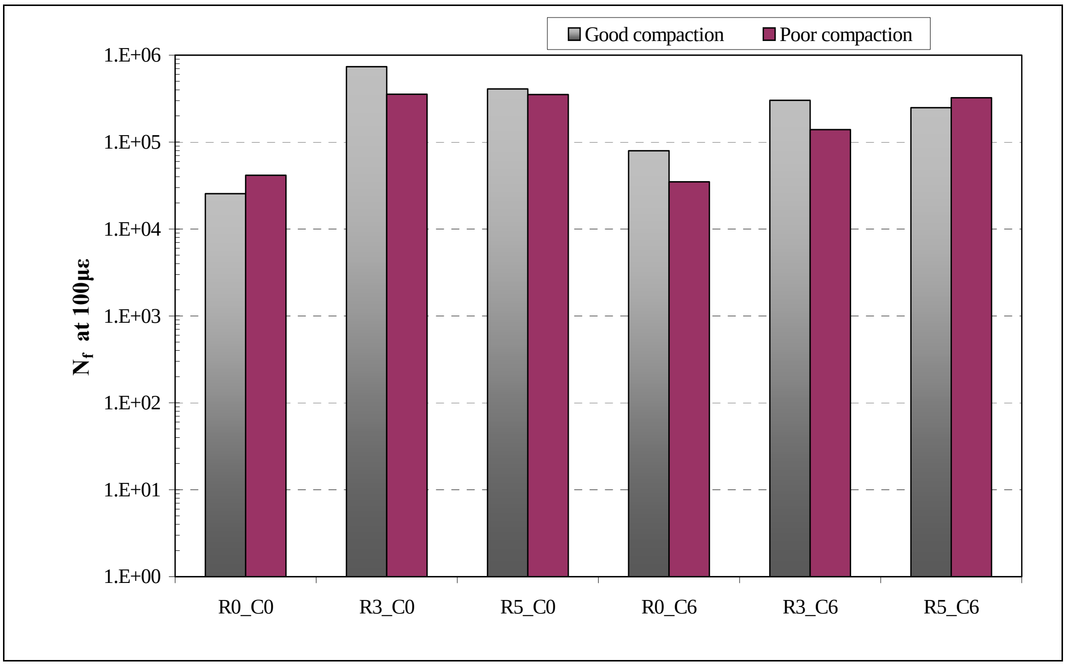


Figure 8.10: Comparison of number of cycles at 100$\mu\varepsilon$ on control and CRM mixtures

8.3.3 Rubber content

The relative fatigue performance of the CRM mixtures compared to the control mixtures are shown in Table 8.7 as both the predicted strain corresponding to a fatigue life of million cycles and the predicted number of fatigue cycles at a strain level of 100 microstrain (με). The results demonstrate that both R3-C0-L and R5-C0-L mixtures can accommodate approximately three times more initial strain compared to R0-C0-L mixtures to achieve a fatigue life of a million cycles. Better fatigue life was also observed in both short-term aged CRM mixtures compared to similarly aged control (R0-C6) mixtures with 56% more initial strain for 3% CRM and 38% for 5% CRM mixtures. In terms of predicted number of cycles to generate 100με, both highly and poorly compacted CRM mixtures can accommodate significantly higher numbers of load pulses compared to the control mixtures. For example, in the case of C0-L mixtures, the fatigue life for 3%CRM mixtures is approximately 35 times greater and 20 times greater for 5%CRM mixture than their corresponding control mixtures. Although the life is not as high for other CRM mixtures, it is still more than three times their corresponding control mixtures.

Comparing 3% CRM with 5% CRM mixtures, the results generally show that irrespective of short-term age conditioning, the highly compacted 3% CRM mixtures generate higher strains for unaged mixtures and can accommodate less load applications than 5% CRM mixture. In the poorly compacted case, 5% CRM mixtures can accommodate more load pulses although higher strains are generated in both age conditioning regimes as mixtures contain higher rubber contents. Therefore, the results suggest that both in terms of predicted strain, $\varepsilon 10^6$, and fatigue life, N_f^{100}, the partial replacement of aggregate with crumb rubber results in a significant increase in fatigue performance.

Table 8.7: Fatigue life comparison as a function of rubber content

Mixture	**Strain @ 10^6 cycles (με)**			**Cycles @ 100 με**		
	0% Rubber	**3% Rubber**	**5% Rubber**	**0% Rubber**	**3% Rubber**	**5% Rubber**
C0-L	36	111	134	20,040	705,734	416,486
C6-L	55	86	76	84,450	301,434	249,970
C0-H	49	76	114	44,066	281,357	384,589
C6-H	52	81	87	32,647	124,877	272,270

8.3.4 Short-Term Ageing

Table 8.8 presents the predicted strain and fatigue life, including percentage changes, as a function of the short-term age conditioning of the loose mixtures. Compared to the effect of rubber content, the short-term ageing of the asphalt mixtures does not appear to have a significant effect on the overall fatigue performance of the control and CRM mixtures, although in general ageing the material does appear to reduce its resistance to fatigue cracking. With the exception of R3-H, the percentage reduction in predicted strain for CRM mixtures is as high as 23% in 3%CRM and 43% in 5%CRM mixtures. This is probably the effect of bitumen stiffening following short-term ageing of the mixtures resulting in an increase in mixture stiffness. However, the results are contradictory to the control mixtures, where although stiffness increased with short-term ageing, the predicted strain is approximately 53% (for highly compacted mixture) and 6% (for poorly compacted mixture) higher than for mixtures produced without conditioning (0 hour).

In terms of number of load cycles associated with 100με, with the exception of R0-C0-L, all the other control and CRM mixtures show a reduction in fatigue life. Although all the mixtures were produced using the same percentage of bitumen with the same manufacturing technique, the flexible rubber is believed to be the main reason for the superior fatigue life characteristics of the CRM mixtures despite initial absorption of lighter fractions of bitumen due to short-term ageing.

Table 8.8: Fatigue life comparison as a function of short-term conditioning

Mixture	Strain @ 10^6 cycles (με)		Percentage change (%)	Cycles @ 100 με		Percentage change (%)
	0 hrs	6 hrs		0 hrs	6 hrs	
R0-L	36	55	53	20,040	84,450	321
R0-H	49	52	6	44,066	32,647	-26
R3-L	111	86	-23	705,734	301,434	-57
R3-H	76	81	7	281,357	124,877	-56
R5-L	134	76	-43	416,486	249,970	-40
R5-H	114	87	-24	384,589	272,270	-29

8.3.5 Compaction Effort

The results presented in Table 8.9 indicate that in terms of the predicted strain and fatigue life, as with the short-term ageing, the testing of mixtures at 4 and 8% air void content does not appear to significantly alter the fatigue performance of either the control or CRM asphalt mixtures. In terms of predicted number of load applications, it can be observed that there was a substantial reduction in fatigue resistance for the 8% air void content R3 mixtures compared to the denser, 4% void content specimens. The reduction is comparatively lower in R5 mixture, indicating that 5% CRM mixture is less affected compared to R3 and R0 mixtures. However, with the exception of R5-C6-H, 3%CRM mixtures generally exhibit higher number of load pulses compared to corresponding 5%CRM mixtures. Finally, although there is a general reduction in fatigue life in less compacted CRM mixtures, their overall fatigue lives are still superior compared to corresponding control mixtures.

Table 8.9: Fatigue life comparison as a function of voids contents

Mixture	**Strain @ 10^6 cycles (με)**		**Cycles @ 100 με**	
	4% air voids	**8% air voids**	**4% air voids**	**8% air voids**
R0-C0	36	49	20,040	44,066
R0-C6	55	52	84,450	32,647
R3-C0	111	76	705,734	281,357
R3-C6	86	81	301,434	124,877
R5-C0	134	114	416,486	384,589
R5-C6	76	87	249,970	272,270

8.3.6 Mixture Types

Direct comparison of the indirect tensile fatigue performance of the control and CRM mixtures from this investigation with other dense graded mixtures was difficult due to the scarcity of data and differences in test conditions. However, results from selected previous research projects conducted on DBM mixtures were used as an indication of the relative performance. Read (1996) at the University of Nottingham conducted an investigation using ITFT for dense graded 20 mm DBM, 20 mm HDM and 28mm DBM mixtures. The fatigue equations from these mixtures were compared with the highly compacted control and CRM mixtures used in this project. All the results in terms of fatigue and strain equations together with predicted strain and fatigue life are presented in Table 8.10. It can be seen that both CRM mixtures have better fatigue performance compared to all the other conventional mixtures. The number of cycles to failure at 100με achieved by the CRM mixtures was much more than achieved by the previous investigation (Read, 1996) on unconditioned 20mm DBM mixtures. The initial tensile strain for 10^6 cycles to failure for the CRM mixtures was also substantially higher than that found for the conventional DBM and HDM mixtures.

Table 8.10 Comparison of fatigue testing of different mixtures with previous research

Mixture type	Mixture properties	Strain equation	με at 10^6 cycles	Fatigue euqation	N_f at 100με (10^3)
R0-C0-L	Bitumen grade=100/150pen Bitumen content =5.25% by mass, Aggregate =limestone, Filler= Limestone Design voids= 4%	$\varepsilon=1579\ N_f^{-0.2815}$	32	$N_f=6.0 \times 10^{10}\ \varepsilon^{-3.2947}$	15
R0-C6-L		$\varepsilon=1876\ N_f^{-0.2553}$	55	$N_f=1.0 \times 10^{12}\ \varepsilon^{-3.55}$	79
R3-C0-L		$\varepsilon=887\ N_f^{-0.1502}$	111	$N_f=5.0 \times 10^{16}\ \varepsilon^{-5.4163}$	735
R3-C6-L		$\varepsilon=563\ N_f^{-0.1362}$	86	$N_f=9.0 \times 10^{19}\ \varepsilon^{-7.2368}$	302
R5-C0-L		$\varepsilon=858\ N_f^{-0.1344}$	134	$N_f=1.0 \times 10^{15}\ \varepsilon^{-4.6945}$	408
R5-C6-L		$\varepsilon=1284\ N_f^{-0.2044}$	76	$N_f=9 \times 10^{14}\ \varepsilon^{-4.4489}$	249
20 mm DBM (Read)	Bitumen grade= 100Pen Bitumen content =4.7% Aggregate= Limestone Void contents =3.5%	$\varepsilon=1562\ N_f^{-0.246}$	52	$N_f=9.81 \times 10^{12}\ \varepsilon^{-4.068}$	72
28 mm DBM (Read)	Bitumen grade= 50Pen Aggregate= Gritstone/Granite Bitumen content =4.7% Void contents =9.3%	$\varepsilon=2595\ N_f^{-0.255}$	77	$N_f=2.45 \times 10^{12}\ \varepsilon^{-3.923}$	349
20 mm HDM (Read)	Bitumen grade= 50Pen Aggregate= Limestone Void contents =3.2%	$\varepsilon=1144\ N_f^{-0.239}$	42	$N_f=5.99 \times 10^{12}\ \varepsilon^{-4.178}$	26

8.4 PERMANENT DEFORMATION

8.4.1 CRLAT Testing Protocol

The permanent deformation resistance of the different asphalt mixtures was determined by means of the Confined Repeated Load Axial Test (CRLAT) on all C0 and C6 mixtures using a direct uniaxial compression configuration on specimens with a diameter of 100 mm and a height of 60 mm. The tests were conducted at 60^0C to simulate extreme high temperature pavement conditions. A picture of the testing arrangement is presented in Figure 2.20 of Chapter 2. The CRLA tests were performed in accordance with the British Standard DD 226: 1996 using the following test parameters:

- Test temperature: 60°C,
- Test duration: 7200 seconds (3600 cycles) with a load pattern 1 second loading on (load application period) followed by one second off (rest period),
- Axial stress: 100 kPa,
- Confining pressure: 70 kPa, and
- Conditioning stress: 10 kPa for 600 seconds
- All test specimens were subjected to 2 hours conditioning at the test temperature prior to testing.

As there is no established parameter to use in assessing the response of CRM mixtures in the CRLA test, a comparison of the results was provisionally made, using the minimum strain rate, mean strain rate and total strain as reported by Brown *et al* (1995) and Gibb (1996) for bituminous mixtures. They also reported that under repeated loading, larger strains and hence lower creep stiffness values might occur due to the effect of the pulsed loading on the aggregate skeleton. A brief explanation of the calculation procedure is presented below.

A typical plot of permanent deformation against load cycles, obtained from permanent deformation tests, shows three phases of material response (Preston, 1991). Figure 8.11 shows an idealised plot of deformation against load cycles in the form generally obtained from permanent deformation tests.

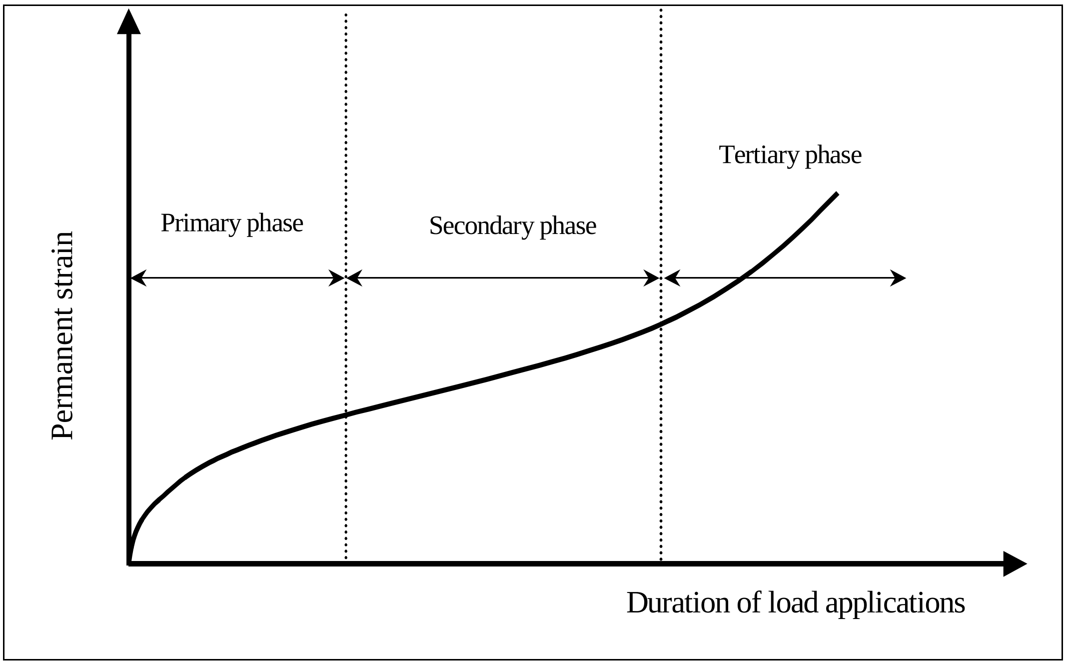


Figure 8.11: Schematic creep curve

Considerable variation between individual test specimens is often observed in the primary phase and normally measurements are taken from the secondary phase where the response is more steady (Morris *et al*, 1974).

- Primary phase, in which a combination of adjustment of loading platens and material densification occurs; generally, the strain rate gradually decreases in this phase.

- Secondary phase, in which a steady state response takes place, attributed to a restructuring of the mineral aggregate matrix.

- Tertiary phase, representing a viscous flow of the binder within the aggregate. In this phase, the strain rate increases and is related to failure of the test specimen. In this investigation the tertiary stage is not included, as the tested specimens did not fail within the required loading time (3600 pulses).

Generally the mean strain rate parameter is weighted to emphasise performance of the specimen in the initial phase of the test whereas the minimum strain rate reflects more the behaviour of the specimen in the later stages. But if the material exhibits low strain rates, the expression of minimum strain rate may not be sufficiently sensitive to discriminate effectively between better performing materials (Brown *et al*, 1995, Gibb, 1996). However, as CRM materials are flexible in nature, the minimum strain rate would be a useful method to determine the deformation behaviour of the mixtures.

The parameters are calculated using the following mathematical expressions:

- Minimum strain rate

After careful observation of all the deformation curves generated in this investigation, the slope was taken from 1500 to 3000 pulses and calculated as;

$$\textit{Minimum strain rate (strain rate of the steady state portion)} = \frac{\varepsilon_{3000} - \varepsilon_{1500}}{1500} \quad \textbf{(8.1)}$$

Where;

ε_{3000} = strain recorded at 3000 pulses

ε_{1500} = strain recorded at 1500 pulses

- Mean strain rate

Mean strain rate was calculated over the whole test period by using the following formula;

$$\textit{Mean strain rate} = \frac{\sum_{i=0}^{3600} \frac{\Delta\varepsilon_i}{\Delta n_i}}{N} \quad \textbf{(8.2)}$$

Where;

$\Delta\varepsilon$ = $\varepsilon_{i+1} - \varepsilon_i$,

Δn = $n_{i+1} - n_i$

ε_i = strain recorded at increment i

n_i = number of cycles elapsed at increment i

N = number of increments at which strain is recorded (3600 load pulses)

- Total strain

Cumulative axial strain at the end of the 3600 load pulses.

8.4.2 CRLAT Test Results

The permanent deformation results in terms of permanent strain versus test duration for the highly compacted R0-C0-L, R3-C0-L and R5-C0-L mixtures tested at 60^0C with 70kPa confinement are presented in Figures 8.12 to 8.14. For brevity only three graphs are shown although similar plots were obtained for the other mixtures and are included in Appendix G.

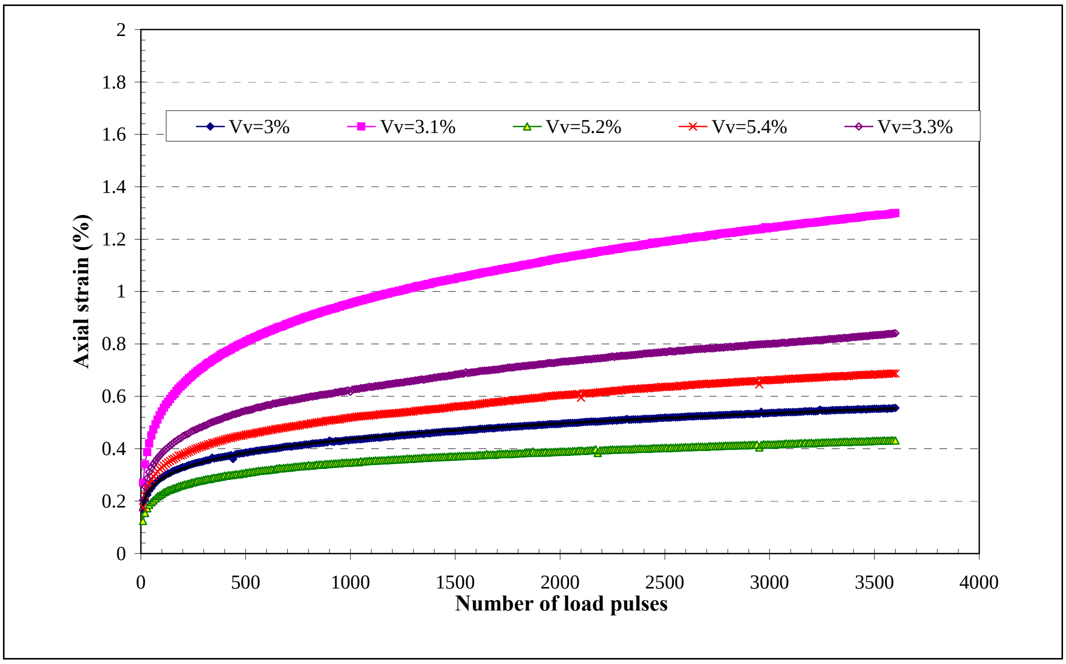


Figure 8.12: CRLAT test results of R0-C0-L mixtures tested at 100kPa stress, 70kPa confinement and at 60^0C temperature

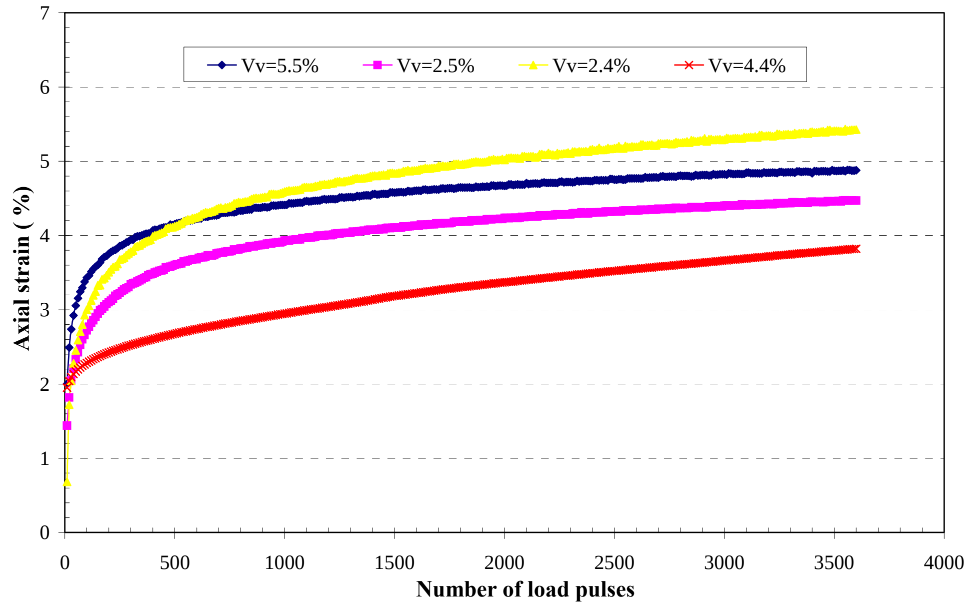


Figure 8.13: CRLAT test results of R3-C0-L mixture tested using 100kPa stress, 70kPa confinement and at 60^{0}C temperature

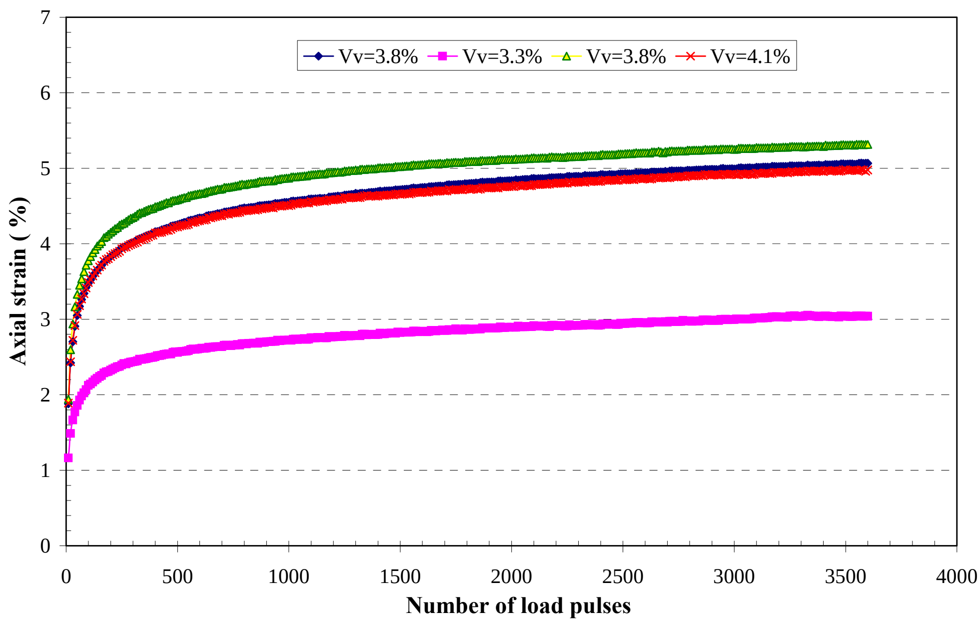


Figure 8.14: CRLAT test results of R5-C0-L mixtures tested using 100kPa stress, 70kPa confinement and at 60^{0}C temperature

The results demonstrate that compared to the control mixtures, the CRM mixtures produce higher permanent strain irrespective of the amount of rubber in the mixtures. The results are similar for all short-term age conditioned mixtures with high and low void contents. It is important to note that after conditioning at 60^{0}C for 2 hours prior to testing, all the CRM specimens expanded vertically, as shown in Figure 8.15. As the mixtures were produced with an extra compaction effort, the rubber particles were compressed in the mixture matrix. But at high temperatures when the bitumen is less capable of halting the rebound of rubber particles, rubber particles expand due to their resilient nature resulting in expansion of the specimen. In addition, it was observed that the expansion was not uniform, probably due to the non-uniform distribution of rubber particles as presented in Section 7.2.4. As a result, initial strain and total strain was significantly increased.

The non-uniform expansion could also affect the test results in the initial stage due to differential readings by the LVDTs on the uneven surface. However, this problem was minimised as the tests progressed to the steady state stage. A typical picture of an R5-C0-L CRM sample after CRLAT testing is presented in Figure 8.16. It can be seen that due to the lower stiffness and expansion during test conditioning, the height of the specimen is considerably reduced.

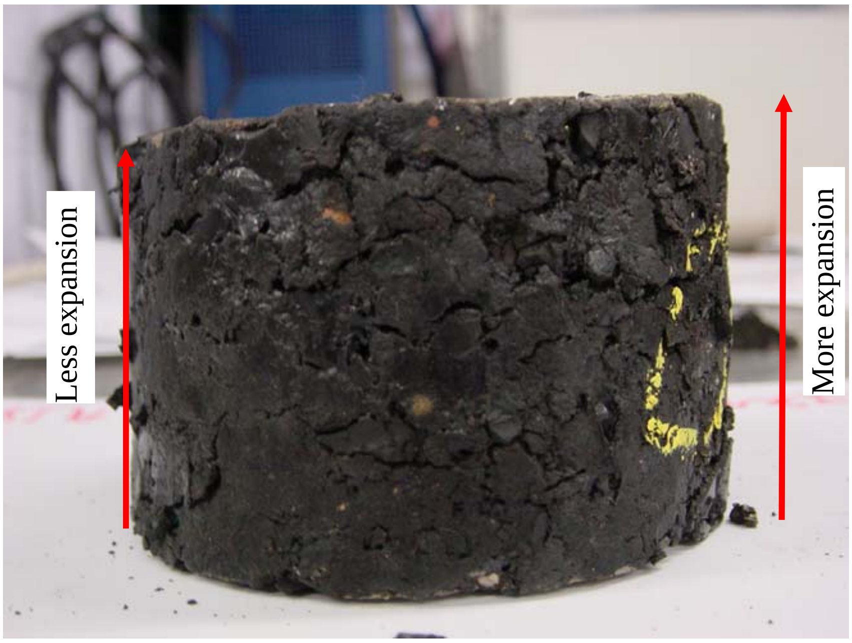


Figure 8.15: R5-C0-L specimen subjected to 2 hours pre-conditioning at 60^{0}C prior to CRLAT testing

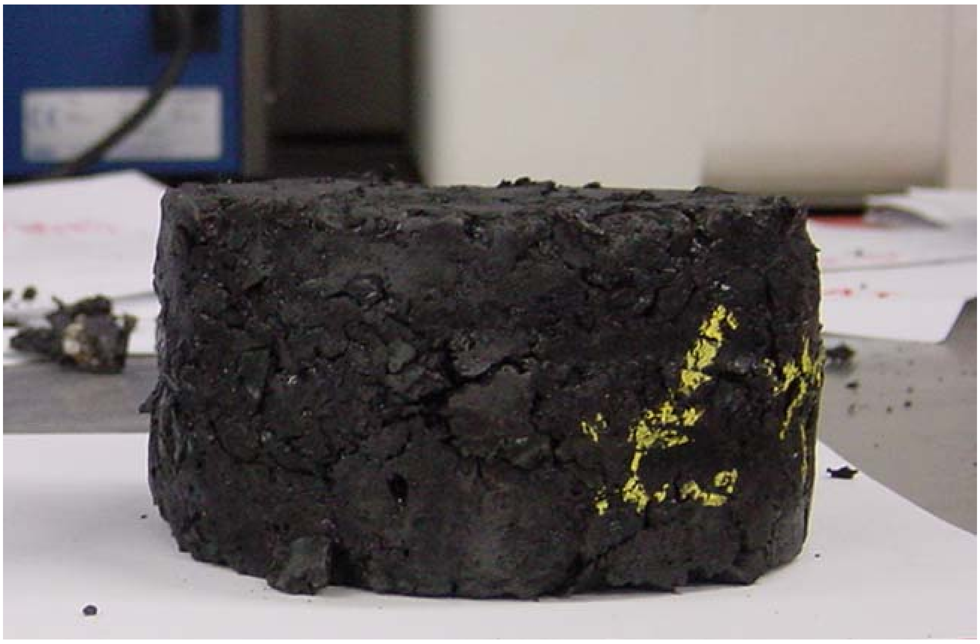


Figure 8.16: R5-C0-L specimen subjected to 2 hours pre-conditioning at 60^0C prior to CRLAT testing and tested at 60^0C with 70kPa confining pressure for 3600 seconds.

The permanent deformation results in terms of ultimate strain or total strain and minimum and mean strain rate for the twelve control and CRM DBM asphalt mixtures are presented as measures of total strain (%) after 3600 cycles, minimum strain rate (microstrain/cycle) from the steady state part (1500 to 3000 pulse) and mean strain rate (microstrain/cycle) over whole test period in Table 8.11. In addition, total strain, mean and minimum strain rate for both high (low air voids) and low (high void content) compacted mixtures are plotted in Figures 8.17 and 8.18.

Table 8.11: CRLAT test results as a function of mean strain rate, minimum strain rate and total strain

Mixture	Mean strain rate (με/cycle)		Minimum strain Rate (με/cycle)		Total strain (%)	
	Avg	Std dev	Avg	Std dev	Avg	Std Dev
R0-C0-L	16	7.9	0.61	0.33	0.76	0.34
R6-C6-L	8	5.0	0.23	0.14	0.40	0.25
R3-C0-L	87	33.2	2.09	0.78	4.65	0.68
R3-C6-L	101	16.6	3.86	1.77	3.75	0.58
R5-C0-L	80	18.9	1.27	0.24	4.60	1.05
R5-C6-L	74	22.0	1.75	1.25	4.74	1.37
R0-C0-H	71	14.7	3.84	1.08	3.16	0.66
R0-C6-H	60	15.9	2.61	1.21	2.67	0.55
R3-C0-H	54	13.1	2.93	0.98	3.54	0.52
R3-C6-H	68	14.1	3.05	1.70	4.32	0.19
R5-C0-H	123	60.3	3.81	1.35	6.00	0.59
R5-C6-H	97	18.1	1.85	0.37	5.63	0.92

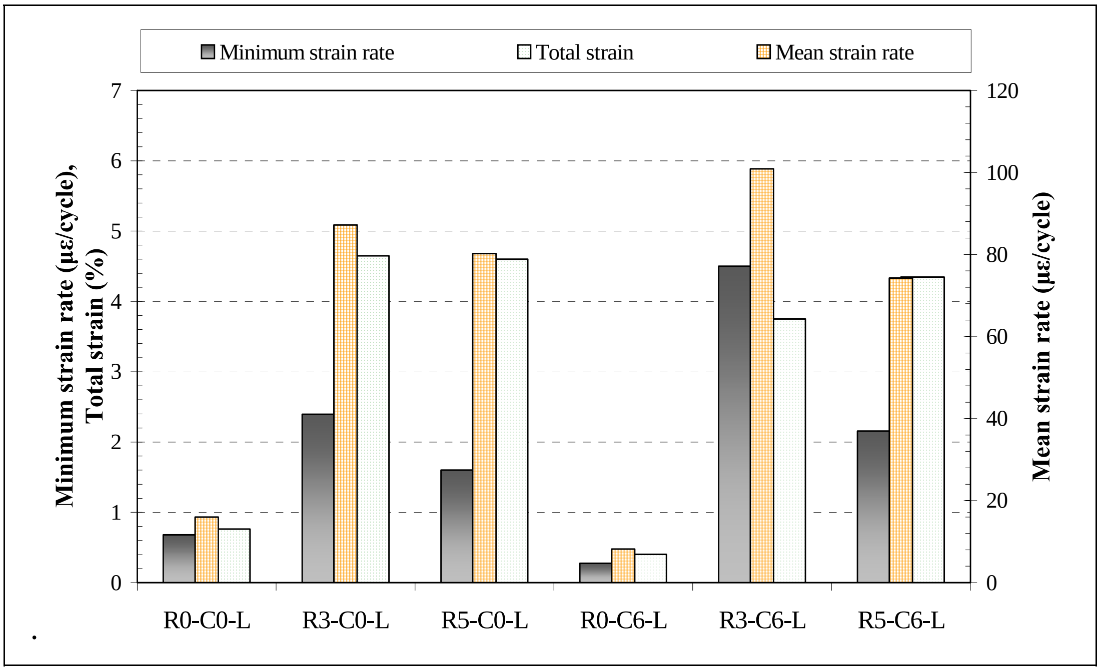


Figure 8.17: Minimum strain rate, ultimate strain and mean strain rate of all highly compacted mixtures

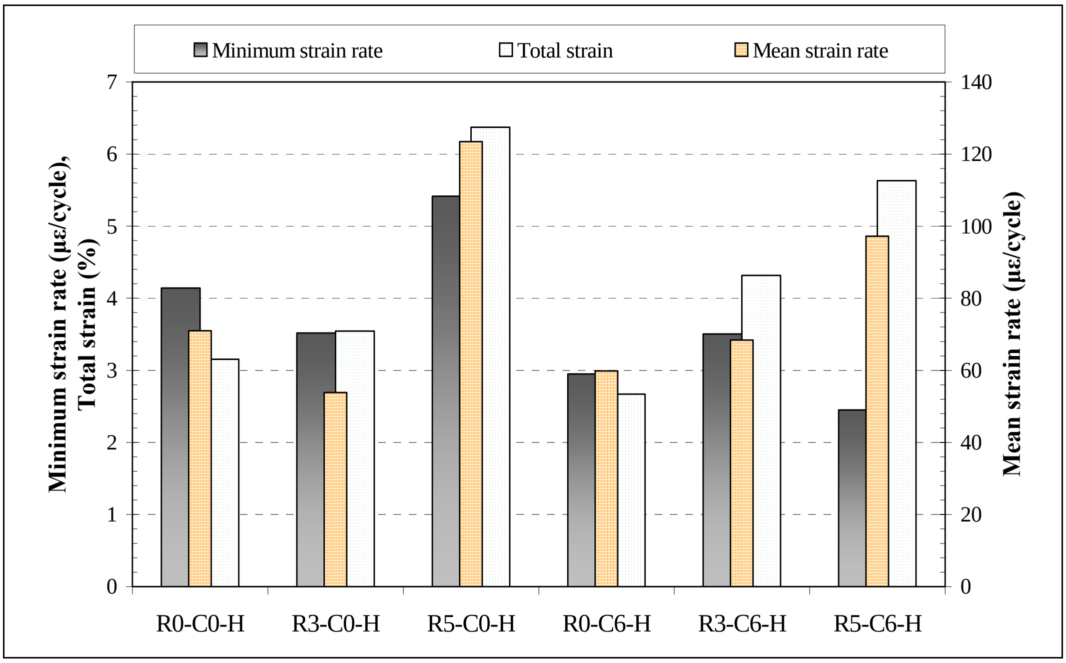


Figure 8.18: Minimum strain rate, ultimate strain and mean strain rate of all low compacted mixtures

In terms of total strain, the replacement of part of the aggregate fraction with crumb rubber results in an increase in permanent strain after 3600 cycles relative to the control mixtures, although this increase is lower for the high void content (8%) mixtures. However, in terms of mean and minimum strain rate, the modification of the DBM mixtures through the addition of crumb rubber only shows an increase for the low void content mixtures while the strain rates for the high void content mixtures are all relatively comparable. It is also interesting to note that although the total strains for the 5% CRM mixtures are greater than those of the 3% CRM mixtures, the minimum strain rates tend to be generally lower for the higher rubber content mixtures.

The differences in material ranking by means of the three permanent deformation test parameters can be attributed to the sensitivity of the total strain parameter to any initial slack in the test apparatus as well as the fundamental differences in what each parameter is measuring. For total strain, any delayed elastic response that cannot be recovered during the one second recovery period will be added to the final strain measurement, while the strain rate parameter is a more direct measurement of the viscous response (permanent strain) of the material. For the highly elastic (rubberised), lower stiffness modulus CRM mixtures, the proportion and actual strain magnitude of the delayed elastic component will inevitably be relatively high resulting in an increase in total strain. For this reason the strain rate parameters (minimum and mean strain rate) can be considered to be a more reliable and accurate means of assessing the permanent deformation performance of the dry process CRM asphalt mixtures although mean strain rate calculations could be affected by high initial strain and total strain. However, both strain rates are exclusively used in the following sections to evaluate the effect of rubber content, short-term ageing and compaction effort on the permanent deformation behaviour.

8.4.3 Rubber Content

The influence of rubber content on permanent deformation performance is assessed in more detail in Table 8.12 where, in addition to the actual strain rate values, the percentage changes in strain rate relative to the four control asphalt mixtures have been included. For the high compaction effort (low void content) mixtures, the effect of crumb rubber modification has significantly reduced the permanent deformation performance of the CRM asphalt mixtures compared to the controls, although it is the 3% CRM mixtures that have produced the larger minimum strain rates rather than the higher rubber content 5% CRM mixtures.

Comparing 3 to 5% CRM mixtures, with the exception of R5-C6-L, the percentage increase in mean strain rates is higher for 5% CRM mixtures. This indicates greater susceptibility to high temperature permanent deformation at the initial stage of testing where the higher amount of rubber is believed to be the main contributory factor. Similar response in both highly and poorly compacted mixtures also suggests that the influence of rubber content is far higher than compaction effort.

For the high void content mixtures, the large reduction in rutting resistance of the two control mixtures means that the relative performance of the control and CRM mixtures are all very similar. In terms of minimum strain rate, there is even a slight improvement in permanent deformation resistance for three of the four combinations of rubber content and conditioning time.

Table 8.12: Relative permanent deformation performance of CRM asphalt mixtures

Minimum strain rate (με/cycle)				Percentage change (%)	
Mixture	**0% Rubber**	**3% Rubber**	**5% Rubber**	**3% Rubber**	**5% Rubber**
C0-L	0.61	2.09	1.27	243	108
C6-L	0.23	3.86	1.75	1578	661
C0-H	3.84	2.93	3.81	-24	-1
C6-H	2.61	3.05	1.85	17	-29
Mean strain rate (με/cycle)				**Percentage change (%)**	
C0-L	16	87	80	444	74
C6-L	8	101	74	1163	65
C0-H	71	54	123	-24	96
C6-H	60	68	97	13	54

8.4.4 Short-Term Ageing

The effect of short-term oven ageing on the permanent deformation performance of the control and CRM mixtures is shown in Table 8.13. As expected, hardening the binder of the control mixtures leads to a decrease in minimum and mean strain rate (improved rutting resistance) in both highly and poorly compacted mixtures. However, the effect of short-term ageing on the permanent deformation performance of the CRM mixtures is not as clear with R3-L showing a large increase in minimum strain rate (85%) while R5-H shows a large reduction (-51%). In terms of mean strain rate, 16% increase of strain rate in R3-L mixture against 8% reduction in R5-L mixture and 26% increase in R3-H against 21% reduction in R5-H mixtures indicates that short-term ageing probably has a better influence on mixtures with higher rubber content.

Table 8.13: Permanent deformation performance of short term conditioned control and CRM asphalt mixtures

	Minimum strain rate (με/cycle)		**Percentage change (%)**
Mixture	**0 hour conditioning**	**6 hours conditioning**	
R0-L	0.61	0.23	-62
R0-H	3.84	2.61	-32
R3-L	2.09	3.86	85
R3-H	2.93	3.05	5
R5-L	1.27	1.75	38
R5-H	3.81	1.85	-51
	Mean strain rate (με/cycle)		**Percentage change (%)**
R0-L	16	8	-50
R0-H	71	60	-15
R3-L	87	101	16
R3-H	54	68	26
R5-L	80	74	-8
R5-H	123	97	-21

8.4.5 Compaction effort

Decreased compaction (higher air voids) has a significant effect on the permanent deformation performance of the control mixtures with the minimum strain rates increasing by over 500 and over 1000 percent for R0-C0-H and R0-C6-H respectively compared to their equivalent low void content mixtures (Table 8.14). However, there is a considerably lower effect of decreasing compaction on the rutting resistance of the CRM mixtures with the possible exception of R5-C0-H, which has a 200 percent increase in minimum strain rate compared to R5-C0-L. Similar results were also observed for mean strain rate confirming earlier observations that CRM mixtures are less susceptible to reduced compaction effort than control mixtures.

Table 8.14: Permanent deformation performance as a function of voids contents.

	Minimum strain rate (με/cycle)		Percentage change (%)
Mixture	**4% air voids**	**8% air voids**	
R0-C0	0.61	3.84	530
R0-C6	0.23	2.61	1035
R3-C0	2.09	2.91	39
R3-C6	3.86	3.05	-21
R5-C0	1.27	3.81	200
R5-C6	1.75	1.85	6
	Mean strain rate (με/cycle)		**Percentage change (%)**
R0-C0	16	71	344
R0-C6	8	60	650
R3-C0	87	54	-38
R3-C6	101	68	-33
R5-C0	80	123	54
R5-C6	74	97	31

8.5 SUMMARY

The mechanical performance of twelve dry process CRM continuously graded asphalt mixtures have been assessed in this laboratory study and compared to the performance of four conventional, primary aggregate mixtures. The mechanical properties that were investigated consisted of stiffness modulus, resistance to permanent deformation and resistance to fatigue cracking. In addition to varying the amount of crumb rubber in the asphalt mixture, the mixtures were also subjected to short-term age conditioning and two degrees of laboratory compaction effort. The summaries of this elemental mechanical investigation are listed below;

- In terms of the bearing capacity, the partial replacement of aggregate with 2 to 8 mm crumb rubber results in a substantial reduction in stiffness modulus of approximately 25% for the 3% CRM asphalt mixtures and 45% for the 5% CRM mixtures.

- The influence of short-term ageing and mixture compaction was found to have a similar effect on mixture stiffness irrespective of whether the mixture had been modified through the addition of crumb rubber. In addition, the short term conditioning of the mixture increased stiffness irrespective of compaction effort.

- Fatigue resistance with both the 3% and 5% CRM asphalt mixtures showed superior performance to that of the control mixtures, with and without short-term age conditioning and at both the low and high air void contents.

- Permanent deformation of CRM mixtures is dominated by the rubber content. CRM mixtures undergo more strain after 3600 load pulses compared to control mixtures. The mean strain rate, which reflects the performance of the material during the initial phase of the test, is much higher compared to control mixtures. This is due to the initial slack of apparatus due to the rough top surface of CRM specimens and the presence of highly elastic rubber with lower stiffness in the CRM mixtures. In addition, conditioning at test temperature generated extensive cracking which raised concern over the durability of the CRM mixtures. The minimum strain rate, which reflects the performance of the material in the later stage of testing, was also higher for CRM mixtures in all testing conditions. The results also demonstrate that CRM mixtures are more susceptible to permanent deformation at high temperatures.

- Compared to the control mixtures, the permanent deformation resistance of the CRM mixtures was found to be less affected by compaction effort. In terms of minimum strain rate, the permanent deformation performance of CRM mixtures demonstrated similar performance at both higher and lower void contents.

- Provided that the CRM asphalt mixtures can be compacted to a sensible air void content (maximum of 8%) and ignoring any adverse durability issues, the material can be considered to demonstrate reasonable mechanical properties relative to a control DBM mixture. The use of the CRM material in a binder coarse layer would reduce the impact of the low stiffness modulus of the material while the increased fatigue resistance would be of overall benefit to the pavement.

CHAPTER 9

Durability of CRM Asphalt Mixtures

9.1 INTRODUCTION

Durability of bituminous mixtures is defined as their ability to resist damage caused by environmental factors, including water, temperature and fuel in the context of a given amount of traffic for a long period of time without any significant deterioration. Many factors affect the durability of bituminous mixtures but it is generally agreed that the two primary factors are embrittlement of the bitumen through age hardening and damage due to moisture (Scholz and Brown, 1994). Water damage is normally manifested in the loss of adhesion between the aggregate and the bitumen commonly referred to as stripping and/or loss of strength or stiffness of the mixture. Long-term ageing (hardening) is primarily associated with the loss of volatile components and progressive oxidation of the bitumen during in-place service in the field. Ageing causes an increase in viscosity of the bitumen and a consequential stiffening of the mixture, which results in an increase in elastic modulus and embrittlement of the bituminous mixture. Although the increase of elastic modulus can improve the load distribution capacity and permanent deformation resistance by producing a stiffer material, the increase in brittleness as a result of excessive hardening often leads to pavement cracking and loss of durability in terms of water resistance and moisture susceptibility (Vallerga, 1981, Li and Nazarian., 1995). Both ageing and water damage mechanisms result in a reduction of the structural integrity of the mixture and can lead to an early failure.

This chapter presents the test results and analysis related to the durability characteristics of dry process CRM asphalt mixtures. The main topics included in this study are the moisture and long-term ageing effects on the stiffness, fatigue and resistance to permanent deformation of the CRM mixtures. In the first section of the chapter, the testing protocol for moisture and long-term ageing are described. In the next sections, the effect of moisture and long-term ageing on stiffness modulus, fatigue and permanent deformation are presented in terms of rubber content, short-term conditioning and compaction efforts. A brief summary of the main results from the durability investigation is presented at the end of the chapter.

9.2 TESTING PROGRAMME

A moisture and long-term ageing study was conducted on control (no rubber) and CRM mixtures produced with the following variables:

- Crumb rubber content by mass of total aggregate: 0, 3 and 5%,
- Short-term age conditioning: 0 and 6 hours storage in an oven at 155°C,
- Compaction effort (air void content): 4 and 8%.

The stiffness properties, fatigue and resistance to permanent deformation of the asphalt mixtures were evaluated using the NAT. The ITSM, ITFT and CRLAT testing protocols and coding system were kept the same as presented earlier in Chapter 8 with the introduction of additional code "M" for moisture conditioning and "A" for long-term oven ageing. For example, MR3-C6-H refers to the moisture conditioned CRM asphalt mixture with 3% rubber content by mass, short-term conditioned for 6 hours with lower compaction effort (8% target voids).

9.2.1 Water Sensitivity Testing Protocol

The procedure for moisture sensitivity testing, which was followed in this investigation, was based on the Link Bitutest testing protocol (Scholz, 1995) for asphalt mixtures, the detail of which were presented in Section 2.4.1.2. It is important to note that the test method used in this study to predict the water sensitivity of bituminous mixtures is highly empirical (Scholz, 1995); as a consequence, it falls short of accurately predicting field performance. After each conditioning cycle, the specimens were allowed to dry for a further 2 hours at 20^0C in a conditioning chamber prior to ITSM testing and the same procedure was followed for the subsequent cycles. The number of moisture conditioning cycles was limited to three for control mixtures and due to significant stiffness reduction, two for the CRM mixtures. A photograph of the R5-C6-L specimen subjected to two moisture conditioning cycles is presented in Figure 9.1. As shown in the figure, it was observed that during partial vacuum conditioning, rubber particles were plucking out from the surface of the specimen. In addition, during six hours moisture conditioning at 60^0C, the CRM specimens expanded in the vertical direction by approximately 3 to 4 mm and consequently, they appeared to be relatively fragile with extensive cracking especially in the mixtures with 5% rubber content.

Furthermore, the stiffness modulus of each test specimen was measured using the ITSM before and after moisture conditioning. The stiffness modulus ratio was then calculated as the ratio of conditioned stiffness to unconditioned stiffness and was plotted against number of conditioning cycles. It is important to note that after ITSM testing, half the specimens from each mixture were tested using the ITFT for fatigue and the other half was subjected to the Confined Repeated Load Axial (CRLAT) test to evaluate the resistance to permanent deformation. In addition, similar mixtures were also tested in their unaged state (as presented in Chapter 8) and the results compared with corresponding moisture conditioned mixtures.

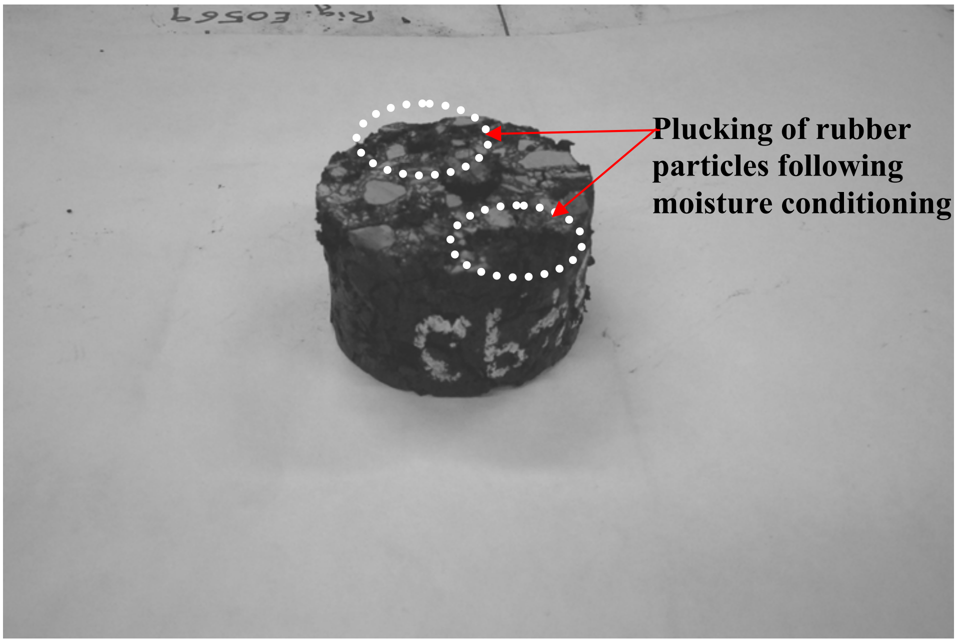


Figure 9.1: R5-C6-L specimen after 2 moisture conditioning cycles

9.2.2 Long-Term Oven Ageing Testing Protocol

The general testing procedure adopted for long-term oven ageing for asphalt mixtures was identical to the SHRP methodology, as set out in project A-003A (AASHTO, 1994) and was presented in Section 2.4.2.3. However, initial trials showed that the CRM specimens required extra protection as significant vertical expansion and rubber plucking (similar to ravelling) was observed during high temperature (85^{0}C) curing and consequently it was impossible to test the damaged specimen in the NAT. To overcome this problem, a simple protective measure was undertaken by covering the specimen with aluminium foil, which was clamped using a thin perforated sheet metal as shown in Figure 9.2.

The ITSM test was performed following the same testing protocols outlined in Section 8.2.1. The stiffness modulus of each specimen was compared before and after long-term ageing. The percentage change was also calculated by taking the difference between unconditioned and long-term conditioned stiffness and divided by the unconditioned stiffness. After ITSM testing following long-term oven ageing, half of the specimens from each mixture were subjected to the ITFT and the other half were subjected to the CRLAT testing.

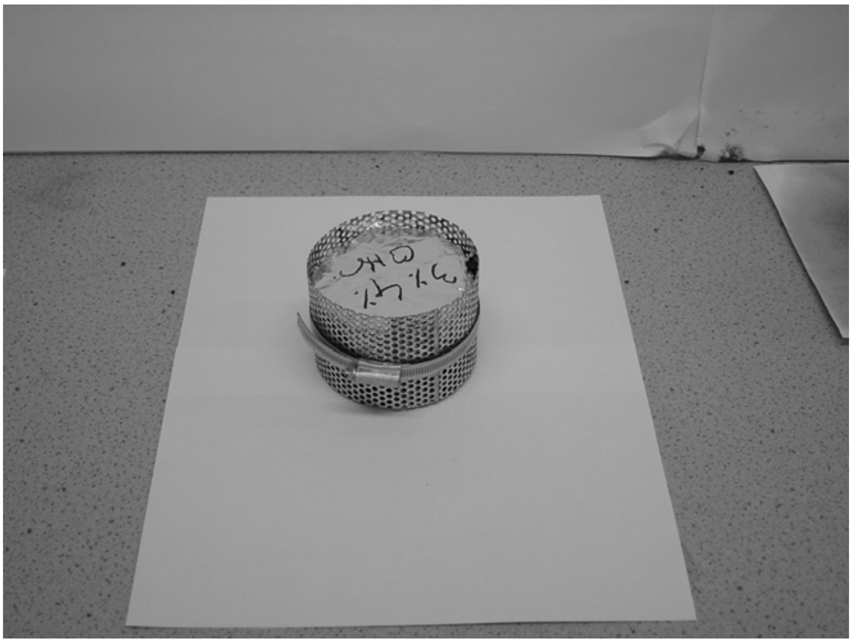


Figure 9.2: Confinement of the samples using steel case and aluminium foil to stop rubber expansion due to long-term oven ageing

9.3 STIFFNESS MODULUS

9.3.1 Moisture Conditioning

Table 9.1 presents a summary of ITSM values of all highly and poorly compacted mixtures including the average, maximum, minimum, standard deviation and calculated percentage saturation using Equation 2.27. The results for individual samples are also included in Appendix E. The results show that there are considerable variations between minimum and maximum stiffness in both control and CRM mixtures. In addition, the stiffness values of 5% CRM mixtures after two moisture-conditioning cycles were very low (<1000MPa) suggesting the adverse effect of moisture on the mixture's bearing capacity. The average stiffness modulus for the control and CRM mixtures subjected to different degrees of moisture conditioning showed that the CRM mixtures undergo a significant reduction in stiffness after the first conditioning cycle. The stiffness reduction was on average 25% for R3 mixtures and 66% for R5 mixtures.

The average stiffness values presented in Table 9.1 were used to calculate the stiffness ratio (ratio of moisture conditioned stiffness to unconditioned stiffness) to investigate the effect of moisture conditioning on the load bearing capacity (stiffness modulus) of the control and CRM asphalt mixtures. All results were analysed in terms of the effect of rubber content, short-term ageing and compaction effort and are discussed in the following sub-sections.

Table 9.1: Summary of stiffness modulus following water sensitivity test for mixtures with target voids of 4% and 8%

Mixture	Voids (%)				Unconditioned stiffness (MPa)				% Saturation	Conditioned stiffness (MPa)											
										1st cycle				2nd cycle				3rd cycle			
	avg	max	min	std	avg	max	min	std		avg	max	min	std	avg	max	min	std	avg	max	min	std
R0_C0_L	**3.2**	4.0	2.4	0.5	**3536**	3646	3360	82	38	**3750**	3907	3575	107	**4136**	4433	3640	224	**4117**	4785	3267	364
R0_C6_L	**3.1**	3.8	2.2	0.5	**4334**	4873	3627	395	42	**5119**	5696	4532	293	**5536**	5832	5040	224	**5617**	6285	4767	364
R3_C0_L	**3.9**	5.9	2.1	1.0	**2753**	3159	2527	190	44	**2131**	2396	1843	147	**2005**	2232	1666	179				
R3_C2_L	**3.5**	5.4	2.2	0.9	**3084**	3349	2825	173	46	**2294**	2500	2022	175	**2122**	2448	1968	165				
R3_C6_L	**4.7**	5.6	3.1	0.7	**3421**	3782	3100	211	47	**2709**	3127	1759	393	**2538**	2881	1783	385				
R5_C0_L	**4.3**	6.0	2.3	1.5	**1984**	2198	1723	181	52	**760**	1142	649	189	**549**	812	422	144				
R5_C2_L	**4.4**	5.7	2.4	1.6	**2165**	2932	1632	555	54	**1213**	1812	793	359	**1084**	1713	727	357				
R5_C6_L	**3.9**	5.5	3.0	0.6	**3398**	4050	2810	335	51	**1155**	1905	732	430	**885**	1378	644	228				
R0_C0_H	**7.6**	8.2	6.6	0.5	**2216**	2768	1908	299	54	**2215**	2522	1949	179	**1986**	2274	1761	169	**1920**	2215	1666	179
R0_C6_H	**7.9**	8.4	7.4	0.2	**3928**	4411	3516	310	50	**3691**	4126	3315	281	**3535**	3822	3095	233	**2386**	3817	2851	346
R3_C0_H	**7.3**	8.2	6.2	0.8	**2154**	2286	1911	121	54	**1979**	2127	1741	131	**1819**	1942	1702	90				
R3_C2_H	**8.3**	9.9	6.5	1.1	**2657**	3261	2232	333	55	**2168**	2537	1826	223	**2053**	2414	1774	224				
R3_C6_H	**8.5**	10.3	6.5	1.1	**2664**	2965	2456	183	52	**2330**	2661	2039	173	**2303**	2560	2007	174				
R5_C0_H	**8.9**	9.9	7.2	0.8	**1467**	1820	1101	241	60	**570**	805	462	107	**529**	826	421	122				
R5_C2_H	**7.8**	10.4	6.1	1.5	**1655**	1929	1860	1724	59	**1007**	1127	914	83	**978**	1142	884	102				
R5_C6_H	**7.3**	9.2	6.3	1	**2642**	3110	2166	308	57	**1225**	1518	875	224	**1142**	1305	901	156				

9.3.1.1 *Rubber Content*

The percentage saturation versus voids contents for the control and CRM mixtures are presented in Figure 9.3. In general, the percentage saturation increases with increasing rubber content in the mixture due to the higher void content of the CRM mixtures. In addition, it was observed that rubber particles generally plucked out from the surface of the specimen which formed paths for water to enter into the mixture matrix, consequently increasing the percentage of saturation during partial vacuum water conditioning. In addition to rubber content, as expected, percentage saturation also increases in the poorly compacted mixtures because of intrusion of water into the voids space. This indicates that the poorly compacted rubber-modified and control mixtures are more likely to be affected by moisture induced damage than the highly compacted control and CRM mixtures.

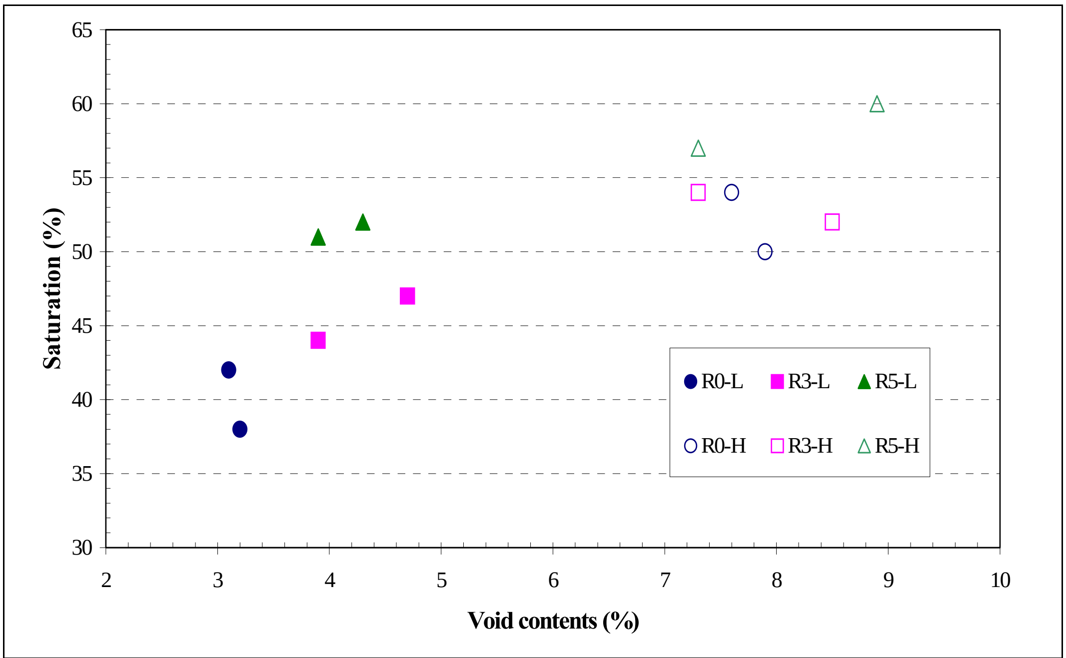


Figure 9.3: Percentage saturation of R0, R3 and R5 mixtures

The stiffness modulus ratio versus conditioning cycles, presented in Figure 9.4 for highly compacted mixtures and Figure 9.5 for poorly compacted mixtures, show that the stiffness modulus ratio for R0 mixtures, increases at lower voids (Figure 9.4) but gradually decreases in mixtures with higher void contents (Figure 9.5). This is expected due to the lower saturation level (40% compared to 52%) of the low void content mixtures which results in a noticeable reduction in moisture damage. On the other hand, irrespective of compaction effort and short-term age conditioning, both the CRM mixtures show a reduction of stiffness with the 3% CRM mixture being significantly lower than the 5% CRM mixtures. This is partly the consequence of increased saturation in the 5% CRM mixture, which allows more water to penetrate into the mixture matrix, and weakens the structure. The increased saturation combined with the rubber rebounding effect during six hours conditioning at 60^{0}C when bitumen is not capable of halting any rebound results in the overall reduction of the mixture's load bearing capacity. Moreover, as the CRM mixtures were produced with an extra compaction effort to achieve target density, the rebound effect of rubber particles is greater in the 5% CRM mixtures due to the extra rubber content in these mixtures.

The modulus ratios were in the range of 0.7 to 0.80 for all the R3 mixtures whereas for the R5 mixtures they were in the range of 0.45 to 0.65. Lottman (1982) reported that the damage in the form of stripping would be severe in the field, when the stiffness ratio decreases to less than 0.80.

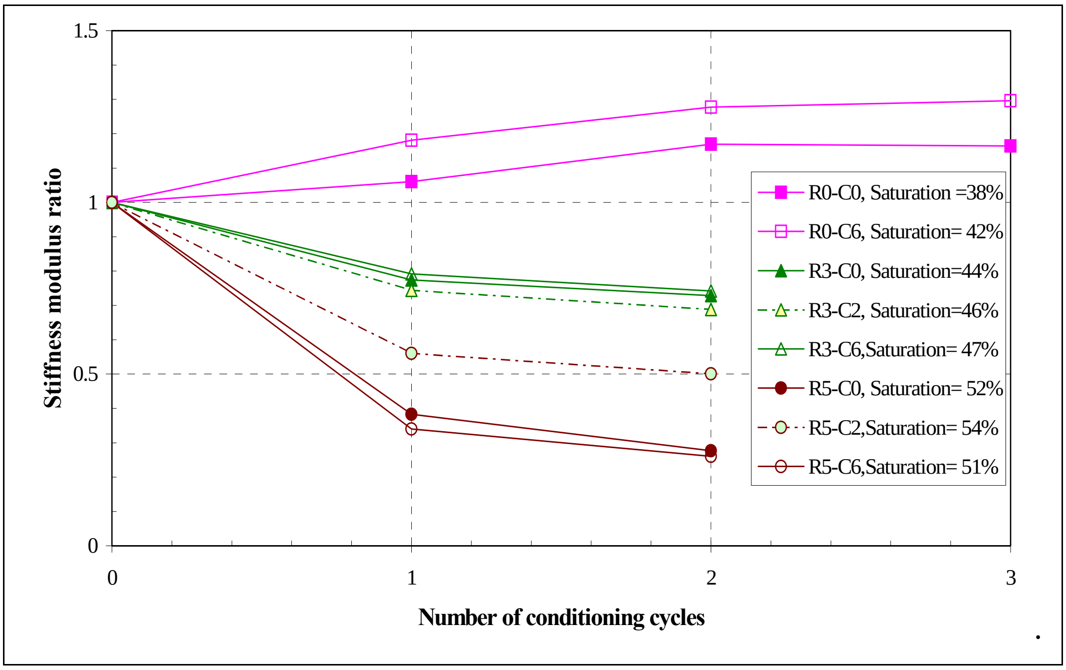


Figure 9.4: Stiffness modulus ratio of the highly compacted mixtures

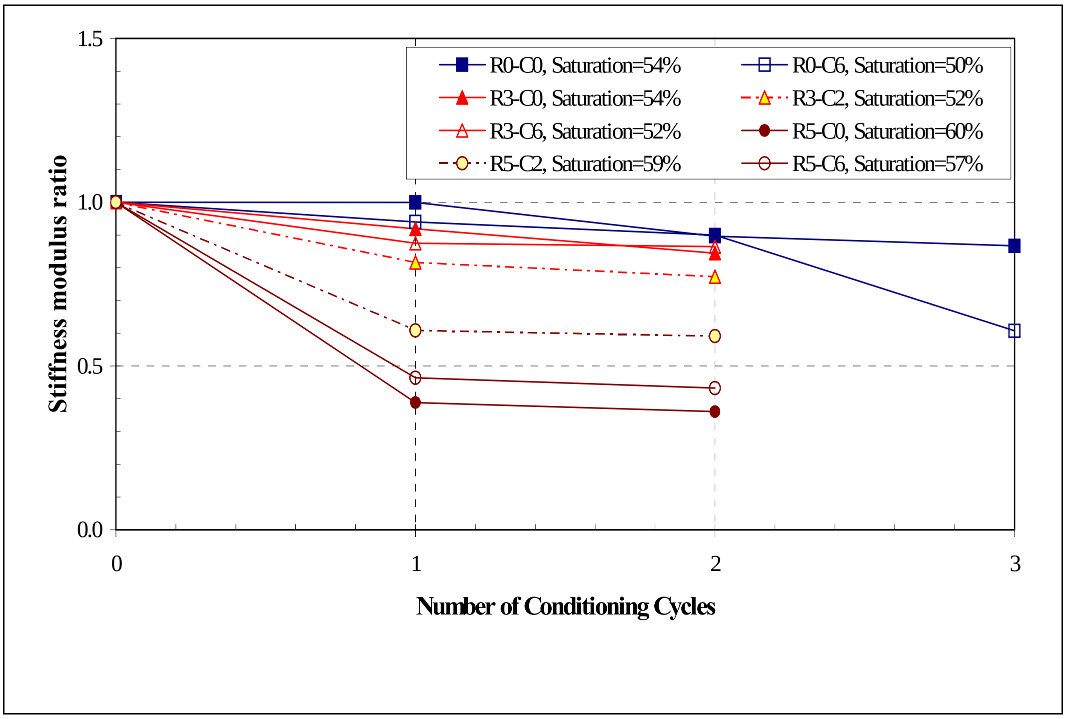


Figure 9.5: Stiffness modulus ratio of the poorly compacted mixtures

9.3.1.2 Short-Term Ageing

The results presented in Figures 9.4 and 9.5 indicate that after two conditioning cycles, the stiffness modulus ratio for all the R3-L mixtures was reduced to approximately 0.7 and for all the R5-L mixtures it was less than 0.5. The results are similar in poorly compacted mixtures although slightly better ratios were observed for both the short-term aged R3-H and R5-H mixtures. On the other hand, highly compacted and short-term aged control mixture (R0-C6-L) showed slightly improved performance compared to R0-C0-L mixture, which may be due to the bitumen hardening during high temperature curing prior to compaction. However, the results are opposite in the poorly compacted control mixture where stiffness ratio reduced to 0.60 for the R0-C6-H mixture compared to 0.9 for the R0-C0-H mixture. This is probably the intrusion of water into the voids space causing more damage in the aged bitumen resulting in the overall reduction in stiffness of the mixture. Therefore, the issue of short-term ageing does not appear to have any added benefit or any adverse effect on the stiffness of the CRM mixtures after moisture conditioning in the way it has for the control mixtures.

9.3.1.3 Compaction Effort

The percentage change in stiffness moduli relative to unconditioned values (0 cycle) for the highly compacted (low void content) and poorly compacted (high void content) mixtures subjected to several moisture conditioning cycles are presented in Table 9.2. It can be seen that in the higher void content control mixture there is a fairly uniform reduction of stiffness up to 13% following three moisture conditioning cycles. This is probably the direct consequence of intrusion of water into the void space as more saturation (as shown in Figure 9.1) was observed during partial vacuum conditioning. For the highly compacted control mixtures, increased stiffness up to 30% indicates that the mixtures had not been subjected to enough conditioning cycles to induce noticeable moisture damage. On the other hand, after two-moisture conditioning cycles, the reduction of stiffness for the highly compacted 3% CRM mixtures is in the range of 26% to 31% against 14% to 23% for poorly compacted mixtures. Compared to the 3% CRM mixtures, the reduction of stiffness was considerably higher in the corresponding R5 mixtures in both void conditions indicating that increasing rubber content in the mixture would lead to a moisture susceptible material. However, based on the test results it appears that the issue of under compaction (high air void content) is less significant for the CRM mixtures than was found for the control mixtures.

Table 9.2: Percentage of stiffness change for control and CRM mixtures due to moisture conditioning

Mixture	**% Change compared to unconditioned stiffness**					
	4% design voids			**8% design voids**		
	1 cycle	**2 cycle**	**3 cycle**	**1 cycle**	**2 cycle**	**3 cycle**
R0_C0	6	17	16	0	-10	-13
R0_C6	18	28	30	-6	-10	-39
R3_C0	-23	-27	-	-8	-16	-
R3_C2	-26	-31	-	-18	-23	-
R3_C6	-21	-26	-	-13	-14	-
R5_C0	-62	-72	-	-61	-64	-
R5_C2	-44	-50	-	-39	-41	-
R5_C6	-66	-70	-	-54	-57	-

9.3.2 Long-Term Oven Ageing

Table 9.3 presents a summary of stiffness values for all long-term aged mixtures including average stiffness, maximum, minimum, standard deviation and percentage stiffness increase due to long-term oven ageing. The results of individual samples are also included in Appendix E. In general, it can be seen that considerable variation exists between the maximum and minimum stiffness values due to the variation of voids contents in the mixtures. In terms of average stiffness modulus, irrespective of short-term conditioning and/or compaction effort, all the control and CRM mixtures have undergone an increase in stiffness modulus. It is important to note that although five days ageing represents approximately 10 years of service life (AASHTO, 1994) where ageing generally increases the stiffness, it is not known whether the magnitude of the stiffness increase resulting from accelerated ageing procedures accurately represents that which occurs in the actual pavement material (Brown *et al*, 1995). In addition, Brown *et al* (1995) reported that this test method appeared to be sensitive to the volumetric proportion of bitumen and air void contents and could be used for comparative purpose. Therefore, the average stiffness results presented in Table 9.3 were exclusively used to analyse the effect of rubber content, compaction effort and the relative influence of short-term ageing versus long-term ageing on mixture stiffness and are discussed in the following sections.

Table 9.3: Summary of stiffness modulus after long-term oven ageing for mixtures with target voids of 4% and 8%

Mixture type	**Air voids (%)**				**Stiffness (MPa)**								**% Increase in stiffness due to oven ageing**
					Unaged				**5 days oven aged**				
	avg	**max**	**min**	**std**	**avg**	**max**	**min**	**std**	**avg**	**max**	**min**	**std**	
R0_C0_L	2.6	2.9	2.3	0.2	**2907**	3325	2586	248	**3528**	3924	3222	233	21%
R0_C6_L	2.6	3.4	2.1	0.4	**4702**	5135	3777	455	**5584**	6172	5152	310	19%
R3_C0_L	4.7	5.8	2.7	0.9	**2518**	2871	2117	306	**2843**	3225	2226	359	13%
R3_C2_L	4.3	6.0	2.5	1.1	**2642**	3196	2147	313	**2966**	3538	2505	307	12%
R3_C6_L	4.0	6.0	2.7	1.1	**3794**	4664	2823	608	**4415**	5515	3250	746	16%
R5_C0_L	4.2	5.5	1.5	1.3	**1907**	2339	1621	239	**2801**	3246	2463	276	47%
R5_C2_L	3.3	5.8	2.1	1.1	**2398**	3097	1795	357	**3063**	3565	2385	389	28%
R5_C6_L	4.5	5.8	3.3	0.7	**2903**	3270	2487	260	**3133**	3561	2578	293	8%
R0_C0_H	8.0	8.6	6.1	0.7	**2302**	2399	2128	91	**2961**	3279	2694	171	29%
R0_C6_H	8.4	8.8	8.0	0.3	**4163**	4582	3703	304	**5333**	6047	4984	370	28%
R3_C0_H	7.5	10.3	6.0	1.2	**2076**	2500	1576	321	**2443**	2886	1642	415	18%
R3_C2_H	8.1	10.0	6.4	1.2	**2222**	2701	1750	308	**2530**	2943	2082	323	14%
R3_C6_H	6.9	9.0	6.0	1.0	**2828**	3357	2515	278	**3007**	3385	2841	218	6%
R5_C0_H	9.0	10.0	6.1	0.4	**1432**	1673	1036	184	**1803**	2353	1435	262	26%
R5_C2_H	9.6	10.2	8.9	0.5	**1602**	1775	1381	166	**2136**	2422	1752	229	33%
R5_C6_H	8.9	10.4	7.7	0.9	**2147**	2376	1821	205	**2354**	2795	1703	350	10%

9.3.2.1 Rubber Content

The average stiffness values for the control and CRM mixtures together with the percentage stiffness increase with long-term oven ageing are presented graphically in Figures 9.6 and 9.7 which show plots of highly and poorly compacted mixtures subjected to various degrees of short-term age conditioning prior to compaction. It is apparent from the results that irrespective of rubber content in the mixtures, stiffness values in general increased due to age hardening of bitumen. This relatively uniform increase in stiffness for all (with or without short-term conditioning, highly and poorly compacted) control mixtures indicates that bitumen hardening is the main contributory element to improve the load bearing capacity.

Compared to R3 mixtures, with the exception of mixtures R5-C6-L and R3-C2-H, the relative increase in stiffness is higher in R5 mixtures indicating more oily components of bitumen are being lost due to the combined effect of oxidation and rubber-bitumen interaction. In addition, the considerable reduction of percentage stiffness increase between R5-C0 and R5-C6 mixtures is probably the influence of rubber softening due to the rubber-bitumen interaction which compensates the effect of bitumen hardening. Therefore, in terms of rubber content, the results indicate that in addition to bitumen hardening, softening of rubber due to rubber-bitumen interaction also contributes to the overall mixture stiffness.

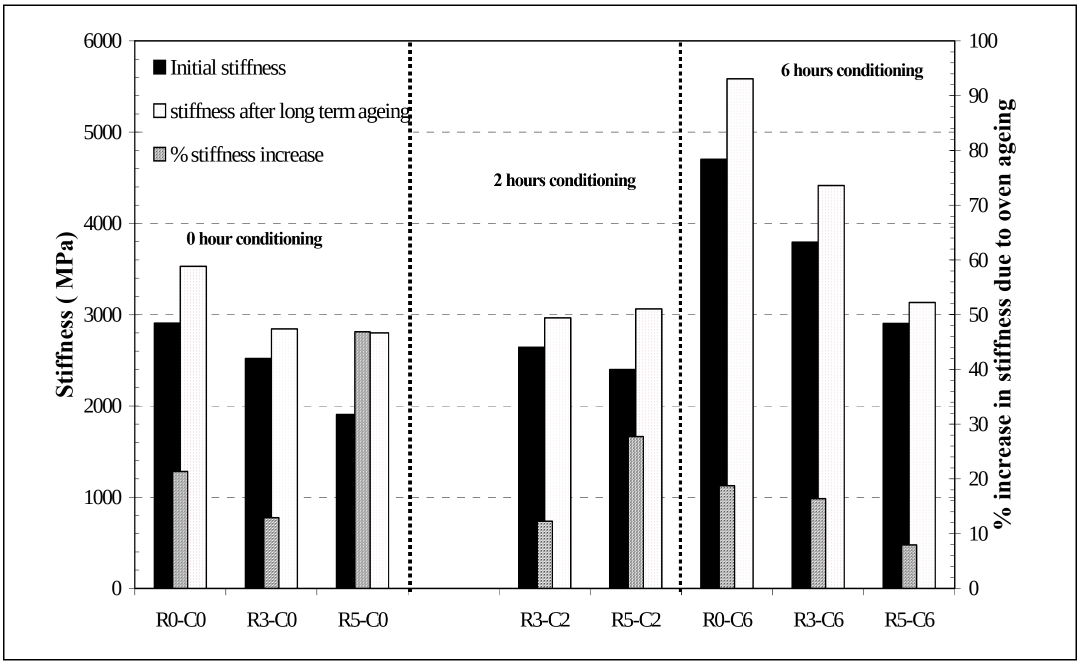


Figure 9.6: Stiffness modulus of highly compacted control and CRM asphalt mixtures

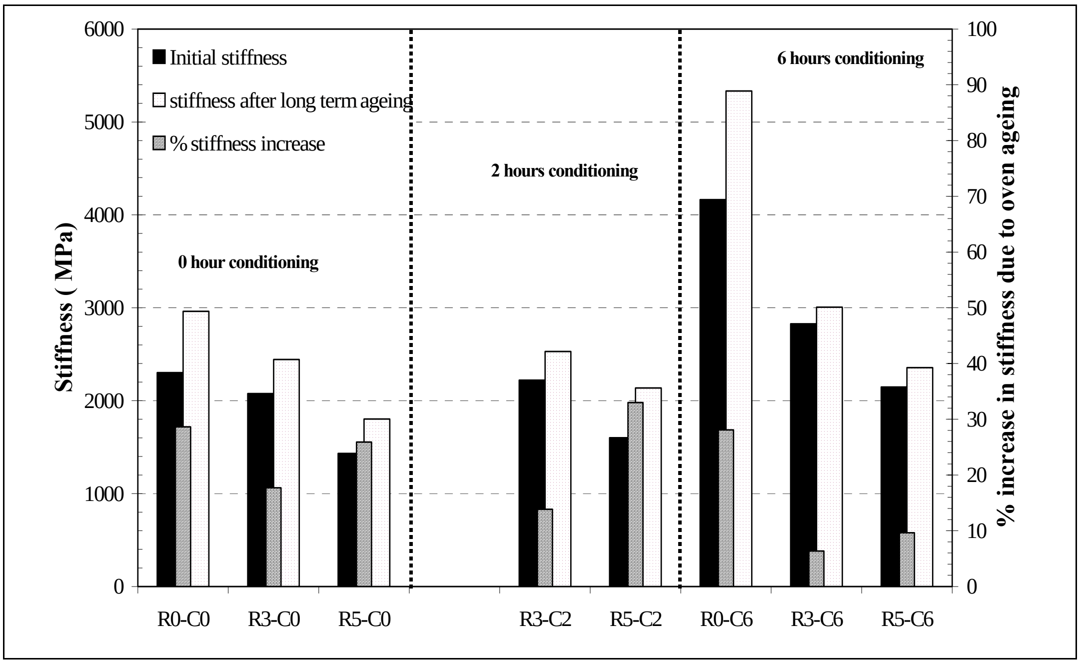


Figure 9.7: Stiffness modulus of poorly compacted control and CRM asphalt mixtures

9.3.2.2 Short-Term Ageing and Long-Term Ageing

A relative influence of both short-term and long-term ageing on the control and CRM mixtures is presented in Table 9.4 including percentage change in stiffness for different ageing conditions. The results were calculated to observe the change in stiffness between 0-6, 0-120 and 6-120 hours of oven ageing for mixtures with both high and low void contents. For simplicity C2 mixtures were not used in the table but their values have been used in Figure 9.8 where the effect of short-term ageing at 155^{0}C and long-term ageing at 85^{0}C is being presented in terms of an ageing index of mixture's stiffness (aged stiffness divided by unaged stiffness) versus ageing period.

The results presented in Table 9.4 illustrate that the relative effect of short-term conditioning is significantly higher compared to long-term ageing which points towards the importance of the mixing, transportation and laying period on mixture's ageing. Another notable observation is the substantial hardening (81%) of the poorly compacted control mixtures compared to the highly compacted mixtures (62%). Similar findings were reported by Whiteoak (1990) where mixtures with lower void contents collected after 15 years of service showed little bitumen ageing compared to similar mixtures with higher void contents as constant ingress of air allows substantial hardening of bitumen.

Table 9.4: Stiffness modulus results for short and long-term conditioned control and CRM mixtures

Mixture	**Average stiffness modulus (MPa)**				**Percentage change (%)**		
	Short-term		**Long-term**		**Short-term**	**Long-term**	
	C0	C6	$C0_{5days}$	$C6_{5days}$	C0-C6	C0-C120	C6-C120
R0_L	2907	4702	3528	5584	62	21	19
R3_L	2518	3794	2843	4415	51	13	16
R5_L	1907	2903	2801	3133	52	47	8
R0_H	2302	4163	2961	5333	81	27	28
R3_H	2076	2828	2443	3007	36	18	6
R5_H	1432	2147	1803	2354	50	26	10

In the case of the highly and poorly compacted CRM mixtures, the effect of bitumen hardening due to short-term and long-term ageing is relatively lower compared to the corresponding control mixtures. It is interesting to note that irrespective of rubber content, the stiffness increase in long-term aged C6 CRM mixtures is relatively lower than C0 mixtures. Three out of four C6 mixtures have shown lower increases in stiffness suggesting that the softening effect of rubber particles following rubber-bitumen interaction may have compensated the hardening of bitumen due to combined influence of rubber-bitumen interaction and oxidation.

The steep slope of ageing index versus curing period plotted in Figure 9.8 also indicates that irrespective of void contents in the mixtures, the relative influence of the short-term ageing is significantly greater on the mixture's hardening than long-term ageing.

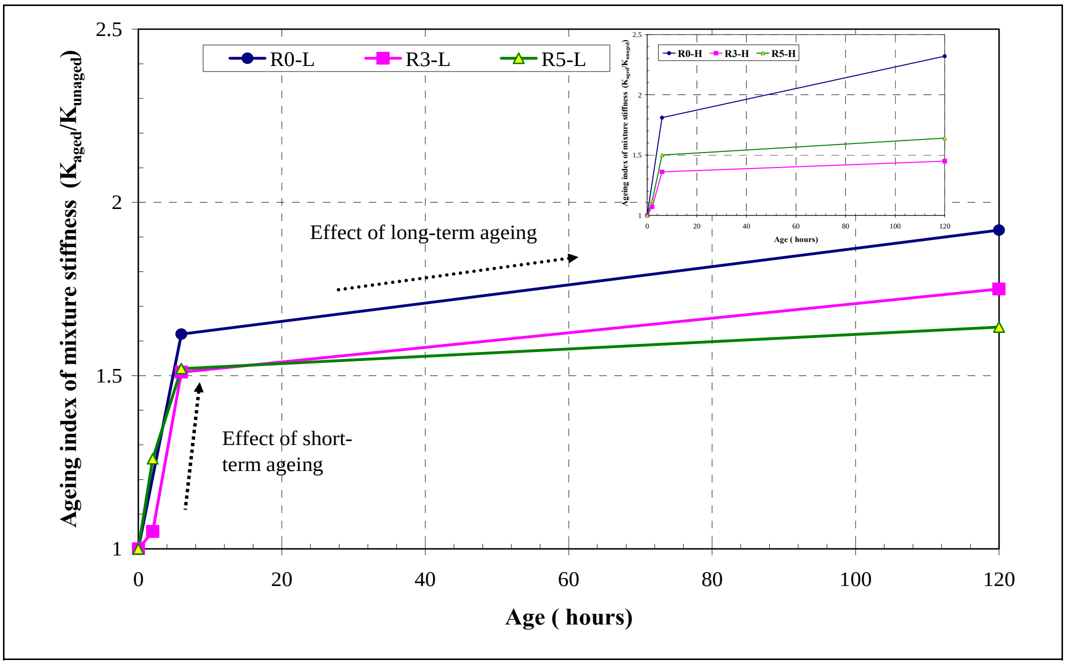


Figure 9.8: Ageing of control and CRM mixtures due to short-term and long-term conditioning

9.3.2.3 Compaction Effort

The stiffness results from Figures 9.6 and 9.7 and Table 9.3 indicate that with the exception of R3-H mixture, there is a fairly uniform increase in stiffness as a result of ageing across all mixtures (control and CRM) despite the changes of air void content. In addition, the ageing of bitumen appears to have more effect on the poorly compacted control mixtures than the highly compacted mixtures where it showed 21% increase in C0-L (19% in C6-L) against 29% in C0-H (28% in C6-H). The increase in stiffness supports the earlier research findings (see Section 9.3.2.2) where more age hardening was observed in poorly compacted mixtures.

On the other hand, for CRM mixtures, relatively similar increases in stiffness in both highly and poorly compacted mixtures indicates the extra bitumen hardening may be compensated by the softening effect of the rubber particles. Therefore, in terms of asphalt mixture stiffness modulus, reduced compaction effort (high air void content) does not appear to have any greater significance on the CRM mixtures durability in the way that it has for the control mixtures.

9.4 FATIGUE PERFORMANCE

The ITFT testing was carried out as detailed in Section 8.3.2 using the following test parameters:

- Test temperature: 20°C,
- Loading condition: Controlled-stress,
- Loading rise-time: 120 milliseconds, and
- Failure indication: 9 mm vertical deformation

As in the unconditioned state, the results were interpreted in terms of initial tensile strain versus number of load repetitions to produce 9 mm vertical deformation. The tensile strain was calculated using Equation 2.26 in Chapter 2. The fatigue parameters, initial tensile strain, number of load repetitions to failure, were plotted for each mixture subjected to moisture and long-term age conditioning and are discussed in the following sections.

9.4.1 Moisture Conditioning

The fatigue lives for the highly and poorly compacted control and CRM mixtures are presented in Figures 9.9 to 9.14. The results are plotted in terms of initial tensile strain and number of cycles to failure. ITFT data for all stress levels are also included in Appendix F. Analysing the results presented in Figures 9.9 to 9.14 and with reference to Appendix F, the following observations can be made;

- In general, the results showed that at both high and low stress levels the number of cycles to failure is considerably reduced in moisture conditioned mixtures than in their unconditioned state.

- Although the fatigue lives for highly compacted and short-term aged control mixture are slightly improved (Figure 9.9) following moisture conditioning, the overall reduction of fatigue lives is significantly higher in all mixtures specially the 5%CRM mixtures.

- Irrespective of compaction effort and short-term ageing, fatigue lives for all CRM mixtures are generally poorer than their unconditioned state results.

- The data points are confined within 10000 cycles for 5%CRM mixtures. This is a consequence of the limitation of the NAT to apply very low stress levels (<100kPa), which are essential to obtain higher number of cycles to failure for very low stiffness CRM material.

- The strains generated in CRM mixtures are considerably higher, especially in 5%CRM mixtures, due to the substantial reduction of the mixture's stiffness following moisture conditioning. As a result, while comparing the relative performance of control and CRM mixtures with their unconditioned state results, the calculation of predicted strain for one million cycles and number of cycles to generate 100 $\mu\varepsilon$ would not be representative due to the lack of widely spread data on the fatigue line. Therefore, the predicted strain was chosen as 200microstrain and predicted life was for 10,000 cycles for comparison purposes.

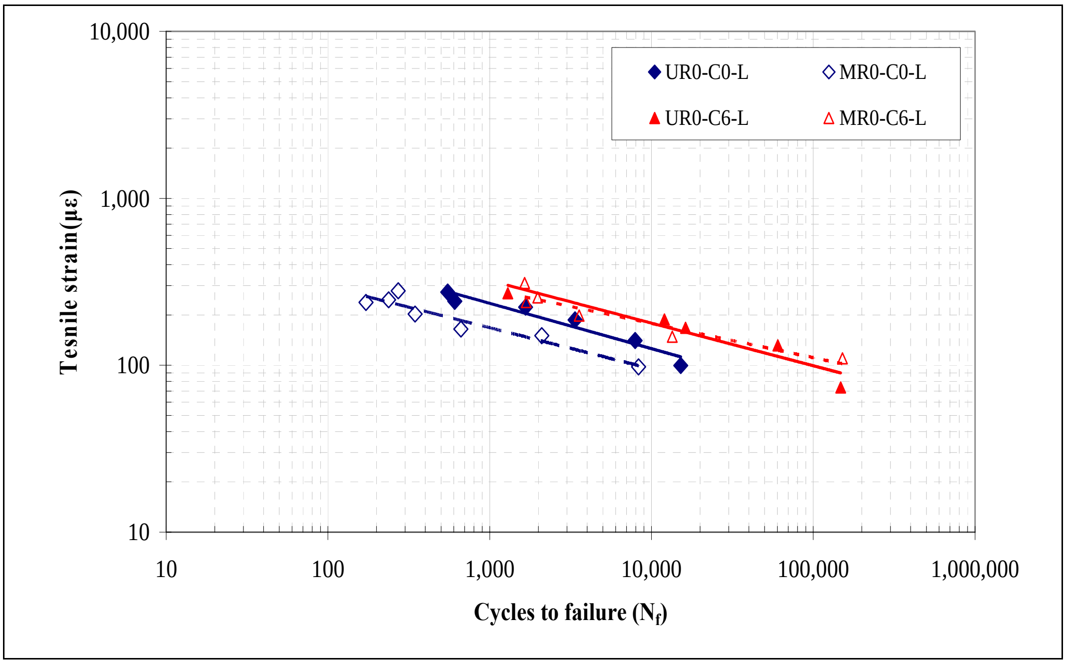


Figure 9.9: ITFT fatigue line for highly compacted control (R0) mixtures tested in unconditioned state and after moisture conditioning

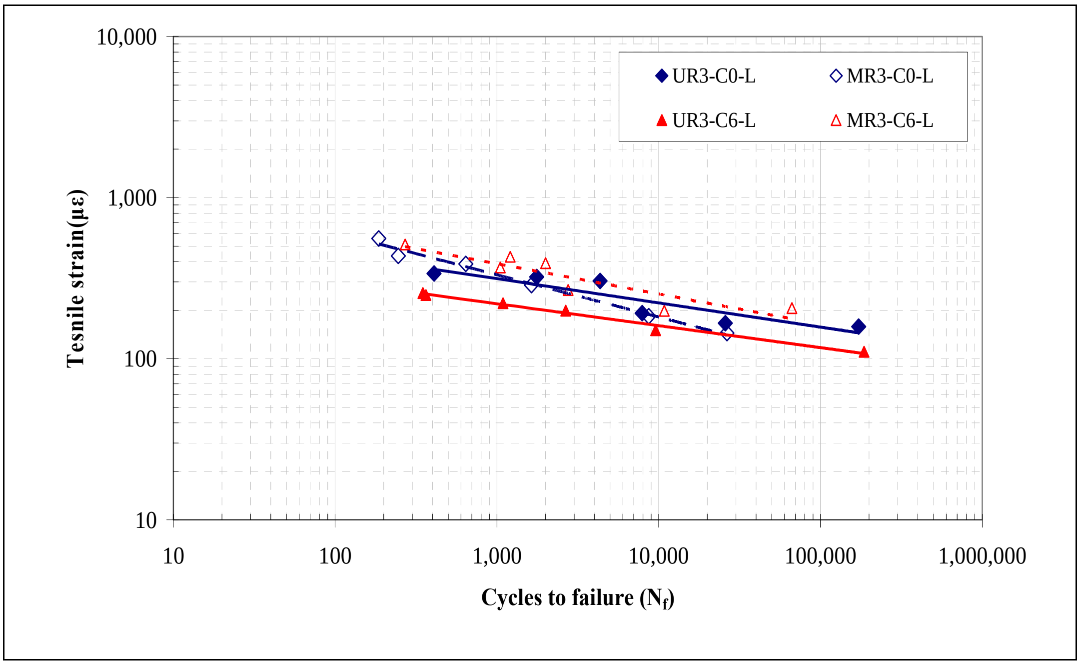


Figure 9.10: ITFT fatigue line for highly compacted 3% CRM (R3) mixtures tested in unconditioned state and after moisture conditioning

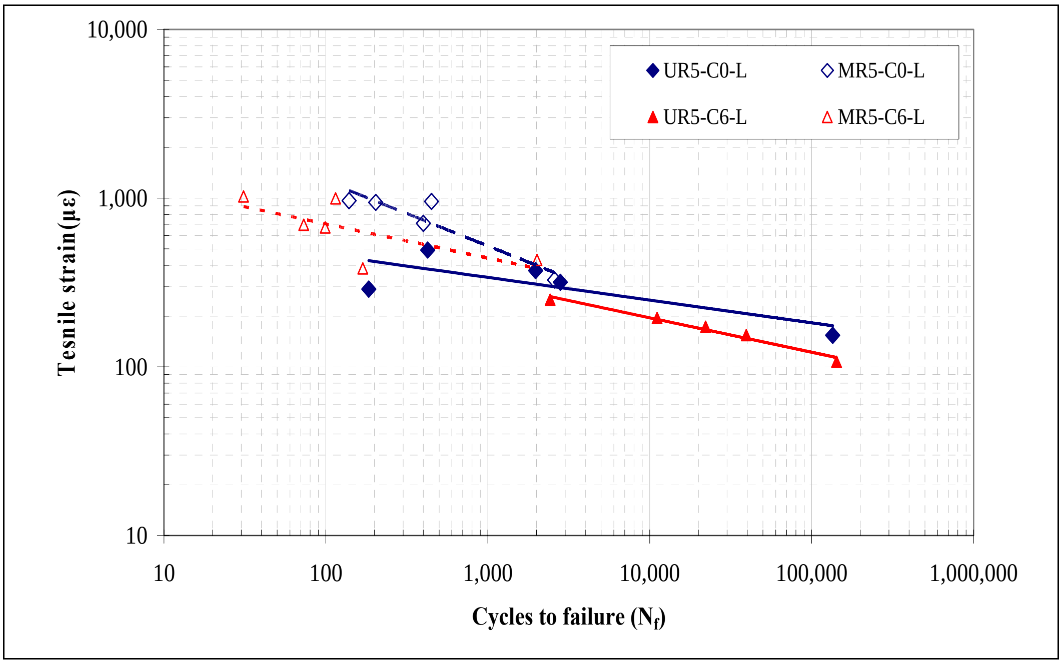


Figure 9.11: ITFT fatigue line for highly compacted 5% CRM (R5) mixtures tested in unconditioned state and after moisture conditioning

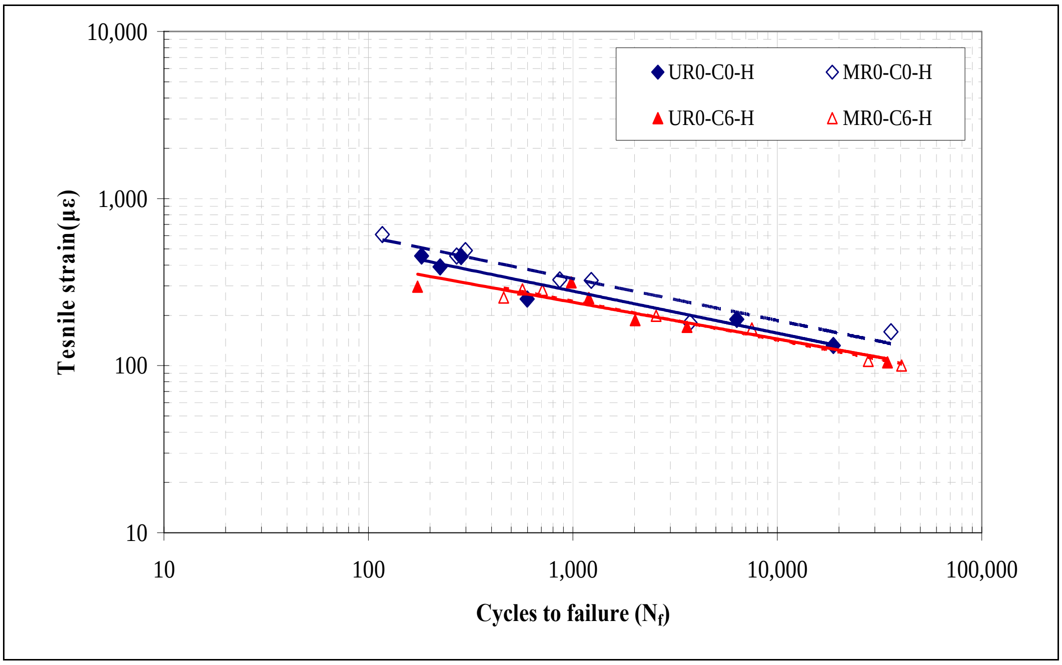


Figure 9.12: ITFT fatigue line for poorly compacted control (R0) mixtures tested in unconditioned state and after moisture conditioning

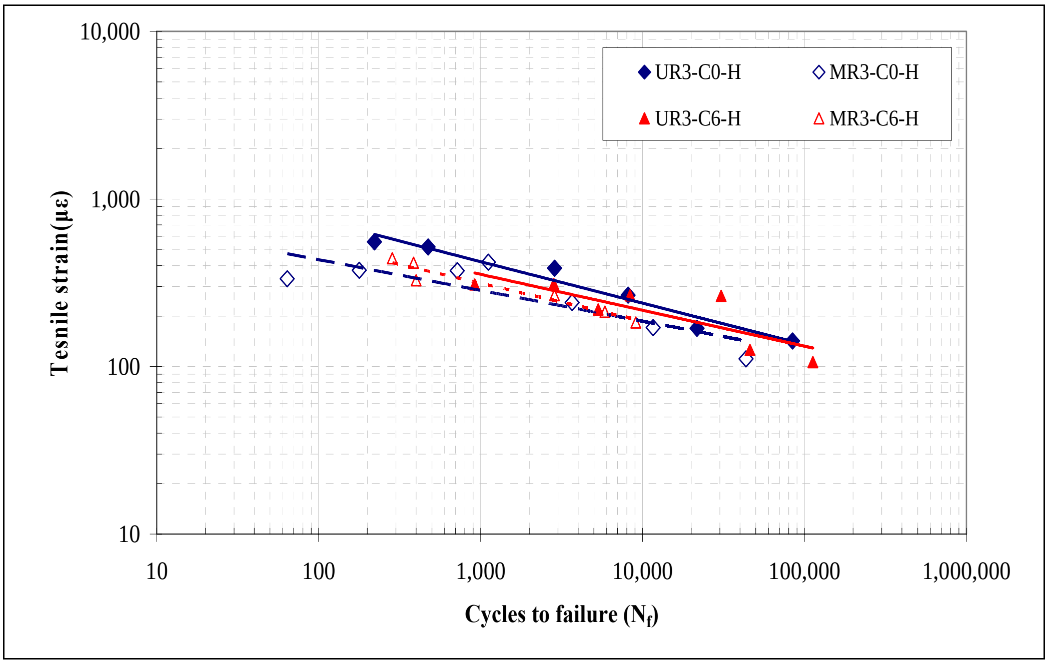


Figure 9.13 ITFT fatigue line for poorly compacted 3% CRM (R3) mixtures tested in unconditioned state and following moisture conditioning

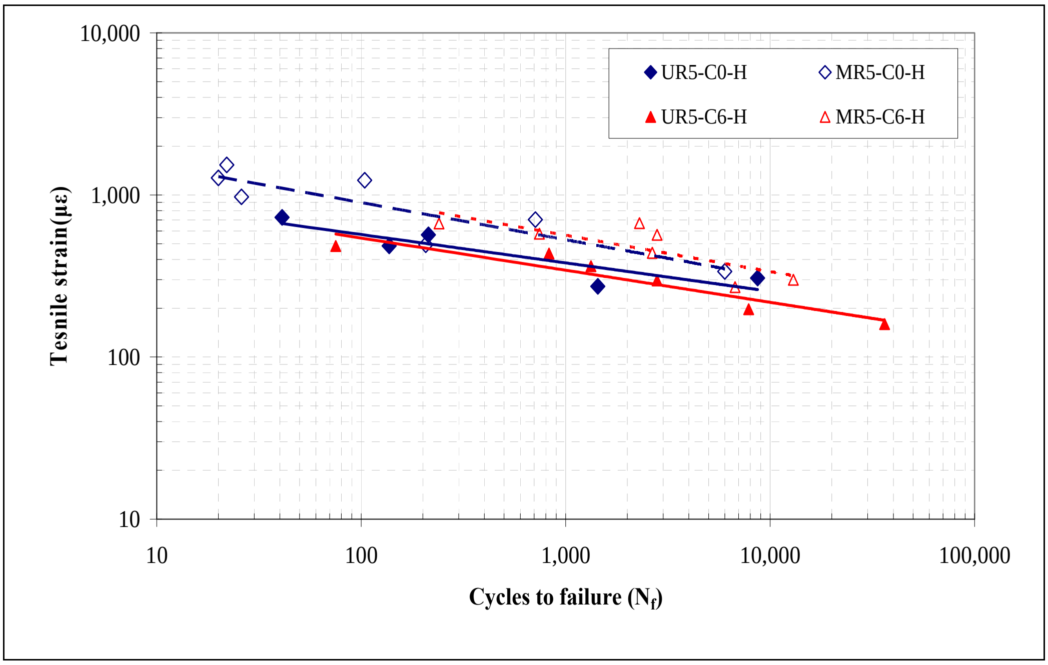


Figure 9.14: ITFT fatigue line for poorly compacted 5% CRM (R5) mixtures tested in unconditioned state and after moisture conditioning

Using the data presented in Figures 9.9 to 9.14, the regression lines in terms of initial strain and number of cycles to failure together with co-efficients of correlation for all mixture are presented in Table 9.5. With the exception of mixtures MR5-C6-L and MR5-C6-H, the fatigue and strain equations have all been established with a fairly high degree of confidence. The difficulty of obtaining consistent results in the indirect stiffness modulus test after two moisture conditioning cycles may have contributed to the scatter in initial strain calculations, and consequently the lower correlation.

Using the equations presented in Table 9.5, as mentioned earlier, the predicted fatigue lives for 200με and the predicted initial strains for 10,000 load cycles were calculated to study the effect of rubber content, short term conditioning and compaction effort and all the results were compared with their corresponding unconditioned state fatigue results calculated using the equations presented earlier in Table 8.6 in Chapter 8.

Table 9.5: Fatigue line equations for different mixtures with 4% and 8% target voids tested after moisture conditioning

Mixture	**Strain equation**	**Fatigue equation**	**R^2**
MR0-C0-L	$\varepsilon = 918xN_f^{-0.25}$	$N_f = 2.0x10^{11}\varepsilon^{-3.70}$	0.91
MR0-C6-L	$\varepsilon = 1167xN_f^{-0.20}$	$N_f = 8.0x10^{13}\varepsilon^{-4.41}$	0.89
MR3-C0-L	$\varepsilon = 2033xN_f^{-0.38}$	$N_f = 1.0x10^{9}\varepsilon^{-2.24}$	0.99
MR3-C6-L	$\varepsilon = 1614xN_f^{-0.20}$	$N_f = 2.0x10^{14}\varepsilon^{-4.41}$	0.82
MR5-C0-L	$\varepsilon = 7353xN_f^{-0.38}$	$N_f = 1.0x10^{9}\varepsilon^{-2.24}$	0.86
MR5-C6-L	$\varepsilon = 1787xN_f^{-0.20}$	$N_f = 8.0x10^{8}\varepsilon^{-2.40}$	0.49
MR0-C0-H	$\varepsilon = 1865xN_f^{-0.25}$	$N_f = 2.0x10^{12}\varepsilon^{-3.68}$	0.92
MR0-C6-H	$\varepsilon = 1217xN_f^{-0.23}$	$N_f = 9.0x10^{12}\varepsilon^{-4.16}$	0.97
MR3-C0-H	$\varepsilon = 1011xN_f^{-.25}$	$N_f = 6.0x10^{12}\varepsilon^{-3.68}$	0.73
MR3-C6-H	$\varepsilon = 1483xN_f^{-0.22}$	$N_f = 2.0x10^{16}\varepsilon^{-4.15}$	0.93
MR5-C0-H	$\varepsilon = 2580xN_f^{-0.23}$	$N_f = 9.0x10^{11}\varepsilon^{-3.36}$	0.77
MR5-C6-H	$\varepsilon = 2652xN_f^{-0.22}$	$N_f = 7.0x10^{10}\varepsilon^{-2.81}$	0.63

9.4.1.1 Rubber Content

The relative performance of the control and CRM mixtures is presented in Table 9.6. In terms of predicted strain, relatively similar strains can be observed for both the unconditioned and moisture conditioned control mixtures, although approximately 30% increase in stiffness for the highly compacted mixtures and 13% reduction in stiffness for poorly compacted mixtures were observed following moisture conditioning.

Table 9.6: Fatigue life comparison as a function of rubber content

Mixture	Strain @ 10^4 cycles (με)						Cycles @ 200 με (in thousand)					
	0% Rubber		3% Rubber		5% Rubber		0% Rubber		3% Rubber		5% Rubber	
	U	M	U	M	U	M	U	M	U	M	U	M
C0-L	126	95	222	185	249	215	3	1	17	7	16	7
C6-L	179	178	161	254	195	277	7	6	2	18	9	3
C0-H	156	187	239	186	255	312	3	7	24	5	15	17
C6-H	144	142	217	187	218	337	2	2	15	6	15	24

U: Unconditioned state, M: Moisture conditioned state

On the other hand, irrespective of compaction effort, the predicted strain is considerably higher in the moisture conditioned CRM mixtures compared to control mixtures. As a result of the significant reduction in stiffness modulus, the strain is generally higher in moisture conditioned 5% CRM mixtures than corresponding 3% CRM mixtures. In terms of predicted fatigue life at 200 microstrain, with the exception of MR0-C0-L, all the other control mixtures showed relatively similar fatigue performance compared to their unconditioned state. In contrast, five out of eight CRM mixtures showed significant reduction of fatigue life indicating that moisture in general has an adverse effect on the fatigue performance. The increased fatigue lives for MR5-C0-H and MR5-C6-H at 200 με compared to their corresponding unconditioned mixtures actually become lower when the predicted life is calculated at 100με. For example, at 100με, the predicted lives for UR5-C0-H and UR5-C6-H were 385,000 and 273,000, whereas they reduced to 172,000 and 169,000 for the corresponding moisture conditioned mixtures. Furthermore, in general, in terms of predicted life, despite the fact that CRM mixtures are adversely affected by moisture conditioning, their relative fatigue performance is still superior to that of the corresponding control mixtures.

9.4.1.2 Short-Term Ageing

The results presented in Table 9.7 indicate that compared to the unconditioned state results, the predicted number of cycles to failure at a strain of 200με is generally lower in both moisture conditioned C0 and C6 mixtures.

Table 9.7: Fatigue life comparison as a function of short-term conditioning

Mixture	**Strain @ 10^4 cycles (με)**				**Cycles @ 200 με(in thousands)**			
	0 hrs oven ageing		**6 hrs oven ageing**		**0 hrs oven ageing**		**6 hrs oven ageing**	
	U	**M**	**U**	**M**	**U**	**M**	**U**	**M**
R0-L	126	95	179	178	3	1	7	6
R3-L	222	185	161	254	17	7	2	18
R5-L	249	215	195	277	16	7	9	3
R0-H	156	178	144	142	3	7	2	2
R3-H	239	254	217	187	24	5	15	6
R5-H	255	277	218	337	15	17	15	24

Similar results can be observed in predicted fatigue life where the numbers of load applications are variable among mixtures. In general, most highly and poorly compacted C0 and C6 CRM mixtures are adversely affected by moisture conditioning with some significant reductions in fatigue life compared to the corresponding unconditioned state. Therefore, compared to the effect of rubber content, the effect of the short-term ageing of the control and CRM mixtures does not appear to have any extra influence on the overall fatigue performance of moisture conditioned CRM mixtures.

9.4.1.3 Compaction Effort

Table 9.8 shows that irrespective of compaction effort, in most cases, the predicted fatigue lives deteriorated considerably with moisture conditioning compared to their corresponding unconditioned state.

Table 9.8: Fatigue life comparison as a function of voids contents

Mixture	**Strain @ 10^4 cycles (με)**				**Cycles @ 200 με(in thousands)**			
	4% air voids		**8% air voids**		**4% air voids**		**8% air voids**	
	U	**M**	**U**	**M**	**U**	**M**	**U**	**M**
R0-C0	126	95	156	178	3	1	3	7
R3-C0	222	185	239	254	17	7	24	5
R5-C0	249	215	255	277	16	7	15	17
R0-C6	179	178	144	142	7	6	2	2
R3-C6	161	254	217	187	2	18	15	6
R5-C6	195	277	218	337	9	3	15	24

Although predicted strain is generally higher in less compacted control mixtures, their relative fatigue performance appears to deteriorate in a similar manner after moisture conditioning. This is despite the increased stiffness modulus after moisture conditioning in the highly compacted control mixtures. On the other hand, the effect of moisture conditioning appears to be similar in both void contents CRM mixtures. However, the reduction of performance is predominantly due to the increased rubber content and variable stiffness of the mixtures than the compaction effort. The primary reason is the plucking of the rubber particles from the surface of the specimen (Figure 9.1) during six hours conditioning at 60^0C which allows water to enter in the mixture matrix, consequently reducing the mixture's cohesion in both highly and poorly compacted mixtures. However, based on the test results from this study, in terms of tensile strain and fatigue life, with a few exceptions, reduced compaction does not have any extra influence on the moisture conditioned CRM mixtures compared to the control mixtures.

9.4.2 Long-Term Oven Ageing

Figures 9.15 to 9.20 present ITFT results for the control and CRM mixtures subjected to long-term oven ageing. The results are plotted in terms of initial tensile strain and number of cycles to failure. In addition, ITFT test data for all the mixtures are included in Appendix F. A few general observations from the results are as follows:

- At high stress levels the fatigue life is reduced in CRM mixtures and measured strain is higher with increasing rubber content in the mixtures. On the other hand, at low stress levels, although C0-L CRM mixtures perform similar to corresponding control mixtures, the number of cycles to failure for other CRM mixtures is lower. These observations indicate that the likelihood of the combined effect of bitumen oxidation and rubber-bitumen interaction following long-term ageing contributes to increased brittleness of the mixture, consequently decreasing the fatigue performance.

- At lower strain levels, in the range of 30 to 200 $\mu\varepsilon$, which are more likely to be experienced in the field, the performance of CRM mixtures is slightly inferior in the C6-L and C6-H cases. The possible explanation for this is that extra bitumen hardening due to rubber-bitumen interaction at short-term and long-term oven ageing combined with the softening effect of rubber particles leads to a weaker matrix which results in reduced tensile properties.

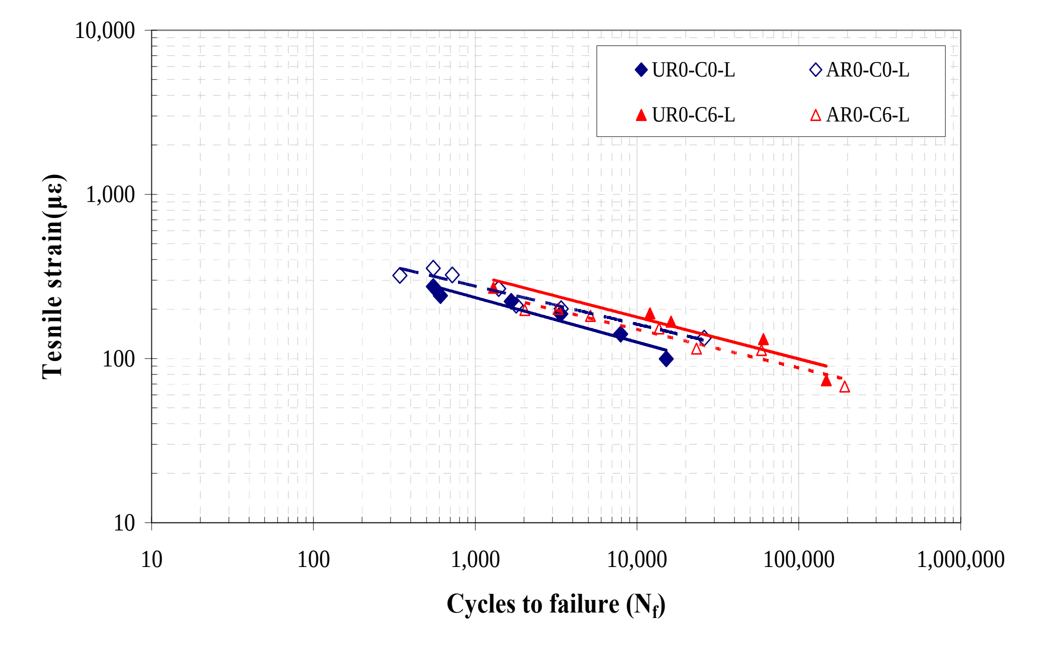


Figure 9.15: ITFT fatigue line for highly compacted control (R0) mixtures tested in unconditioned state and following long-term oven ageing

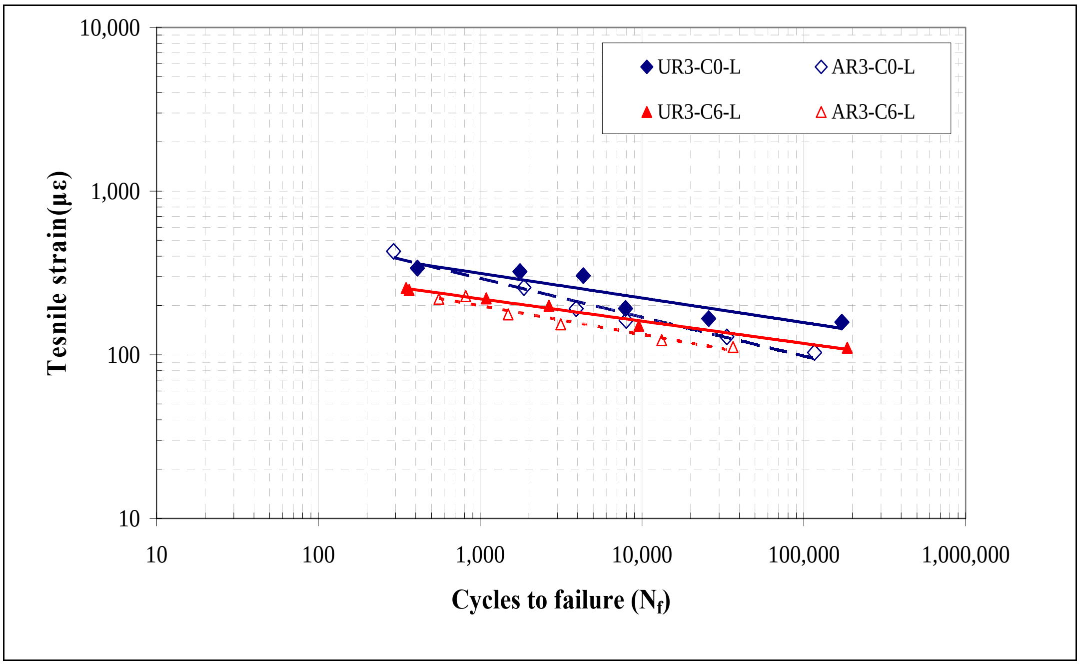


Figure 9.16: ITFT fatigue line for highly compacted 3% CRM (R3) mixtures tested in unconditioned state and following long-term oven ageing

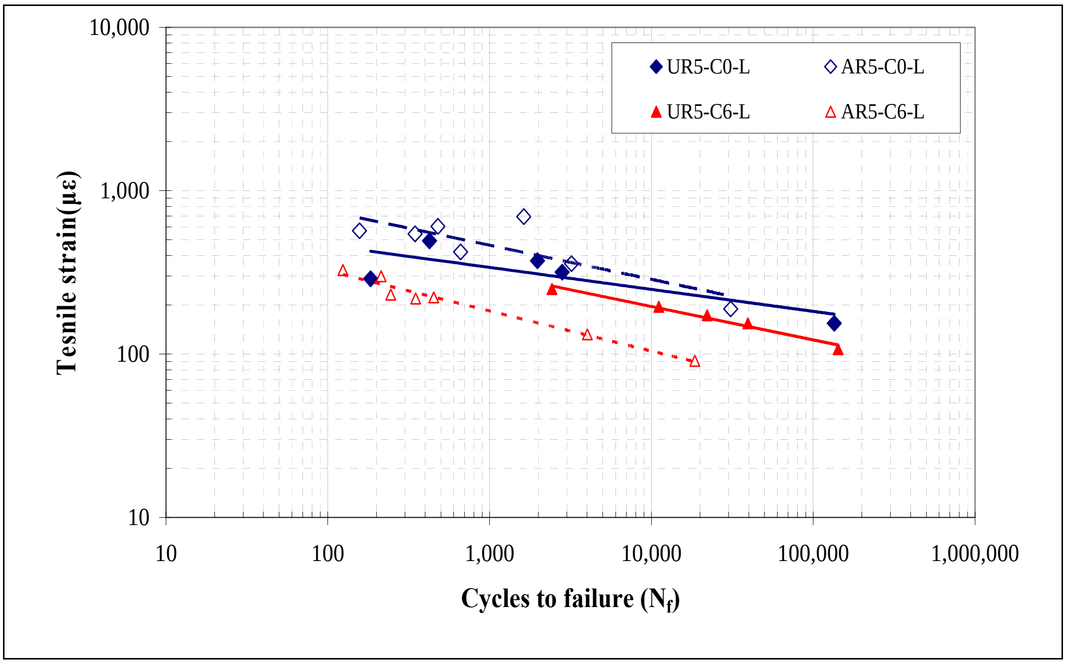


Figure 9.17: ITFT fatigue line for highly compacted 5% CRM (R5) mixtures tested in unconditioned state and following long-term oven ageing

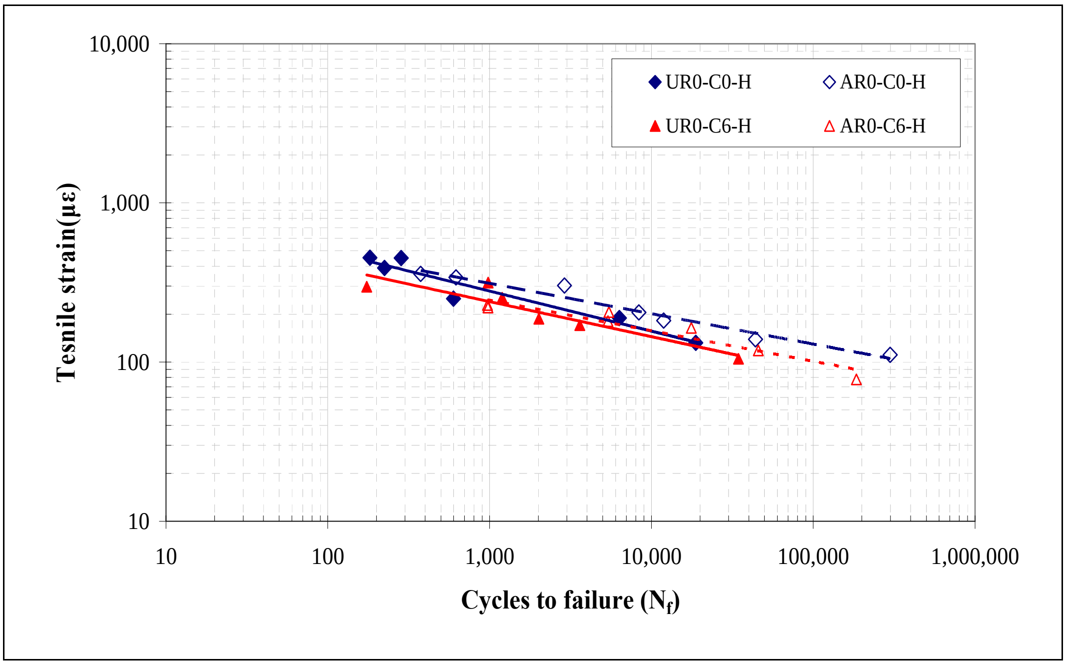


Figure 9.18: ITFT fatigue line for poorly compacted control (R0) mixtures tested in unconditioned state and following long-term oven ageing

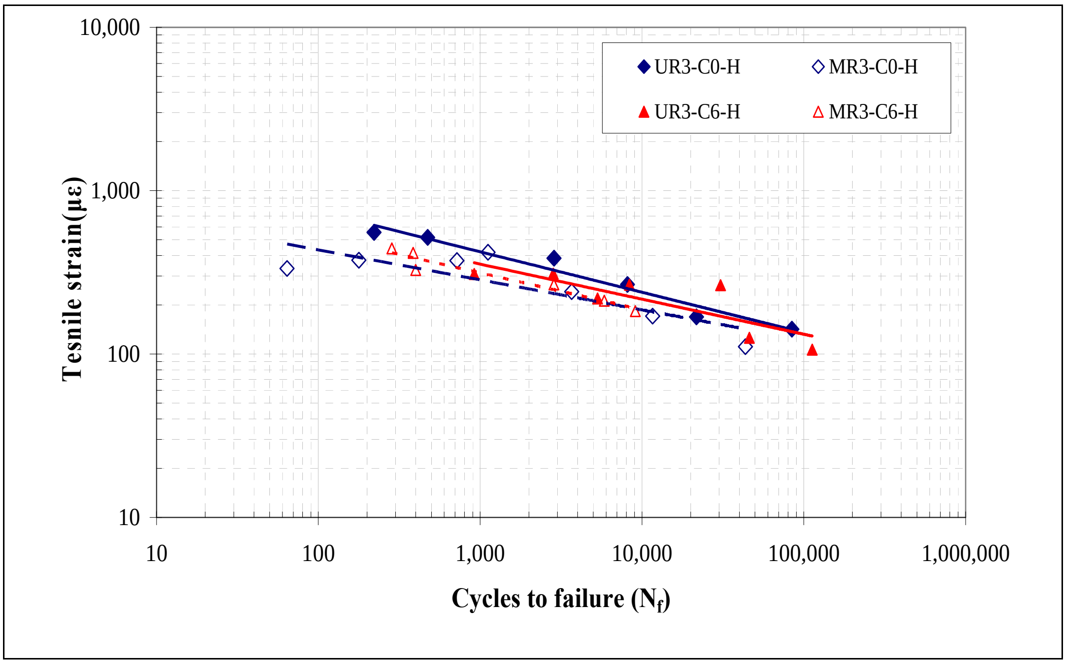


Figure 9.19: ITFT fatigue line for poorly compacted 3%CRM (R3) mixtures tested in unconditioned state and following long-term oven ageing

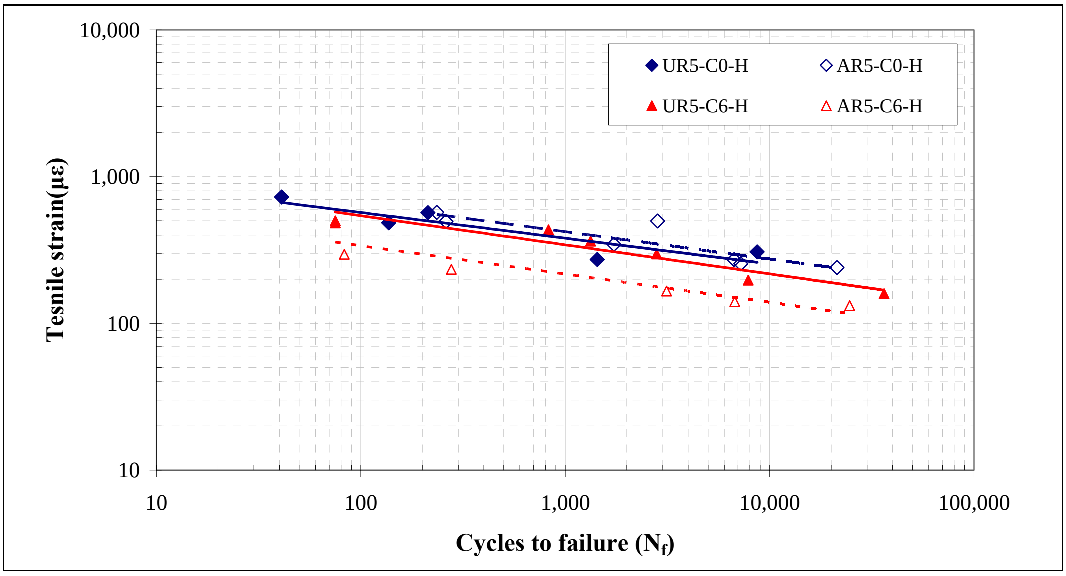


Figure 9.20: ITFT fatigue line for poorly compacted 5%CRM (R5) mixtures tested in unconditioned state and following long-term oven ageing

The strain and fatigue regression equations are presented in Table 9.9 and it can be seen that regression lines for all mixtures showed a high level of confidence. As with the unconditioned state, the predicted fatigue life for 100με and the predicted strain for one million load cycles were calculated using the equations in Table 9.9 to study the effect of rubber content, short-term conditioning and compaction effort and all results were compared with their corresponding unconditioned state results from equations in Table 8.6.

Table 9.9: Fatigue line equations for control and CRM mixture tested after long-term oven ageing

Mixture Type	Strain equation	Fatigue equation	R^2
AR0-C0-L	$\varepsilon = 1371xN_f^{-0.23}$	$N_f = 6.0x10^{12}\varepsilon^{-4.01}$	0.93
AR0-C6-L	$\varepsilon = 1306xN_f^{-0.24}$	$N_f = 6.0x10^{12}\varepsilon^{-4.02}$	0.94
AR3-C0-L	$\varepsilon = 1509xN_f^{-0.24}$	$N_f = 1.0x10^{13}\varepsilon^{-4.10}$	0.97
AR3-C6-L	$\varepsilon = 671xN_f^{-0.18}$	$N_f = 3.0x10^{15}\varepsilon^{-5.43}$	0.95
AR5-C0-L	$\varepsilon = 1944xN_f^{-0.22}$	$N_f = 1.0x10^{14}\varepsilon^{-4.22}$	0.93
AR5-C6-L	$\varepsilon = 999xN_f^{-0.25}$	$N_f = 9.0x10^{11}\varepsilon^{-3.95}$	0.97
AR0-C0-H	$\varepsilon = 1165xN_f^{-0.19}$	$N_f = 4.0x10^{15}\varepsilon^{-5.05}$	0.96
AR0-C6-H	$\varepsilon = 922xN_f^{-0.19}$	$N_f = 2.0x10^{14}\varepsilon^{-4.72}$	0.91
AR3-C0-H	$\varepsilon = 739xN_f^{-0.18}$	$N_f = 1.0x10^{15}\varepsilon^{-5.17}$	0.91
AR3-C6-H	$\varepsilon = 477xN_f^{-0.15}$	$N_f = 5.0x10^{16}\varepsilon^{-6.13}$	0.91
AR5-C0-H	$\varepsilon = 1504xN_f^{-.19}$	$N_f = 1.0x10^{16}\varepsilon^{-5.04}$	0.97
AR5-C6-H	$\varepsilon = 822xN_f^{-0.19}$	$N_f = 2.0x10^{13}\varepsilon^{-4.41}$	0.85

9.4.2.1 Rubber Content

The results presented in Table 9.10 show that with the exception of R5-C0-H, there are significant reductions in both predicted strains and fatigue lives of all aged CRM mixtures compared to their corresponding unconditioned state. This indicates that age hardening of the mixtures causes excessive embrittlement.

Table 9.10: Fatigue life comparison as a function of rubber content

Mixture	Strain @ 10^6 cycles (με)						Cycles @ 100 με (in thousands)					
	0% Rubber		3% Rubber		5% Rubber		0% Rubber		3% Rubber		5% Rubber	
	U	A	U	A	U	A	U	A	U	A	U	A
C0-L	36	56	111	57	134	93	20	57	706	64	416	366
C6-L	55	51	86	59	76	34	84	54	301	41	250	11
C0-H	49	84	76	66	114	106	44	321	281	46	385	828
C6-H	52	65	81	61	87	57	33	72	125	28	272	30

U: Unconditioned state, A: Long-term oven aged state

Three out of the four control mixtures (only exception is C6-L) have shown increased predicted strain and fatigue for both the highly and poorly compacted cases. The results indicate that age hardening of control mixture leads to better long-term fatigue performance. On the other hand, although all the mixtures were produced using the same percentage of bitumen using the same manufacturing technique, the flexible rubber particles are believed to be the main cause of absorption of the lighter fractions of bitumen during long-term ageing which consequently leads to a stiffer material and increased brittleness.

9.4.2.2 Short-Term Ageing

Table 9.11 shows that as with the effect of rubber content, the overall long-term fatigue performance of the short-term aged CRM mixtures appear to be reduced. In general, the predicted strains for the aged CRM mixtures are reduced compared to their unconditioned state which indicate the consequence of the increased stiffness after long-term ageing. Compared with the unconditioned state, three out of four CRM mixtures showed more reduction in predicted strain in C6 conditioning state against C0 state (only exception is R3-L).

Table 9.11: Fatigue life comparison as a function of short-term conditioning

Mixture	**Strain @ 10^6 cycles (με)**				**Cycles @ 100 με(in thousand)**			
	0 hrs conditioning		**6 hrs conditioning**		**0 hrs conditioning**		**6 hrs conditioning**	
	U	**A**	**U**	**A**	**U**	**A**	**U**	**A**
R0-L	36	56	55	51	20	57	84	54
R0-H	49	84	52	65	44	321	33	72
R3-L	111	57	86	59	706	64	301	41
R3-H	76	66	81	61	281	46	125	28
R5-L	134	93	76	34	416	366	250	11
R5-H	114	106	87	57	385	828	272	30

In terms of predicted fatigue life, ageing appears to reduce the overall long-term fatigue performance of both C0 and C6 CRM mixtures although for the control mixtures it appears that fatigue life increases with ageing. The results are similar for highly and poorly compacted mixtures. In addition, generally the reduction of fatigue life is greater for six hours ageing (C6) than without ageing (C0) for the

CRM mixtures, suggesting that in addition to normal oxidation, more absorption of lighter fractions reduces the cohesive and adhesive strength of the bitumen and consequently increases the brittleness of the material.

9.4.2.3 Compaction Effort

With the exception of R5-C0-H, irrespective of compaction effort, compared to the unconditioned state, all the CRM mixtures showed a significant reduction in fatigue life as a result of long-term oven ageing. In addition, compared to the effect of rubber content and short-term ageing, the results presented in Table 9.12 indicate that long-term ageing of CRM mixtures with 4 and 8% air void content are similarly affected with a uniform reduction of predicted strain and fatigue life being observed compared to their unconditioned mixtures.

On the other hand, the fatigue performance of the control mixtures appears to be better in both void contents indicating that hardening of bitumen would increase fatigue performance. Therefore, in terms of long term ageing, the issue of under compaction (high air void content) did not appear to have any greater significance on the CRM mixtures than it has on the control mixtures.

Table 9.12: Fatigue life comparison as a function of voids contents

Mixture	**Strain @ 10^6 cycles ($\mu\varepsilon$)**				**Cycles @ 100 $\mu\varepsilon$ (in thousand)**			
	4% air voids		**8% air voids**		**4% air voids**		**8% air voids**	
	U	**A**	**U**	**A**	**U**	**A**	**U**	**A**
R0-C0	36	56	49	84	20	57	44	321
R0-C6	55	51	52	65	84	54	33	72
R3-C0	111	57	76	66	706	64	281	46
R3-C6	86	59	81	61	301	41	125	28
R5-C0	134	93	114	106	416	366	385	828
R5-C6	76	34	87	57	250	11	272	30

9.5 PERMANENT DEFORMATION

The resistance to permanent deformation was carried out on all mixtures after moisture conditioning and long-term oven ageing. The tests were conducted using the same testing protocol outlined in Section 8.4.1 Chapter 8. The results were analysed for individual mixtures subjected to moisture conditioning and long-term oven ageing and the results were compared with corresponding unconditioned state test results and presented in the following sections. It is important to note that similar expansion was once again observed during the two hours high temperature (60^0C) conditioning prior to permanent deformation testing (see Figure 8.14).

9.5.1 Moisture Conditioning

Typical plots of cumulative irrecoverable axial strain versus loading time for individual samples of MR0-C6-L, MR3-C6-L and MR5-C6-L are presented in Figures 9.21 to 9.23. Similar plots for all other mixtures are included in Appendix G. It should be reported that some specimens from the 5% CRM mixtures became very fragile following moisture conditioning that resulted in specimen failure under loading and these results were, therefore, omitted from the comparison. The summary of average percentage of total axial strain for all the mixtures is presented in Table 9.13.

Table 9.13: Total strain for all mixtures subjected to moisture conditioning

Mixture	Total strain (%)	
	avg	stdev
MR0-C0-L	1.54	0.7
MR0-C6-L	1.34	0.2
MR3-C0-L	4.42	0.1
MR3-C6-L	5.68	0.3
MR5-C0-L	12.11	1.2
MR5-C6-L	10.58	3.1
MR0-C0-H	3.34	0.4
MR0-C6-H	1.53	0.4
MR3-C0-H	6.97	0.5
MR3-C6-H	7.28	0.3
MR5-C0-H	12.27	1.0
MR5-C6-H	8.87	0.8

A list of general observations from Figures 9.21 to 9.23 and Table 9.13 is presented below;

- The average percentage of total permanent strain for MR0-C6-L is approximately 1.34%, which is considerably lower than the MR3-C6-L and MR5-C6-L mixtures where 5.68% and 10.6% permanent strain was generated after 3600 load pulses. Similar results can also be observed for the other mixtures which indicate that the moisture conditioned CRM mixtures have less resistance to high temperature deformation than the control mixtures. The lower stiffness of the moisture conditioned CRM material, high initial deformation and rough top surface are believed to be the main factors generating these high permanent strains.

- Although short-term ageing appears to improve rutting resistance in the control mixtures, no noticeable difference can be seen in both CRM mixtures.

- Lower compaction seems to reduce the rutting resistance in the control mixtures, but no effect can be observed in the CRM mixtures.

- The permanent strain for the 5% CRM mixtures is considerably higher than the 3% CRM mixtures. This indicates that 5% CRM mixtures are more susceptible to moisture induced damage.

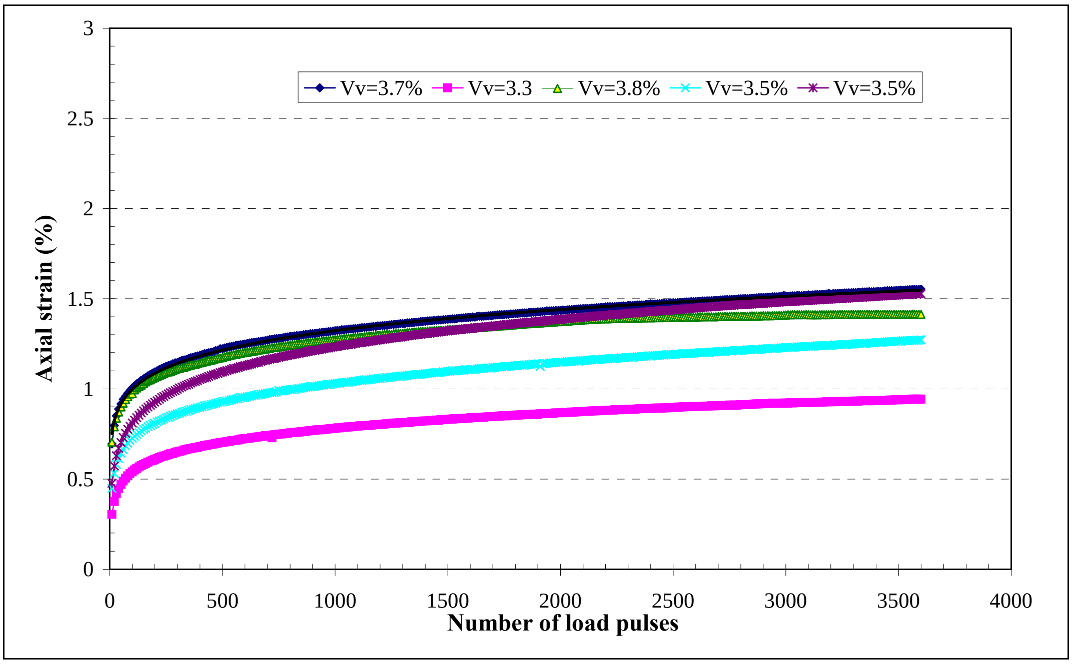


Figure 9.21: CRLAT test results of R0-C6-L mixture tested after moisture conditioning with 70kPa confinement at 60^{0}C

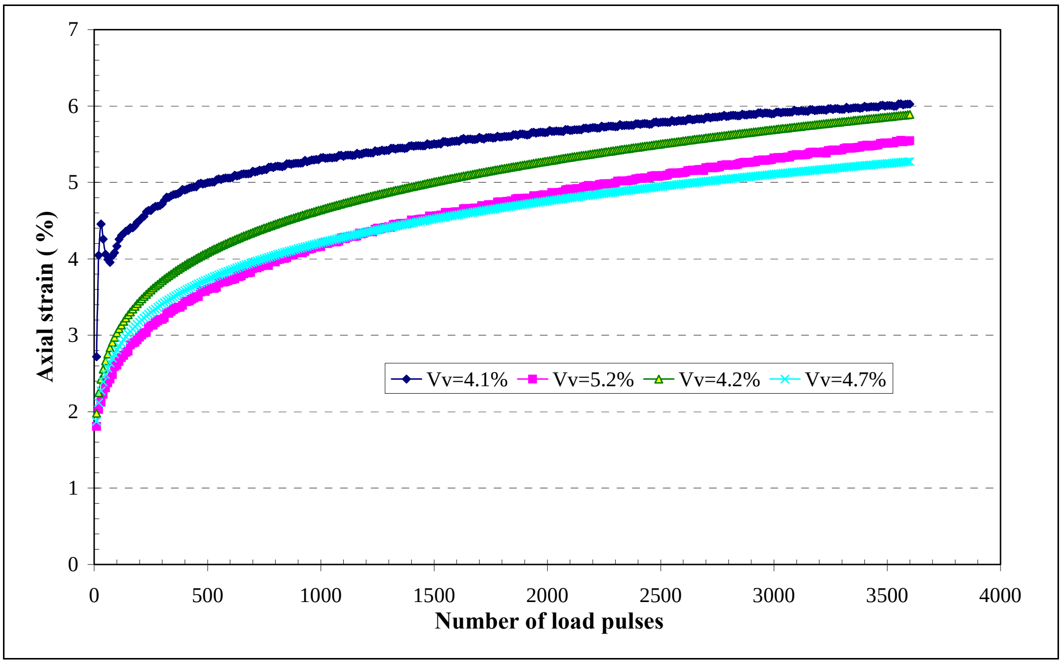


Figure 9.22: CRLAT test results of R3-C6-L mixture tested after moisture conditioning with 70kPa confinement at 60^{0}C

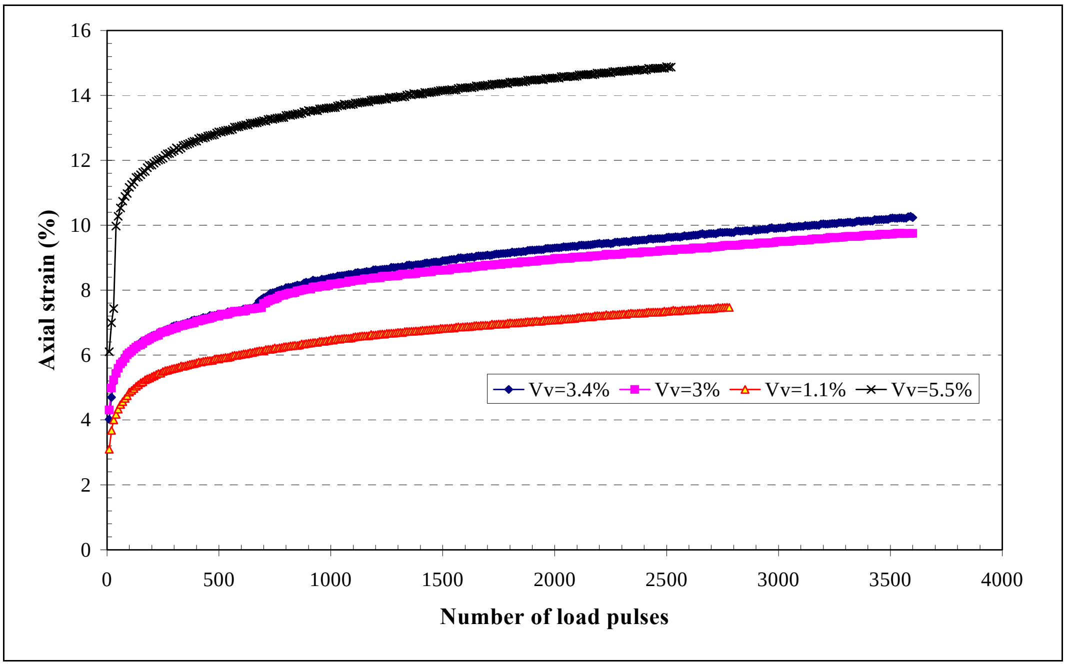


Figure 9.23: CRLAT test results of R5-C6-L mixture tested after moisture conditioning with 70kPa confinement at 60^{0}C

As with the unconditioned state, the moisture conditioned deformation results were analysed more critically by calculating mean and minimum strain rates using equations 8.1 and 8.2. The summary results of mean and minimum strain rate are listed in Table 9.14 and plotted in Figures 9.24 and 9.25. The results show that in general, compared to the unconditioned state testing (See Table 8.11 in Chapter 8), the resistance to permanent deformation for continuously graded DBM control mixtures decreases due to the reduction of the mixture's cohesive and adhesive properties following moisture conditioning. On the other hand, the magnitude of mean strain, influenced by the initial stage of testing, is significantly greater for CRM mixtures compared with their corresponding unconditioned state values. This is likely to be the combined consequence of highly elastic rubber particles, lower stiffness modulus of CRM mixtures after moisture ageing, initial slack in the test operation and the rough top surface of CRM specimens. Therefore, the mean strain rate parameter may not be a representative way to explain the permanent deformation behaviour.

Table 9.14: Summary of mean strain rate, minimum strain rate for all mixtures subjected to moisture conditioning

Mixture	**Mean strain rate (με/cycle)**		**Minimum strain rate (με/cycle)**	
	avg	**stdev**	**avg**	**stdev**
MR0-C0-L	41	24	1.1	0.5
MR0-C6-L	23	4	0.8	0.2
MR3-C0-L	71	13	4.8	1.0
MR3-C6-L	100	8	4.0	1.0
MR5-C0-L	301	121	8.6	3.7
MR5-C6-L	208	95	6.0	1.0
MR0-C0-H	75	9	4.5	0.5
MR0-C6-H	32	12	1.6	0.8
MR3-C0-H	144	14	7.0	1.1
MR3-C6-H	148	52	5.3	2.8
MR5-C0-H	218	45	7.1	2.6
MR5-C6-H	141	15	4.1	1.1

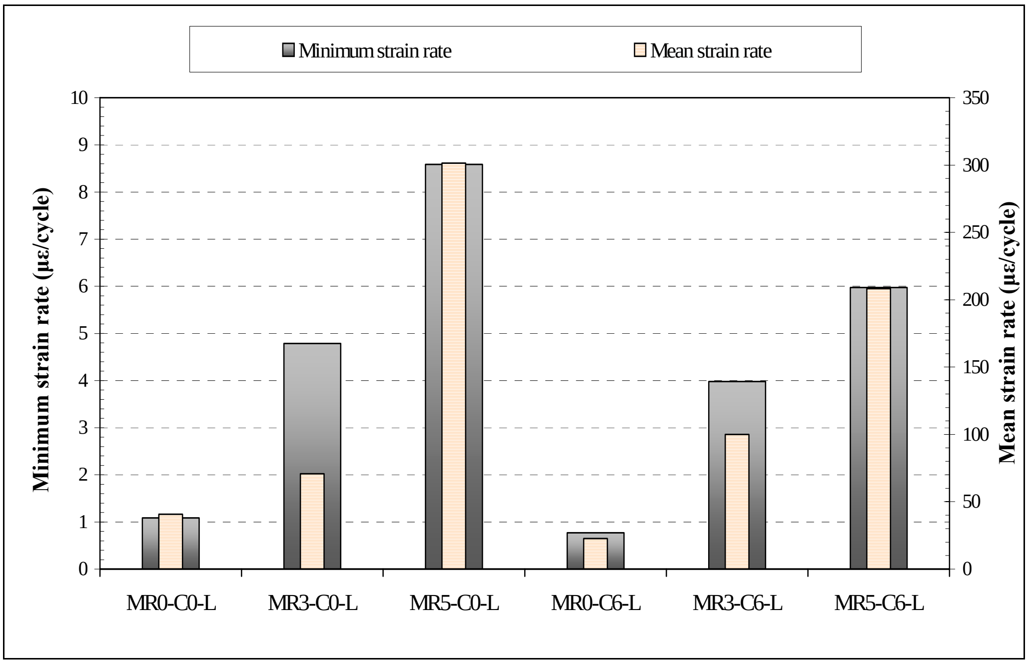


Figure 9.24: Minimum strain rate and mean strain rate of all highly compacted mixture subjected to moisture conditioning

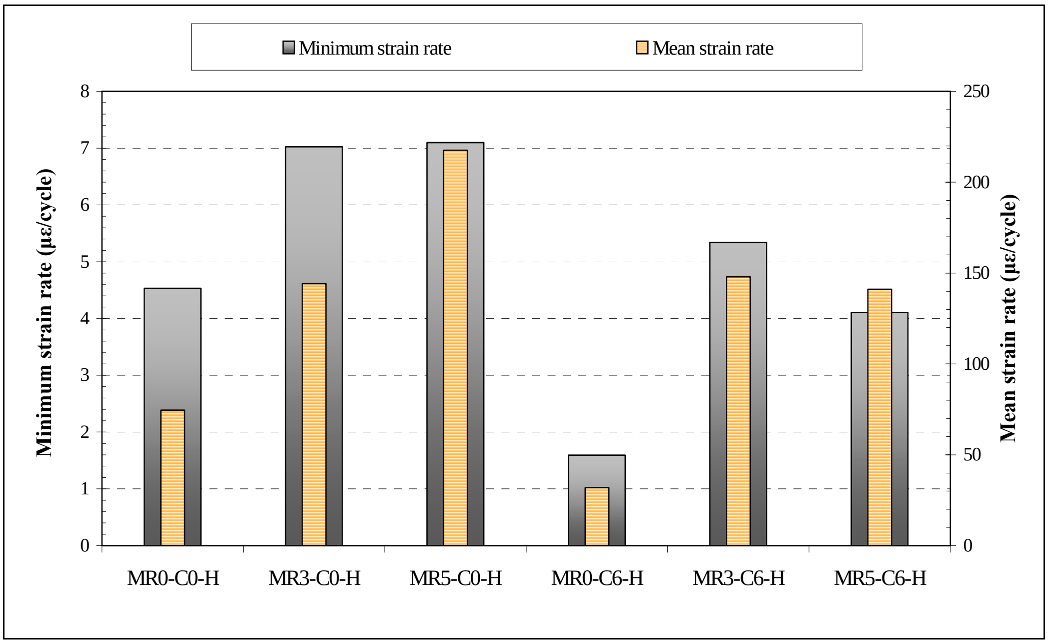


Figure 9.25: Minimum strain rate and mean strain rate of all low compacted mixtures subjected to moisture conditioning

Considering all these factors, the minimum strain rate parameter which represents the response in the steady state part (see page 189), can be considered to be a more reliable and accurate means of assessing the permanent deformation of the dry process CRM asphalt mixtures and is exclusively used in the following sections to compare to the results obtained from unconditioned specimens. In the following subsections, the results are interpreted and discussed in terms of the effect of rubber content, short-term ageing and compaction effort.

9.5.1.1 Rubber Content

The minimum strain rates for the control and CRM mixtures tested in their unconditioned and moisture conditioned state are presented in Table 9.15. The percentage changes relative to the control mixtures are also included to compare the performance before and after moisture conditioning. In general, irrespective of short-term ageing of the loose mixture, moisture conditioning appears to reduce the rutting resistance of both highly and poorly compacted control mixtures.

Table 9.15: Relative permanent deformation performance of CRM mixtures subjected to unconditioned and moisture conditioned test

Minimum strain rate (με/cycle)							Percentage change (%)			
Mixture	**0% rubber**		**3% rubber**		**5% rubber**		**3% rubber**		**5% rubber**	
	U	**M**	**U**	**M**	**U**	**M**	**U**	**M**	**U**	**M**
C0-L	0.61	1.1	2.09	4.8	1.27	8.6	243	336	108	682
C6-L	0.23	0.8	3.86	4.0	1.75	6.0	1578	400	661	650
C0-H	3.84	4.5	2.93	7.0	3.81	7.1	-24	56	-1	58
C6-H	2.61	1.6	3.05	5.3	1.85	4.1	17	231	-29	156

In the case of CRM mixtures, the strain rate for moisture conditioned R3 mixtures increases as high as 4.8με/cycle (compared to 2.09με/cycle in the unconditioned state) for the highly compacted case and 7 με/cycle (compared to 2.93με/cycle in the unconditioned state) for the poorly compacted case. The performance of the moisture conditioned 5% CRM mixtures are even more seriously deteriorated. Although three out of four 5% CRM mixtures performed better compared to the 3% CRM mixtures when tested in the unconditioned state, there is a large reduction in rutting resistance in the 5% CRM mixtures following moisture conditioning. In addition, compared to the unconditioned state test results the percentage changes relative to the control mixtures are also generally inferior following moisture conditioning, indicating that CRM mixtures are more susceptible to moisture damage, therefore, less durable in the field.

9.5.1.2 Short-Term Ageing

The effect of short-term ageing on the permanent deformation performance in terms of minimum strain rate of the control and CRM mixtures that were subjected to testing in the unconditioned and moisture conditioned states are shown in Table 9.16. In general, irrespective of compaction effort, both C0 and C6 mixtures undergo a reduction in rutting resistance due to moisture conditioning. For highly compacted C0 control mixtures, the increase of strain rate is around 80% in the moisture conditioned state, while in C6 mixtures it was 248%.

Table 9.16: Permanent deformation performance of short term conditioned control and CRM asphalt mixtures subjected to unconditioned and moisture conditioning

Mixture	**Minimum strain rate (με/cycle)**				**Percentage change compared to unconditioned state (%)**	
	0 hours conditioning		**6 hrs conditioning**		**0 hour**	**6 hour**
	U (a)	***M (b)***	***U(c)***	***M (d)***	***(a-b)/a***	***(c-d)/c***
R0-L	0.61	1.1	0.23	0.8	-80	-248
R0-H	3.84	4.5	2.61	1.6	-17	39
R3-L	2.09	4.8	3.86	4.0	-130	-4
R3-H	2.91	7.0	3.05	5.3	-141	-74
R5-L	1.27	8.6	1.75	6.0	-577	-243
R5-H	3.81	7.1	1.85	4.1	-86	-122

But the figures for the poorly compacted control mixtures were a different story, with a 39% decrease in strain rate (improved rutting resistance) in the C6 mixtures compared to unconditioned state testing. This is contradictory to the earlier stiffness results, where moisture conditioning leads to a reduction of stiffness for all poorly compacted control mixtures. It is generally believed that less compaction is one of the primary reasons that lead to higher permanent deformation.

Evaluating 3% CRM mixtures against 5% CRM mixtures, irrespective of short-term ageing, the increase of strain rates is significantly higher for 5% CRM mixtures than 3% CRM mixtures in both void contents. It appears that for individual CRM mixtures, the percentage reduction in three out of four short-term aged mixtures (C6) is slightly lower compared to without short-term aged (C0) mixtures. The possible explanation is that the short-term aged mixtures are slightly stiffer than unaged (0 hour ageing prior to compaction) mixtures which leads to a marginally better rutting resistance. Therefore, compared to the effect of rubber contents in the mixtures, the marginal improvement of rutting resistance in short-term aged mixtures seems to have not much influence on the overall rutting performance of the moisture conditioned CRM mixtures. The results indicate that mixtures with more rubber contents are more susceptible to moisture damage, therefore less durable to resist permanent deformation during their service life.

9.5.1.3 Compaction Effort

The effect of compaction effort on the permanent deformation resistance following moisture conditioning is shown in Table 9.17. For each mixture type, the percentage change was calculated by comparing the strain rates from mixtures with 4% and 8% voids content tested in their unconditioned state to the corresponding moisture conditioned state. In general, compared to the unconditioned state results, irrespective of compaction effort, with moisture conditioning the resistance to permanent deformation decreased considerably for both control and CRM mixtures.

Table 9.17: Comparison of permanent deformation performance of control and CRM asphalt mixtures subjected to unconditioned and moisture conditioned testing

Mixture	Minimum strain rate (με/cycle)				Percentage change compared to unconditioned state (%)	
	4% air voids		8% air voids		4% voids	8% voids
	U (a)	*M(b)*	*U (c)*	*M (d)*	*(a-b)/a*	*(c-d)/c*
R0-C0	0.61	1.1	3.84	4.5	-80	-17
R0-C6	0.23	0.8	2.61	1.6	-248	39
R3-C0	2.09	4.8	2.91	7.0	-130	-141
R3-C6	3.86	4.0	3.05	5.3	-4	-74
R5-C0	1.27	8.6	3.81	7.1	-577	-86
R5-C6	1.75	6.0	1.85	4.1	-243	-122

For example, the strain rate is increased by as much as 248% in the control mixtures, 141% increase in R3 and 577% in R5 mixtures. In addition, although the decrease is slightly lower with increasing voids contents (decreased compaction) in the control mixtures, in general, the effect of moisture conditioning seems to have a similar effect for higher and lower voids content CRM mixtures. Therefore, it can be said that although resistance to permanent deformation decreased considerably for the control mixtures as a result of increasing void content (decreased compaction) the relative effect of compaction is considerably lower on CRM mixtures.

9.5.2 Long Term Oven Ageing

The cumulative irrecoverable axial strain versus loading time for the individual samples of AR0-C6-L, AR3-C6-L and AR5-C6-L mixtures are presented in Figures 9.26 to 9.28. Similar plots for all other mixtures are included in Appendix G. In addition, the percentage of total axial strain for all mixtures is presented in Table 9.18.

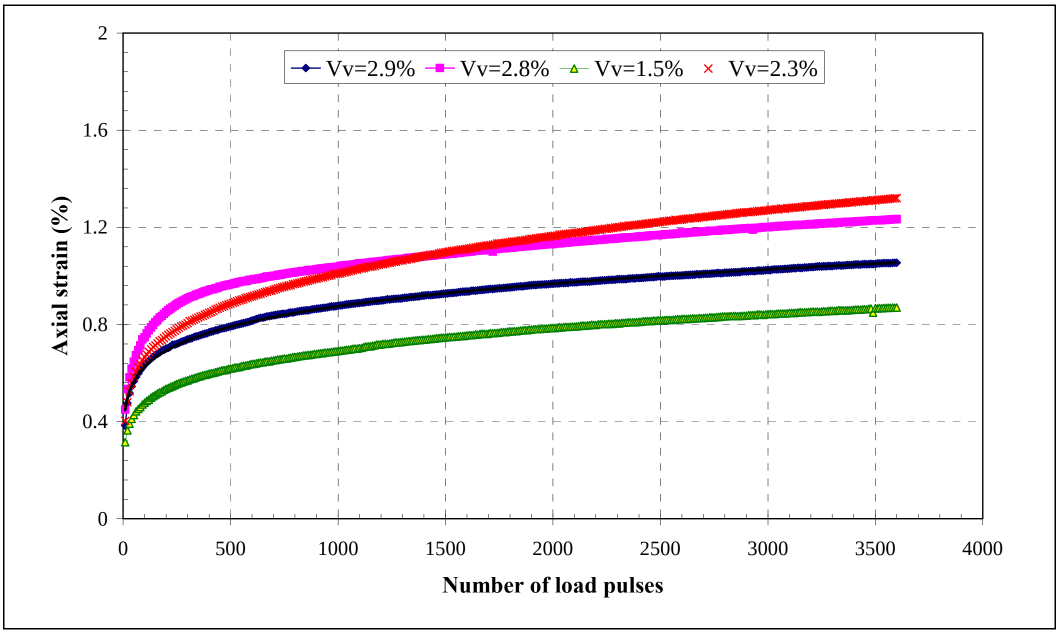


Figure 9.26: CRLAT test results of R0-C0-L mixture tested after long-term oven conditioning with 70kPa confinement at 60^{0}C

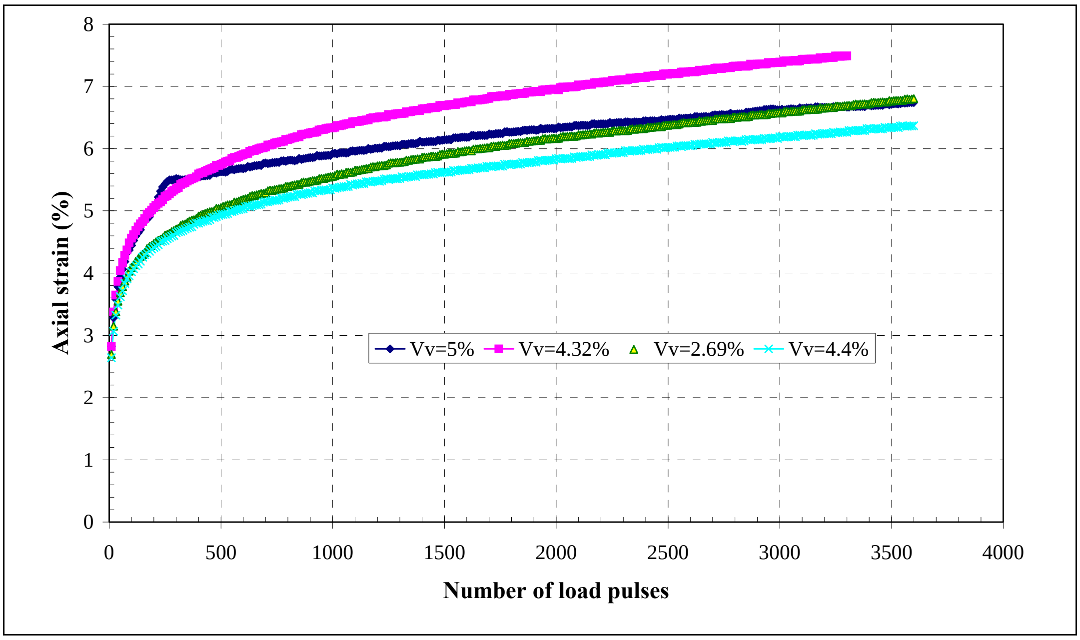


Figure 9. 27: CRLAT test results of R3-C0-L mixture tested after long-term oven conditioning with 70kPa confinement at 60^{0}C

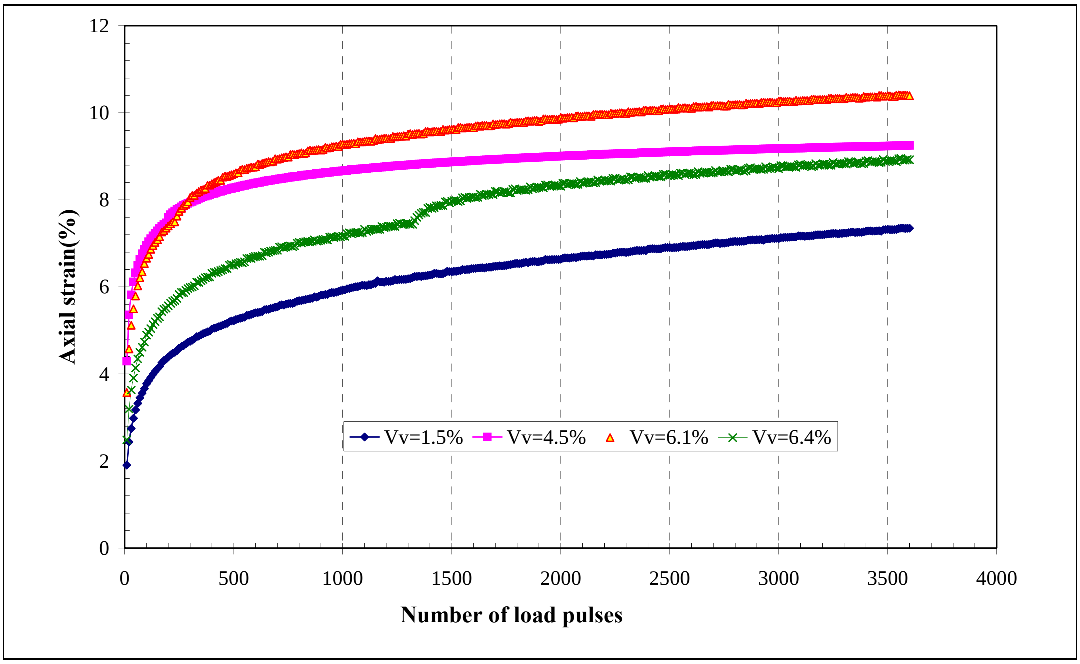


Figure 9. 28: CRLAT test results of R5-C0-L mixture tested after long-term oven conditioning with 70kPa confinement at 60^{0}C

Table 9.18: Average percentage of total strain for all mixtures subjected to long-term ageing

Mixture	Total strain (%)	
	avg	stdev
AR0-C0-L	1.12	0.2
AR0-C6-L	1.10	0.2
AR3-C0-L	6.85	0.5
AR3-C6-L	4.30	1.0
AR5-C0-L	9.00	1.5
AR5-C6-L	4.3	1.3
AR0-C0-H	1.57	0.3
AR0-C6-H	0.96	0.4
AR3-C0-H	5.04	0.8
AR3-C6-H	4.31	2.1
AR5-C0-H	9.96	1.3
AR5-C6-H	11.49	0.7

Based on the results presented above, a list of observations is listed below;

- Similar to the unconditioned and moisture conditioned results, in the long-term aged mixtures, the percentage of total strain generally increases with increasing rubber content indicating that the mixtures with higher percentages of rubber are more susceptible to high temperature oven ageing.

- Permanent deformation resistance appears to be slightly inferior for the aged control and CRM mixtures compared to their unconditioned state test results presented in Table 8.11. The average permanent strain for both R3-C6-L and R5-C6-L mixtures is 4.30% (3.75% in UR3-C6-L and 4.74% in UR5-C6-L) against 1.10% (0.40% in UR0-C6) for the control mixtures. Similar results, with the exception of R0-C6-H, were also observed in poorly compacted mixtures where long-term ageing results reduced the resistance to permanent deformation.

- It is interesting to note that, compared to moisture conditioned state results (Table 9.13), the lower permanent strain for long-term aged control and CRM mixtures demonstrates that migration of water into the asphalt mixture matrix leads to a weaker bonding compared to oxidation of bitumen at high temperature ageing.

- The results indicate that for the poorly compacted control mixture, long-term ageing leads to bitumen hardening of the mixtures, which consequently increases the permanent deformation resistance. For CRM mixtures, it is likely that the combined effect of extra bitumen hardening due to rubber-bitumen interaction and softening effect of rubber due the absorption of the lighter fractions of the bitumen following long-term ageing results in a reduction of resistance to permanent deformation.

As with the unconditioned state, the long-term aged conditioned deformation results were also analysed more critically by calculating mean and minimum strain rates using equations 8.1 and 8.2. The summary results of mean and minimum strain rate are listed in Table 9.19 and plotted in Figures 9.29 and 9.30. In terms of mean strain rate, comparing results from Table 9.19 against Table 8.11, the long-term aged modified DBM mixtures through the addition of crumb rubber show an increase in mean strain rate for both low and high voids content mixtures while the strain rates for less compacted unconditioned mixtures were relatively comparable.

Table 9.19 Mean strain rate and minimum strain rate strain for all mixtures subjected to long- term ageing

Mixture	Mean strain rate (με/cycle)		Minimum strain rate (με/cycle)	
	avg	stdev	avg	stdev
AR0-C0-L	20	4.3	0.8	0.2
AR0-C6-L	15	2.8	0.40	0.2
AR3-C0-L	117	16.9	4.1	0.7
AR3-C6-L	71	18.7	2.3	0.7
AR5-C0-L	160	26.9	3.9	1.6
AR5-C6-L	91	30.0	4.9	1.8
AR0-C0-H	23	9.8	1.0	0.4
AR0-C6-H	19	8.7	0.6	0.3
AR3-C0-H	104	25.4	5.9	1.5
AR3-C6-H	94	41.0	4.0	1.9
AR5-C0-H	228	87.7	8.6	3.0
AR5-C6-H	188	10.7	5.1	1.4

It is also interesting to note that the mean strain rate for all long-term aged C6 mixtures is considerably lower than corresponding C0 mixtures. Similar findings were also observed in unconditioned state testing where four out of six C6 mixtures showed less mean strain compared to C0 mixtures. As mean strain rate indicates material performance at the early stage of testing where material densification occurs, the decreased rate for C6 mixtures indicates that increased stiffness due to bitumen hardening following short-term ageing may contribute to better rutting resistance.

However, as explained in Section 9.6.1, the interpretation of mean strain rate may not be appropriate for CRM mixtures to assess the permanent deformation performance due to the following reasons;

- Initial slack of the testing apparatus due to the rough top surface of the CRM specimen where rubber particles were exposed.
- Highly elastic nature of the rubber particles with low stiffness of the CRM mixtures after high temperature conditioning.
- Any delayed elastic response that cannot be recovered during the one second recovery period.

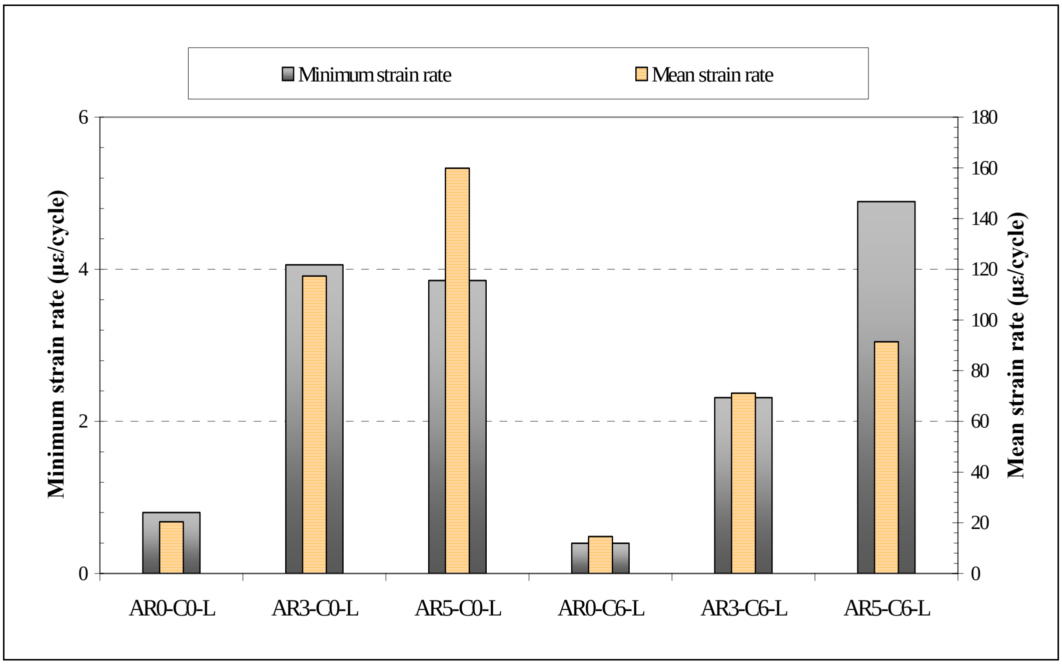


Figure 9.29: Minimum strain and mean strain rate of all highly compacted mixture subjected to long-term ageing

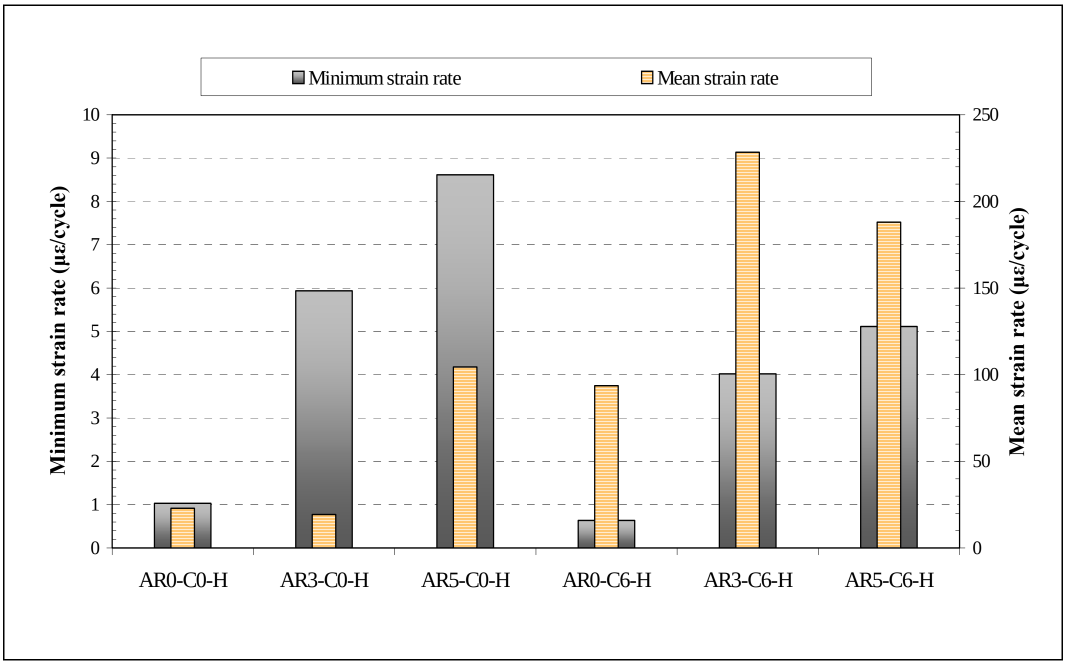


Figure 9.30: Minimum strain and mean strain rate of all low compacted mixtures subjected to long-term ageing

On the other hand, minimum strain rate is a more direct way to measure the viscous response (permanent strain) of the material in the steady state part (1500-3000 cycles) and is a more reliable parameter to assess material performance. Therefore, the minimum strain rate parameter was used to compare all mixtures and is used in the following sections to interpret and discuss the results in terms of the effect of rubber content, short-term ageing and compaction effort.

9.5.2.1 Rubber Content

Table 9.20 presents a comparative rutting performance in terms of minimum strain rate of the control and CRM mixtures in their unconditioned state and following long-term oven age conditioning. The results are also compared with control mixtures by calculating the percentage changes for CRM mixtures in their unconditioned and long-term aged conditioned states. The results show that, in general, long-term oven ageing results in a reduction in resistance to permanent deformation for both 3% and 5% CRM mixtures by increasing minimum strain rate, indicating CRM mixtures are more susceptible to high temperature ageing and are therefore, less durable in the longer-term compared to conventional DBM mixtures.

Table 9.20: Relative permanent deformation performance as a function of rubber content of CRM asphalt mixtures subjected to long-term ageing

Mixture	**Minimum strain rate (με/cycle)**						**Percentage change (%) relative to control mixtures**			
	0% rubber		**3% rubber**		**5% rubber**		**3% rubber**		**5% rubber**	
	U	**A**	**U**	**A**	**U**	**A**	**U**	**A**	**U**	**A**
C0-L	0.61	0.8	2.09	4.1	1.27	3.9	243	413	108	388
C6-L	0.23	0.40	3.86	2.3	1.75	4.9	1578	475	661	1125
C0-H	3.84	1.0	2.93	5.9	3.81	8.6	-24	490	-1	760
C0-H	2.61	0.6	3.05	4.0	1.85	5.1	17	567	-29	750

Compared to unconditioned state test results, the percentage changes relative to control mixtures are also generally inferior where three out of four R3 mixtures (the exception is R3-C6-L) and four out of four R5 mixtures are showing increased strain rate compared to unconditioned state results. In addition, the increase is generally higher for 5%CRM mixtures indicating that the permanent deformation resistance decreased with increasing rubber content in the mixtures.

9.5.2.2 Short-Term Ageing

The effects of long-term oven ageing on the performance in terms of rutting resistance of the control and CRM mixtures produced with and without short-term ageing prior to compaction are shown in Table 9.21. The results are evaluated by calculating the percentage change compared to unconditioned state testing in both void contents.

It can be observed that compared to the unconditioned state results, the minimum strain rate for both highly and poorly compacted C6 control mixtures is slightly inferior to that of the C0 mixtures following long-term oven ageing. The results indicate that although age hardening of the bitumen can improve the rutting resistance, the excessive amount of bitumen oxidation can lead to a brittle mixture.

Table 9.21: Relative permanent deformation performance as a function of short-term ageing of CRM asphalt mixtures subjected to long-term ageing

Mixture	Minimum strain rate (με/cycle)				Percentage change compared to unconditioned state (%)	
	0 hours conditioning		6 hrs conditioning		0 hour	6 hour
	U (a)	*A (b)*	*U(c)*	*A (d)*	*(a-b)/a*	*(c-d)/c*
R0-L	0.61	0.8	0.23	0.40	-31	-74
R0-H	3.84	1.0	2.61	0.6	74	77
R3-L	2.09	4.1	3.86	2.3	-96	40
R3-H	2.91	5.9	3.05	4.0	-103	-31
R5-L	1.27	3.9	1.75	4.9	-207	-180
R5-H	3.81	8.6	1.85	5.1	-126	-176

On the other hand, with the exception of R3-C6-L, all the other C0 and C6 CRM mixtures have shown poorer performance compared to unconditioned state results. These observations indicate that although short-term ageing of the CRM mixtures marginally improves the rutting resistance, its influence is relatively minor compared to the effect of increasing rubber content in the mixtures.

9.5.2.3 Compaction Effort

The influence of compaction effort on the rutting performance for the control and CRM mixtures subjected to long-term age conditioning is listed in Table 9.22. In terms of strain rate, both control mixtures with higher and lower void contents appear to be less affected following long-term oven ageing than the CRM mixtures. For the CRM mixtures, similar increases in strain rate for both highly and poorly compacted mixtures are evident (the exception is R3-C6-L) but the rate is slightly higher for R5 mixtures, indicating that mixtures with increasing rubber contents are less capable of maintaining rutting performance in the long-term. In addition, a slightly reduced strain rate for the highly compacted (low void content) 3%CRM mixtures indicates that compaction may produce better aggregate-rubber-bitumen interlock thus improving rutting resistance. But overall the CRM mixtures showed considerable increases in strain rate following long-term ageing, thus indicating reduced long-term durability irrespective of compaction effort.

Table 9.22: Relative permanent deformation performance as a function of compaction effort of CRM asphalt mixtures subjected to long-term ageing

Mixture	Minimum strain Rate (με/cycle)				Percentage change compared to unconditioned state (%)	
	4% air voids		8% air voids		4% voids	8% voids
	U (a)	*A(b)*	*U (c)*	*A (d)*	*(a-b)/a*	*(c-d)/c*
R0-C0	0.61	0.8	3.84	1.0	-31	74
R0-C6	0.23	0.40	2.61	0.6	-74	77
R3-C0	2.09	4.1	2.91	5.9	-96	-103
R3-C6	3.86	2.3	3.05	4.0	40	-31
R5-C0	1.27	3.9	3.81	8.6	-207	-126
R5-C6	1.75	4.9	1.85	5.1	-180	-176

9.6 SUMMARY

The NAT has been used to conduct a durability study on continuously graded CRM asphalt mixtures. The results are compared to the performance of conventional, primary aggregate mixtures produced under similar conditioning and compaction regimes. Both the control and CRM asphalt mixtures were subjected to two forms of distress, Link Bitutest Test (Scholz, 1995) protocol for moisture induced damage and SHRP methodology (AASHTO, 1994) for age hardening. The stiffness modulus, fatigue and permanent deformation properties were measured after each type of test and compared with the results obtained from unconditioned state testing. The test results were interpreted in terms of the effect of rubber content, short-term ageing and compaction effort on performance.

The Indirect Tensile Stiffness Modulus Test (ITSM) was used to measure the stiffness modulus of the control and CRM mixtures subjected to moisture conditioning and long-term ageing. All the results were compared with similar mixtures tested in their unconditioned state. In terms of load bearing capacity (stiffness modulus), CRM mixtures are found to be more susceptible to moisture induced damage compared to conventional DBM mixtures. The reduction in stiffness was approximately 30% for mixtures with 3% crumb rubber and as high as 70% for 5% CRM mixtures after only one moisture conditioning cycle. Visual inspection showed that plucking of rubber particles from the CRM specimen after moisture conditioning was predominant especially in mixtures with 5% rubber contents. On the other hand, long-term oven ageing showed increased stiffness for both control and CRM mixtures, indicating that age hardening of bitumen improved load-bearing capacity. In addition, the influence of short-term oven ageing and compaction effort was found to be less significant in both CRM mixtures, as changes in stiffness are mostly dominated by the rubber content of the mixture. In addition, compared to long-term ageing, short-term ageing for both control and CRM mixtures was found to be more influential on the mixture's stiffness indicating the importance of the production stage.

The Indirect Tensile Fatigue Test (ITFT) was used to measure the fatigue performance of the control and CRM mixtures subjected to moisture conditioning and long-term ageing. All results were interpreted in terms of predicted strain and fatigue life and compared with similar mixtures tested in their unconditioned state. The results indicated that both moisture conditioning and long-term ageing adversely affected the performance of the CRM mixtures with reductions in fatigue life compared to the corresponding unconditioned state. The reductions in overall fatigue performance of the CRM mixtures subjected to moisture conditioning were mainly due to the reduction in stiffness and extensive cracking. On the other hand, although the stiffness of all mixtures (control and CRM) increased with age hardening, excessive loss of lighter fractions from the bitumen through rubber-bitumen interaction during short-term and long-term ageing lead to a reduction in adhesive/cohesive strength and consequently increased the brittleness of the CRM mixture resulting in a reduction of long-term fatigue life. In general, the compaction effort did not appear to have any more significance on the CRM mixtures than it had on the control mixtures.

The Confined Repeated Load Axial Test (CRLAT) was used to measure the resistance to permanent deformation of the control and CRM mixtures subjected to moisture conditioning and long-term oven aging. The permanent deformation analysis in terms of mean and total strain rate was found not to be suitable for CRM mixtures because the rough top surface, highly elastic nature of the rubber particles and low stiffness of CRM mixtures caused initial densification. However, minimum strain rate was used as it provides a more accurate way to assess the viscous response (permanent deformation) of the bituminous mixtures in the steady state portion of the deformation curve. Significant reductions in permanent deformation resistance were observed for all CRM mixtures following moisture conditioning and long-term ageing. The effect of rubber content was found to be the dominant factor in the permanent deformation resistance, where an increase in rubber content in the mixtures reduced the overall rutting resistance. In addition, short-term ageing and compaction effort had more influence on the rutting performance of control mixtures, whereas, no significant difference was observed in both high and low void content CRM mixtures.

CHAPTER 10

Pavement Modelling

10.1 INTRODUCTION

In the previous chapters laboratory investigations on the elemental mechanical properties of the CRM mixtures designed to use as a binder course layer have been reported. In this chapter, analytical modelling has been conducted to study the theoretical design life of a typical pavement structure with and without a CRM binder course layer. The pavement model consists of three bituminous material layers, namely, SMA surfacing, CRM and conventional DBM binder course layer and HDM base layer including Type 1 sub-base on a relatively weak foundation support. The details of the pavement model are presented in Section 10.3.

For analysis purposes, the design parameters, such as stiffness and fatigue equation for each of the bituminous layers were obtained from the following sources:

- A 10 mm maximum aggregate size SMA surfacing: a stiffness modulus value of 2000MPa was taken from the Link Bitutest Project (1996).

- CRM and conventional DBM binder course layer: Stiffness and fatigue parameters are obtained from elemental mechanical testing results presented in Chapter 8.

- A 20 mm maximum aggregate size HDM base layer: Stiffness and fatigue properties were obtained from earlier research conducted by Read (1996) where average stiffness values were 3357 MPa and the fatigue equation was $N_f = 5.99 \text{ X } 10^{12}\ \varepsilon^{-4.178}$.

In addition, a Poison's ratio of 0.35 was assumed for each bituminous layer as this value is routinely used in pavement design. The multilayer linear elastic analysis package BISAR (Bitumen Stress Analysis in Roads) (Shell, 1998) was used for the analysis and the pavement design was carried out using the Nottingham Design Method (Brown and Brunton, 1986). A brief description of BISAR and the Nottingham design method is included in section 10.2 of this chapter.

The results were interpreted in terms of the effect of rubber content, short-term ageing and compaction effort of the CRM material on pavement life. A comparative study on pavement life was also performed by altering the binder course layer position and also changing both binder course and base layer thickness.

In the first part of the chapter, a brief overview of the pavement design methods, in particular, the Nottingham Design Method and the analytical modelling using BISAR are presented. The next sections contain the main analysis of the results in different scenarios with a summary at the end.

10.2 PAVEMENT DESIGN AND MODELLING

The thickness of pavement layers can be calculated using either empirical or analytical design methods. The empirical design method aims at correlating parameters that significantly affect pavement performance and thicknesses. On the other hand, in the analytical design method, the objective is to limit the strain in the pavement structure to a level that reduces the potential for significant distress. In the following section, the procedure of the Nottingham Analytical Design Method (Brown and Brunton, 1986) has been presented including a brief description of the major two failure criteria, fatigue and permanent deformation. In addition, an overview of the analytical modelling using BISAR is presented.

10.2.1 Nottingham Design Method

The Nottingham design method is a chart-based method derived by computer analysis. In this method the following design parameters are essential;

- Asphalt layer thickness
- Asphalt stiffness
- Subgrade stiffness
- Annual average air temperature (AAAT)
- Vehicle speed
- Design traffic

To achieve the design temperature, AAAT is multiplied by 1.92 and 1.47 for fatigue and permanent deformation design temperatures. The allowable tensile and compressive strains for fatigue and permanent deformation are then calculated at the bottom of the bound layer and at the top of the subgrade. In each case, maximum allowable strain is calculated which could be defined as the number of load applications in millions of standard axles to reach in critical or failure conditions. Finally, using charts or equations, design standard axles and critical strains are used to calculate layer thickness for both critical and failure lives of the pavement**.** Traditionally two modes of failure are associated with the loading of a pavement and they are illustrated schematically in Figure 2.12 in Chapter 2. The design method considers two modes of pavement failure:

- Maximum tensile strain at the bottom of the bound layer, which is related to repetitive horizontal strains that are induced by each load application and cause fatigue cracking.

- The maximum compressive strain on top of the subgrade to reduce accumulation of permanent strain to a level, which reduces the potential for excessive subgrade deformation which causes structural rutting.

10.2.1.1 Fatigue Criterion

The design criterion for fatigue cracking is tensile strain with the maximum value occurring at the bottom of the layer mid-way between the dual wheels in the direction of trafficking. The design criterion defines the fatigue life for critical or failure conditions.

Critical condition is the first appearance of wheel path cracking and failure represents the fully cracked state. Careful assessment of available research has shown that insitu lives to “Failure” are about 440 times and critical condition is 77 times the fatigue life obtained in the laboratory. In this study, laboratory fatigue lives were measured using the ITFT test for individual mixtures and are used to calculate estimated design life for critical and failure states. The fatigue equations for the control and CRM mixtures tested in the unconditioned state are presented in Table 8.6.

10.2.1.2 Permanent Deformation Criterion

The deformation criterion, using subgrade strain ε_z, is less fundamentally based than for fatigue cracking (Thom, 1996). It is used to ensure that excessive rutting does not occur during the pavement design life and is based on the back-analysis of various pavements of known performance. It is, hence, an indication of rutting rather than a direct measure of it. However, prevention of excessive rutting is dealt with by limiting the maximum vertical strain on the subgrade (ε_z) and the rutting criterion for critical and failure conditions are described using the following equations.

$$N_{CR} = f_r \left[\frac{7.6 \times 10^8}{\varepsilon_z^{3.7}} \right] \quad \textbf{\textit{(10.1)}}$$

$$N_F = f_r \left[\frac{3 \times 10^9}{\varepsilon_z^{3.57}} \right] \quad \textbf{\textit{(10.2)}}$$

Where,

N_{CR}	=	life to critical condition (10 mm rut)
N_F	=	life to failure (20 mm rut)
f_r	=	a rut factor to account for the hot mixture type; used 1.56 for DBM mixtures.

10.2.2 Analytical Modelling Using BISAR

The stress-strain relationship of bituminous material is viscoelastic in nature, which depends on the temperature and loading duration. Bitumen Stress Analysis in Roads (BISAR) was developed by Shell Research in the early 1970s and is widely used in pavement engineering, as it allows linear elastic analysis in a layered structure and produces comprehensive calculations to provide the strain and stress profiles in a pavement structure resulting from different types of loading. In BISAR, the following configuration and material behaviour are assumed;

- The system consists of horizontal layers of uniform thickness resting on a semi-infinite base or half space.
- The layers extend infinitely in horizontal directions.

- The material of each layer is homogeneous and isotropic.
- The materials are elastic and have a linear stress-strain relationship.

The model is loaded on top of the structure with one or more circular loads, with a uniform stress distribution over the loaded area. The stiffness, Poison's ratio, thickness of the layer and the co-ordinate of the positions of the centres of the loads are required as input parameters. In addition, the centres of the loads and the positions at which stresses, strains and displacement have to be calculated are given as co-ordinates in a fixed Cartesian co-ordinate system. The programme calculates the eigen values and eigen vectors of the stress and strain tensors, the principal stresses and strains and corresponding principal directions.

It is important to note that BISAR only satisfies the assumption of linear elastic theory under limited conditions and uses a simplified pavement structure and loading for modelling purpose. In addition, BISAR cannot analyse the non-linear behaviour of the pavement, pavement response under dynamic loading and presence of crack in the pavement structure.

10.3 PAVEMENT MODELLING

10.3.1 Model Structure

A simplified pavement structure was chosen to carry out theoretical analysis. The pavement was considered as an elastic multi-layer system with CRM and DBM mixtures investigated in this research project being used as a binder course layer. Two five layered (three bituminous layers with a sub-base and a subgrade) fully flexible pavement structures were modelled with 300 mm and 240 mm bituminous layer thickness where full bond was assumed between layers. Eight CRM and four control mixtures were placed in two different scenarios. In the first scenario, the binder course layer was placed in between surface and base layers whereas in the second scenario it was placed in between the base and sub-base layer. In both cases the tensile strain was calculated under the bituminous layer and compressive strain on top of the subgrade. In addition, further models were analysed to investigate the influence of CRM binder course layer and overall bituminous layer thickness on pavement life.

10.3.2 Material Parameters

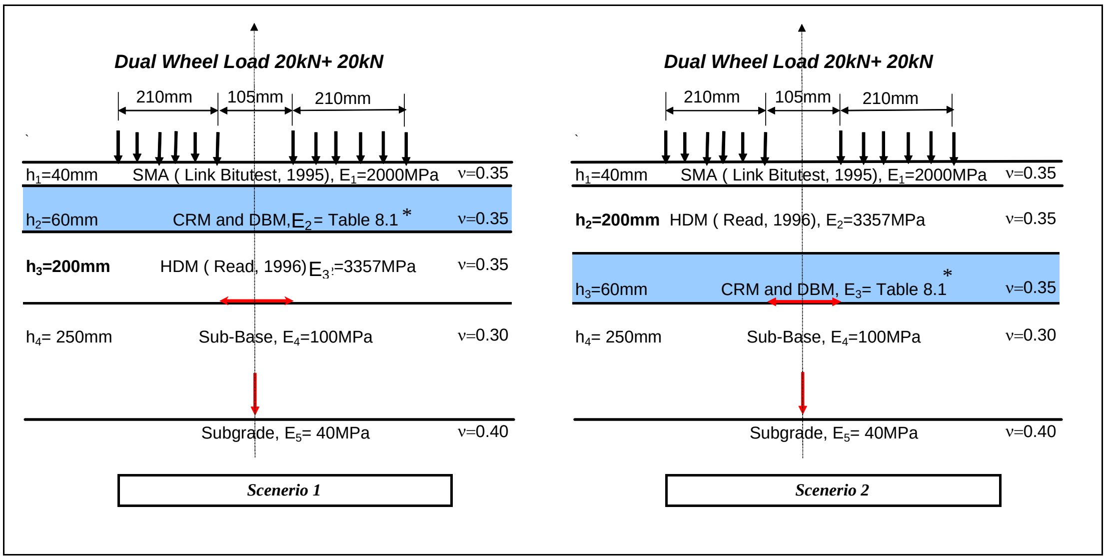
The individual layer thickness of the bituminous layer was chosen to reflect typical pavement thicknesses used in UK flexible construction. In addition, Type 1 Sub-base was chosen from the Design Manual for Roads and Bridges (DMRB, 2003) with a thickness of 250 mm and stiffness of 100MPa. The subgrade was assumed to extend to infinity at a relatively weak condition where a constant stiffness of 40MPa was assumed.

10.3.3 Loading

BISAR allows for a variety of loading conditions as a direct input in the model analyses. Normally, vehicular loads are represented as circular uniformly distributed loads. In this study, a standard vertical dual wheel load is considered where it was assumed to have dual 20kN wheels, each with a contact radius of 105 mm.

10.4 ANALYSIS OF RESULTS

The pavement models including layer parameters are presented in Figure 10.1.



** stiffness values are in Table 8.1*

Figure 10.1: Pavement model

The results shown in Tables 10.1 and 10.2 present the BISAR output of tensile and compressive strains for model 1 (Figure 10.1) where tensile strain was measured at the bottom of bound layer (Z=300 mm) and compressive strain was calculated on top of subgrade (Z=550 mm). The tables also present results for all highly and poorly compacted control and CRM mixtures modelled as a binder course layer and positioned in scenario 1 and scenario 2. These strains from scenarios 1 and 2 are then used to calculate design traffic using the Nottingham Design Method for both critical and failure conditions. The fatigue equation for HDM base layer used in Scenario 1 was obtained from earlier research conducted by Read (Read, 1996) and the fatigue equation for CRM binder course layer used in Scenario 1 and as base in Scenario 2 was obtained from Table 8.6 in Chapter 8. The fatigue life from the ITFT equation was then multiplied by appropriate factors outlined in the Nottingham Design Method to get predicted design traffic in the field. The design traffic for permanent deformation in the field was also calculated using Equations 10.1 and 10.2 to represent both critical.

Table10.1: Fatigue and permanent deformation criteria of highly compacted CRM mixtures placed in between surface course and base case (Scenario 1) and in between base course and sub-base (Scenario 2)

Mixture	**Stiffness (MPa)**	**ε_t at Z=300 mm**	**ε_z at Z=550 mm**	**N_f lab**	**N_f at Field (msa)**		**Permanent deformation (msa)**	
					critical	**failure**	**critical**	**failure**
Scenario 1(DBM/HDM)								
R0-C0-L	3642/3357	103	163	23,323	2	10	8	60
R3-C0-L	2708/3357	103	168	23,802	2	11	7	53
R5-C0-L	2032/3357	105	173	21,782	2	10	6	48
Scenario 2(HDM/DBM)								
R0-C0-L	3357/3642	98	162	21,752	2	10	8	61
R3-C0-L	3357/2708	109	170	436,591	34	196	7	51
R5-C0-L	3357/2032	120	178	177,388	14	80	6	43
Scenario 1 (DBM/HDM)								
R0-C6-L	4937/3357	98	157	28,434	2	13	9	68
R3-C6-L	3703/3357	100	162	26,170	2	12	8	60
R5-C6-L	3393/3357	101	164	25,525	2	11	8	58
Scenario 2(HDM/DBM)								
R0-C6-L	3357/4937	86	153	146,700	11	66	10	75
R3-C6-L	3357/3703	97	161	376,549	29	169	8	62
R5-C6-L	3357/3393	101	164	243,970	19	110	8	58

Table 10.2: Fatigue and permanent deformation criteria of poorly compacted CRM mixtures placed in between surface course and base case (Scenario 1) and in between base course and sub-base (Scenario 2)

Mixture	**Stiffness (MPa)**	**ε_t at Z=300 mm**	**ε_z at Z=550 mm**	**N_f lab**	**N_f at Field (msa)**		**Permanent deformation (msa)**	
					critical	**failure**	**critical**	**failure**
Scenario 1(DBM/HDM)								
R0-C0-H	2245/3357	104	171	22491	2	10	6	50
R3-C0-H	2112/3357	119	177	144597	2	10	6	49
R5-C0-H	1502/3357	107	178	19660	2	9	6	43
Scenario 2(HDM/DBM)								
R0-C0-H	3357/2245	117	175	25032	2	11	6	46
R3-C0-H	3357/2112	119	177	144597	11	65	6	44
R5-C0-H	3357/1502	130	186	114396	9	51	5	37
Scenario 1 (DBM/HDM)								
R0-C6-H	3928/3357	100	161	26644	2	12	8	62
R3-C6-H	2899/3357	102	167	24394	2	11	7	55
R5-C6-H	2297/3357	104	171	22673	2	10	7	50
Scenario 2(HDM/DBM)								
R0-C6-H	3357/3928	95	159	40475	3	18	8	64
R3-C6-H	3357/2899	107	168	101144	8	46	7	53
R5-C6-H	3357/2297	116	175	142285	11	64	6	46

and failure conditions. This data was subsequently used to investigate the effect of rubber content, short-term age conditioning and compaction effort on pavement life and are discussed in the following sections.

Although this approach is considered to give a useful indication of pavement life, it has to be recognised that the assumption of bottom up cracking is not always realistic. Furthermore, the use of a fatigue characteristic for the bottom layer alone is also open to question.

10.4.1 Rubber Content

The results presented in Table 10.3 (extracted from Tables 10.1 and 10.2) show that the position of CRM binder course layer has a significant influence on the pavement life. In scenario 1, fatigue is the critical mode of failure, although compared to the control mixtures the reduction of fatigue life is not greatly influenced by the incorporation of crumb rubber in the mixture. This is interesting as the stiffness modulus of the CRM mixtures was considerably lower (25% less in 3% CRM, 46% less in 5% CRM mixtures) than the control mixtures indicating the reduced stiffness of the binder course layer has less influence on the fatigue performance. On the other hand, when the CRM mixtures are placed in between the base and sub-base layers (Scenario 2), fatigue life was increased up to 196msa and 80msa for mixtures containing 3% and 5% crumb rubber respectively. In contrast, the position of control mixtures does not appear to have any influence on the fatigue performance.

In terms of permanent deformation for scenario 1, the design traffic to failure appeared to decrease with increasing rubber content in the mixtures. Additionally, for the CRM mixtures, although the critical mode of failure in Scenario 2 was the deformation failure, the design standard axles are still relatively close to Scenario 1. The results indicate that changing the position of CRM layer would increase fatigue performance without significantly compromising deformation resistance. It is interesting to note that the position of the control mixtures does not appear to have any implication on the pavement life regardless of its position. Therefore, when placed in Scenario 2, the flexible rubber particles in the CRM mixtures are believed to be the main contributor for the superior fatigue performance without compromising the overall deformation resistance.

Table 10.3: Design traffic to failure as a function of rubber content

Mixture	Design traffic to failure (msa)											
	Fatigue						Permanent deformation					
	Scenario 1			Scenario 2			Scenario 1			Scenario 2		
	Rubber content in percent											
	0%	*3%*	*5%*	*0%*	*3%*	*5%*	*0%*	*3%*	*5%*	*0%*	*3%*	*5%*
C0-L	10	11	10	10	196	80	60	53	48	61	51	43
C6-L	13	12	11	66	169	110	68	60	58	75	62	58
C0-H	10	10	9	11	65	51	50	49	43	46	44	37
C6-H	12	11	10	18	46	64	62	55	50	64	53	46

10.4.2 Short-Term Ageing

It can be observed from Table 10.4 that compared to the effect of rubber content, the short-term ageing of the CRM mixtures does not appear to have much effect on the overall pavement life. In both scenarios, the change in pavement life is predominantly due to the position of the CRM layer although slightly improved pavement life in terms permanent deformation can be observed for short-term aged mixtures compared to their corresponding unaged mixtures.

Table10.4: Design traffic to failure as a function of short-term ageing

Mixture	Design traffic to failure (msa)							
	Fatigue				Permanent deformation			
	Scenario 1		Scenario 2		Scenario 1		Scenario 2	
	Short-term ageing							
	0 hour	*6 hours*	*0 hour*	*6 hours*	*0 hour*	*6 hours*	*0 hour*	*6 hours*
R0-L	10	13	10	66	60	68	61	75
R0-H	10	12	11	18	50	62	46	64
R3-L	11	12	196	169	53	60	51	62
R3-H	10	11	65	46	49	55	44	53
R5-L	10	11	80	110	48	58	43	58
R5-H	9	10	51	64	43	50	37	46

10.4.3 Compaction Effort

Table 10.5 shows that in Scenario 1, in terms of fatigue life, the issue of under compaction does not appear to have much influence on the overall fatigue performance of the pavement. This is expected, as the binder course layer does not have much structural significance on the pavement life. On the other hand, a significant reduction in fatigue life was observed for the less compacted case compared to the highly compacted mixtures when the CRM layer was placed in between the base and sub-base layers (Scenario 2). However, the fatigue life is still relatively better in Scenario 2 than Scenario 1 indicating that the flexible rubber particles can accommodate larger numbers of traffic despite being less stiff due to poor compaction.

A similar effect was observed in permanent deformation behaviour, although the effect of under compaction on creep was relatively small compared to fatigue performance. This is rational as the compressive strain was measured on top of the subgrade, and the influence of a 60mm thick CRM layer would be less due to the presence of the 250 mm thick sub-base.

Table 10.5: Design traffic to failure as a function of compaction effort

Mixture	**Design traffic to failure (msa)**							
	Fatigue				**Permanent deformation**			
	Scenario 1		**Scenario 2**		**Scenario 1**		**Scenario 2**	
	Voids Contents							
	4%	*8%*	*4%*	*8%*	*4%*	*8%*	*4%*	*8%*
R0-L	10	10	10	11	60	50	61	46
R0-H	13	12	66	18	68	62	75	64
R3-L	11	10	196	65	53	49	51	44
R3-H	12	11	169	46	60	55	62	53
R5-L	10	9	80	51	48	43	43	37
R5-H	11	10	110	64	58	50	58	46

10.4.4 CRM Layer Thickness

The effect on pavement life was studied by changing the CRM layer thickness placed in between the base and sub-base layers (Scenario 2). The thickness was chosen to reflect the practicality of the model, therefore, it was restricted to 40, 50 (Figure 10.2) and 60 mm (Figure 10.1), as the minimum layer thicknesses should be at least twice the maximum aggregate size and thicknesses greater than 60 mm were considered to be less feasible because the thickness of the main structural layer would have to be decreased to maintain 300 mm overall thickness. The materials, geometrics and loading conditions of the model were similar to Figure 10.1 with the alternative being the different CRM layer thickness. The analysis was performed on both high and low compacted mixtures with 3% and 5% rubber contents produced with or without short-term ageing.

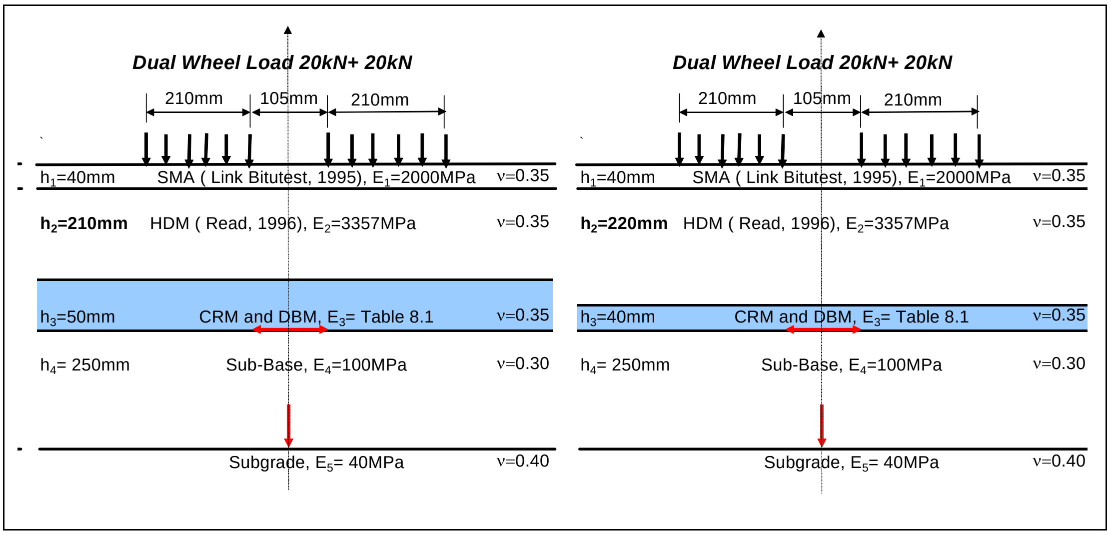


Figure 10.2: Pavement Model with different CRM layer thickness

The tensile strain was calculated at the bottom of the base (@240 mm, @250 mm and @260 mm) and CRM layer (@300 mm) and compressive strain was on top of the sub-grade. Tables 10.6 and 10.7 present the summary results for both high and low compacted mixtures.

It can be seen that irrespective of short-term ageing and compaction effort, the tensile strain is generally influenced by the stiffness of the CRM layer, where less stiff mixtures produced higher tensile strain at the bottom of base and CRM layer. In addition, the strain values under the base layer also increased with decreasing CRM layer thickness. This is despite the increase of base layer thickness with decreasing CRM layer to maintain overall bound layer thickness of 300mm. The reduction of the flexible CRM layer thickness may have contributed to higher tensile strain and, consequently, the overall fatigue lives have decreased under the base layer.

Another important observation is that although the tensile strain under the flexible CRM material is higher than conventional DBM mixtures, the fatigue failure for this model is always under the main structural layer and the lives reduced considerably with decreasing CRM layer thickness. The results indicate that the better tensile properties of the CRM materials would be beneficial as long as sufficient CRM layer thickness is provided.

In terms of permanent deformation, as expected, the change of CRM layer thickness does not have significant influence on the subgrade strain as long as the overall pavement thickness remains fixed. The results are consistent with less compacted mixtures and mixtures with short-term ageing.

Table 10.6: The strain and predicted traffic as a function of highly compacted CRM layer thickness in Scenario 2

Layer Thickness (mm)	Mixture Type	Tensile Strain (με)		Compressive strain (με)	Design traffic to failure (msa)		
					Fatigue		Deformation
SMA= 40 HDM=200 **CRM =60**		***@ 240***	***@300***	***@550***	***@ 240***	***@300***	***@550***
	R3-C0-L	65	109	170	70	196	51
	R3-C6-L	53	97	161	163	170	62
	R5-C0-L	76	120	178	37	81	43
	R5-C6-L	57	100	164	127	110	58
SMA= 40 HDM=210 **CRM=50**		***@250***	***@300***	***@550***	***@250***	***@300***	***@550***
	R3-C0-L	70	109	169	51	203	52
	R3-C6-L	59	97	161	105	166	61
	R5-C0-L	80	118	176	30	88	45
	R5-C6-L	63	101	164	84	110	58
SMA= 40 HDM=220 **CRM=40**		***@260***	***@300***	***@550***	***@260***	***@300***	***@550***
	R3-C0-L	76	108	169	38	214	53
	R3-C6-L	66	98	162	67	161	61
	R5-C0-L	84	116	174	25	96	47
	R5-C6-L	69	101	164	57	110	58

Table 10.7: The strain and predicted traffic as a function of poorly compacted CRM layer thickness in Scenario 2

Layer Thickness (mm)	Mixture Type	Tensile Strain (με)		Compressive strain (με)	Design traffic to failure (msa)		
					Fatigue		Deformation
SMA = 40 DBM =200 **CRM = 60**		***@ 240***	***@300***	***@550***	***@ 240***	***@300***	***@550***
	R3-C0-H	75	119	177	40	65	44
	R3-C6-H	63	107	168	83	46	53
	R5-C0-H	87	130	186	21	52	37
	R5-C6-H	72	116	175	48	64	46
SMA = 40 HDM = 210 **CRM = 50**	R3-C0-H	***@250***	***@300***	***@550***	***@250***	***@300***	***@550***
	R3-C6-H	79	117	175	32	69	46
	R5-C0-H	68	106	168	59	46	54
	R5-C6-H	89	127	182	19	58	40
	R3-C0-L	76	114	173	37	68	48
SMA = 40 HDM = 220 **CRM = 40**	R3-C0-H	***@260***	***@300***	***@550***	***@260***	***@300***	***@550***
	R3-C6-H	83	115	173	26	74	48
	R5-C0-H	74	106	167	42	47	54
	R5-C6-H	92	123	179	17	67	42
	R3-C0-H	81	112	172	29	73	49

10.4.5 Pavement Layer Thickness

The effect of the overall pavement layer thickness was studied by comparing results obtained from Model 1 (Figure 10.1) and Model 2 (Figure 10.3) where tensile and compressive strains were computed for both scenarios 1 and 2.

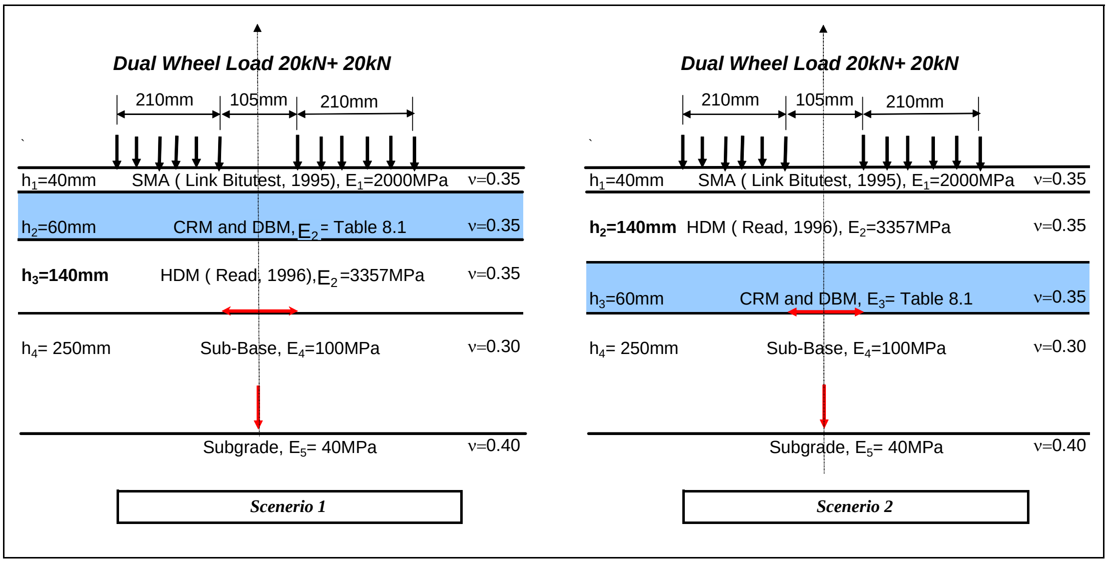


Figure 10.3: Pavement Model with different base layer thickness

It can be seen that in Model 1 the thickness of the bound layer was 300 mm which was reduced to 240 mm in Model 2 but in both cases the thickness of SMA surfacing, CRM binder course layer and Sub-base layer was kept constant. Therefore, it was possible to investigate the effect of the main structural base thickness on pavement life as a result of the presence of the CRM layer.

The summary of the results, in terms of design traffic to failure, is presented in Table 10.8. It can be observed that as a consequence of the reduction of bound layer thickness from 300 mm to 240 mm, the pavement life in both fatigue and permanent deformation conditions was reduced significantly. In terms of fatigue life, the reduction is more than three times in both scenarios although the life is comparatively better in Scenario 2 than Scenario 1. For permanent deformation, a 60mm reduction of base layer thickness leads to an almost three times less accommodation of traffic before the pavement fails in structural rutting. Therefore, to maintain overall structural performance it is important to maintain a reasonable thickness of base layer as this layer functions as a main load bearing part of the pavement structure.

Table 10.8: Effect of Bituminous layer thickness

Mixture	Design traffic to failure (msa)							
	Fatigue				Permanent deformation			
	Scenario 1		Scenario 2		Scenario 1		Scenario 2	
	Bituminous layer thickness (mm)							
	Z@240	*Z@300*	*Z@240*	*Z@300*	*Z@490*	*Z@550*	*Z@490*	*Z@550*
R0-C0-L	3	10	3	10	18	60	18	61
R0-C6-L	3	13	21	66	20	68	23	75
R3-C0-L	3	11	31	196	16	53	15	51
R3-C6-L	3	12	16	69	18	60	19	62
R5-C0-L	2	10	15	80	14	48	13	43
R5-C6-L	3	11	23	110	17	58	17	58
R0-C0-H	3	10	3	11	15	50	14	46
R0-C6-H	3	12	5	18	18	62	19	64
R3-C0-H	3	10	17	65	15	49	13	44
R3-C6-H	3	11	15	46	16	55	16	53
R5-C0-H	3	9	10	51	13	43	11	37
R5-C6- H	3	10	14	64	15	50	14	46

10.5 SUMMARY

This chapter has focused on the analytical modelling of a particular pavement structure with CRM binder course to predict pavement performance under loading. The analysis has been conducted in two scenarios. In scenario 1, the CRM layer was placed in between the surface and base layers, to simulate the effect of a CRM binder course layer in a typical pavement structure. In scenario 2, the CRM layer was placed in between the base and sub-base to investigate the effect of an alternative positioning of the flexible layer. The material properties for the surface and base layers were chosen from previous research and the stiffness of the control and CRM binder layers were used from unconditioned state NAT testing results. Two pavement models were analysed with varying thicknesses of bituminous layer in both scenarios and two failure criteria, fatigue and permanent deformation, were investigated using the Nottingham Design Method. The results were interpreted in terms of the effect of rubber content, short-term ageing and compaction effort. In addition, the effects of the bituminous layer thickness and CRM layer thickness on pavement life were also investigated.

The effect of rubber content was assessed in both scenarios by comparing the results with the control mixtures. In Scenario 1, despite lower stiffness modulus, CRM mixtures showed similar performance in terms of fatigue life compared to the control mixture although slightly reduced deformation resistance was observed with increasing rubber content in the mixtures. On the other hand, in Scenario 2, the fatigue life of the same pavement model increased more than 5 times without further reduction of deformation resistance. The flexibility of the CRM mixtures is believed to be the main reason for improved fatigue properties.

The effect of short-term ageing was also investigated. Although slightly improved fatigue and deformation performance were observed for the control mixtures, no significant effect on the CRM mixtures was evident. In addition, as expected, under compaction does not appear to influence the fatigue performance of both control and CRM mixtures when placed in between the surfacing and base layers. On the other hand, compared to the highly compacted mixtures, the performance of the poorly compacted control and CRM mixtures was reduced when the layers were placed in between the base and sub-base. The relative performance of the pavement with CRM layer was still significantly higher compared to the control mixtures. In terms of resistance to permanent deformation, in general, the performance of pavements with less compacted control and CRM mixtures appeared to be slightly inferior in scenario two than in scenario one. In contrast, the performance of the pavement with highly compacted control and CRM mixtures was similar in both scenarios. The results indicate that for this particular model, the lower stiffness of the CRM mixtures does not have a significant adverse effect on the resistance to permanent deformation but improved fatigue life can be achieved as long as the mixtures are properly compacted.

The thickness of the CRM layer in scenario two was changed without altering other layer thicknesses to assess the influence on pavement life. The thickness was changed from 60 mm to 40 mm and no noticeable change in the performance was observed. In general, although more tensile strain was developed under the CRM layer than under the base layer, the better fatigue properties of the CRM mixtures contributed to the improved fatigue life. In addition, the deformation resistance was not influence by the reduction of CRM layer thickness.

The effect of bituminous layer thickness on pavement life has been studied and it was observed in both scenarios that there were significant reductions in fatigue and permanent deformation with decreasing main structural layer thickness. This suggests the importance of the main structural layer in the pavement life.

Finally, from this particular model study, the salient finding is that the use of CRM material in a binder course layer would reduce the impact of the low stiffness modulus of the material. The increased fatigue life without further compromising deformation resistance would be of overall benefit to the pavement.

CHAPTER 11

Conclusions and Recommendations

11.1 INTRODUCTION

The primary aim of the research was to characterise the material and mixture properties of dry process CRM asphalt mixtures. In this chapter, the conclusions and recommendations that resulted from the research are presented. Section 11.2 presents the most important conclusions; the salient sub-conclusions related to the specific chapters are included in the subsequent sub-sections.

11.2 CONCLUSIONS

The main conclusions that can be drawn from the research undertaken in this thesis are:

- The basket drainage bitumen absorption method was found to be a simple and effective way to monitor rubber-bitumen interaction. The crumb rubber has been shown to absorb bitumen and swell when added together at mixing temperatures. The rate and amount of absorption is mainly dependent on the test temperature and complex chemical nature of bitumen but only marginally dependent on the bitumen type and grade. In addition to normal oxidation, the residual bitumen experienced further changes in mechanical and rheological properties in terms of increase in stiffness, elasticity, viscosity and reduction in penetration.

- In general, the stiffness for all the rubber-bitumen composite mixtures decreased due to the combined effect of short-term ageing of the loose mixtures and increased voids content following short-term ageing. The bitumen grade and testing temperature also influenced the stiffness. Phase angle, on the other hand, was not significantly influenced by bitumen type and grade used to produce the composite specimen, short-term ageing of the mixture, test temperature, stress level and frequency.
- In terms of the bearing capacity, the partial replacement of aggregate with 2 to 8 mm crumb rubber results in a significant reduction in stiffness modulus. The influence of short-term ageing and mixture compaction was found to have a similar effect on mixture stiffness irrespective of whether the mixture had been modified through the addition of crumb rubber. The high temperature (60^0C) permanent deformation resistance of the CRM mixtures was found to be inferior to that of the control mixtures although the differences in performance were lower for the high void content (8%) mixtures. In terms of strain rate, the permanent deformation performance of the 8% void content mixtures was very similar. The opposite was found for fatigue resistance with both the 3% and 5% CRM asphalt mixtures showing superior performance to that of the control mixtures, with and without short-term age conditioning and at both low and high air void contents.
- CRM mixtures were found to be more susceptible to moisture induced damage than conventional DBM mixtures. The permanent deformation resistance of the moisture conditioned CRM mixtures was found to be inferior to that of the control mixtures. On the other hand, although the fatigue performance deteriorated after moisture conditioning, the predicted strain and fatigue lives were still better than their corresponding control mixtures. The reduction of overall fatigue performance of the CRM mixtures after moisture conditioning was predominantly due to the considerable reduction in stiffness.

- The ageing of the CRM mixtures leads to an increase in stiffness through excessive loss of the lighter fraction of bitumen because of the combined effect of normal oxidation and rubber-bitumen interaction following short-term and long-term ageing. Consequently, the brittleness of the mixtures increased due to the loss of adhesive and cohesive strength of the material resulting in a reduction in fatigue life and resistance to permanent deformation of the long-term aged mixtures. However, the predicted strain and fatigue lives, calculated from fatigue and strain equations, were still better than similarly aged corresponding conventional mixtures. In addition, the compaction effort did not appear to have further significance on the long-term ageing properties for the CRM mixtures then it had on the control mixtures.
- The analytical study has indicated that the better fatigue properties of the CRM binder course layer can be useful to prolong the pavement life without significantly compromising the deformation resistance.

11.2.1 Constituent Materials

The main objectives of the literature review on the constituent materials (bitumen, rubber, tyres) including a brief overview on pavement engineering, was to provide the reader with the relevant background knowledge of the study. Some important conclusions that resulted from the literature review are presented below.

- The bitumen constitution and structure have an important contribution to the mechanical and rheological properties of the bitumen.
- Bitumen is a thermoplastic and viscoelastic material which means, at low temperature and/or short loading time bitumen behaves as a glassy solid, and at high temperature and/or long loading times bitumen acts as a Newtonian fluid.
- Vulcanisation and compounding are the main processes which provide engineering properties to raw rubber. Vulcanisation transforms the raw rubber into a strong, elastic and rubbery hard state. The most common vulcanisation process is through sulphur. In compounding, a number of ingredients are added to modify and improve the physical properties of rubber. The first reason for compounding is to incorporate the ingredients and ancillary

substances necessary for vulcanisation. The second is to adjust the hardness and modulus of the vulcanised product to meet the end requirement.

- There are four main components in the tyre, namely elastomer (rubber) compound, carbon black, fabric and steel. The elastomer is composed of natural and synthetic rubber. The proportion varies according to the size and use of the tyre. The generally accepted rule of thumb is that the larger the tyre and the more rugged its intended use, the greater will be the ratio of natural to synthetic rubber.
- The life span for a tyre is approximately 5-7 years before being defined as waste (scrap tyre). The scrap tyre can be usable in different forms including production of crumb rubber for asphalt mixtures.
- Pavement design is predominantly associated with the assumption of linear elastic theory under limited conditions and often involves the simplification of the pavement structure and loading. In analytical design, two classical strain criteria, the vertical compressive strain on top of subgrade and horizontal tensile strain at the bottom of the bound layer, are used as performance criteria to design against permanent deformation and fatigue cracking respectively.
- Moisture damage and long-term ageing are the two main durability criteria that bituminous mixtures are require to sustain for the period of their design life. Laboratory testing methods for both moisture susceptibility and ageing are highly empirical and consequently fall sort to predicting actual field performance.

11.2.2 Crumb Rubber Modified Asphalt Mixtures

A relevant literature review on the swelling mechanism of rubber and a brief overview of the CRM mixtures including filed applications and case studies are reported. The salient conclusions that can be drawn from these reviews are;

- When rubber is added to the bitumen at high temperature, it imbibes proportions of the bitumen and swells. The amount of swelling mostly depends on the temperature, particle size, test duration, viscosity and complex chemical nature of the solvent.

- Case studies on both dry and wet process CRM methods revealed that the field performance of the dry process is not as consistent as the wet process, and the distresses are mainly in the form of cracking, fretting and rutting. Although application of the dry process is logistically identical to conventional mixtures and consumes larger quantities of waste tyres, only limited fundamental studies have been conducted to resolve those issues and, therefore, it is less popular than the wet process.

11.2.3 Rubber-Bitumen Interaction

The interaction between the single source of crumb rubber and the eight penetration grade bitumens, in terms of the increase in rubber mass and bitumen absorption were studied using a basket drainage bitumen absorption method. The lists of conclusions that can be drawn from this investigation are:

- The basket drainage bitumen absorption method was found to be a simple and effective technique to monitor rubber-bitumen interaction.
- The swelling potential of different batches of crumb rubber produced from scrap truck tyres were found to be consistent which indicated similar chemical and physical properties.
- The initial rate of bitumen absorption is directly related to the viscosity (penetration grade) as well as the chemical composition (crude source) of the binders with the softer (less viscous) and lower asphaltene content binders having the highest rates of absorption.
- The maximum increase in rubber mass (bitumen absorption) after 48 hours at 160°C or at equiviscous temperature of bitumen appears to be independent of crude source and only marginally related to the penetration grade of the bitumen. The softer, high penetration grade bitumens generally resulted in a greater amount of absorption compared to the harder, low penetration grade bitumens.

11.2.4 Chemical and Mechanical Testing of Residual Bitumen

The chemical and mechanical properties of the virgin, aged (without rubber) and residual bitumen collected following rubber-bitumen interaction were determined. The following conclusions can be presented based on the results obtained from the investigations:

- In addition to traditional oxidation of bitumen at high temperatures, the residual bitumens experienced further changes in their chemical constitution as a result of the crumb rubber-bitumen interaction and the absorption, by the rubber, of the lighter, more volatile fractions of the bitumen.
- The residual binders showed a considerable increase in high temperature viscosity, complex modulus and elastic response compared to both the virgin as well as the oxidised binders. In general, the increase in viscosity, stiffness and elastic response can be considered to be similar irrespective of crude source or penetration grade, although the absolute values for these parameters will be lower for the "softer" bitumens.
- Any changes in the rheological properties of the low penetration grade bitumen in a dry process CRM asphalt binder following rubber-bitumen interaction could result in the binder becoming embrittled with a loss of flexibility and the ability to resist cracking and fretting. On the other hand, the use of high penetration grade bitumen would increase both the rate and possibly the amount of rubber swelling and therefore the shape and rigidity of the rubber would be affected. However, the binder should still have sufficient flexibility following the rubber-bitumen interaction to resist cracking and fretting.

11.2.5 Idealised Rubber-Bitumen Composite Mixtures

Ranges of composite mixtures were produced using hard and soft bitumen from two different sources and subjected to short-term ageing up to six hours at 160^0C prior to compaction. All the mixtures were tested using very low stress levels (10kPa to 50kPa) and five frequencies (0.1, 0.2, 1, 2, 5Hz) and three temperatures (5, 20, 35^0C). The resultant axial strains were measured and used to calculate the dynamic mechanical properties such as complex modulus and phase angle. The salient conclusions from this investigation are:

- Using the bitumen film thickness for rubber-bitumen proportion in the idealised composite mixtures was found to be a realistic and simple way to simulate the proportion in the dry process CRM mixtures.
- The stress-strain response was found to be non-linear and highly elastic in nature and the strain generated under load was significantly higher indicating rubber dominance in the response.
- Idealised rubber-bitumen composite mixtures have shown that the mixtures become voided due to the rubber-bitumen interaction following short-term ageing. In addition, the composite mixture stiffnesses were found to be influenced by the testing temperature, but only marginally dependent on test frequencies.
- In terms of bitumen types, the stiffness reduction is significantly higher for mixtures produced using softer bitumens (M1, V1) than for mixtures produced using harder bitumens (M3, V3). In general, stiffness for all composite mixtures reduced due to the combined effect of short-term ageing and voids content. Phase angle on the other hand is not significantly influenced by bitumen type and grades used, short-term ageing of the mixture, mixture volumetrics, test temperature, stress level and frequency. The overall phase angle of the mixture was within the range of 14^0-18^0 and mainly influenced by the rubber component in the mixture indicating the viscoelastic nature of the CRM asphalt mixture could be compensated by using more flexible rubber particles.

11.2.6 Mixtures Design

A continuously graded, 20mm maximum aggregate size Dense Bitumen Macadam (DBM) asphalt mixture, as specified in BS 4987-1:2001, was used to manufacture a range of control and dry process crumb rubber modified (CRM) mixtures. The CRM mixtures were produced using 3% and 5% crumb rubber content by mass of total aggregate at two degrees of short-term age conditioning and two air void contents. The following conclusions can be drawn from the mixture design chapter:

- The partial replacement of aggregate with crumb rubber particles from the continuously graded Dense Bitumen Macadam (DBM) required approximately 10% higher bitumen content than conventional primary aggregate mixtures to overcome the initial rubber-bitumen interaction during short-term conditioning.
- The CRM mixtures require extra compaction effort to achieve the target density due to the "bouncy" nature of the rubber particles. In addition, the specimens are required to be kept inside the mould for a prolonged period to allow the specimen to reach room temperature and the bitumen to gain sufficient strength to stop rubber rebounding.
- Due to the lower density of the rubber particles, the density of the CRM mixtures reduces with increasing rubber content as well as voids in the mixtures.
- A relative comparison with other widely used dry process CRM mixtures has shown that CRM mixtures used in this project contain larger particles sizes and quantities of crumb rubber with less binder content than other patented dry process mixtures. In addition, the CRM mixtures for this project were designed to be used as a binder course layer while other dry processes were mainly for surfacing application.

11.2.7 Asphalt Mixture Mechanical Properties

The elemental mechanical performance of the twelve dry process CRM continuously graded asphalt mixtures (3% and 5% rubber contents, 0, 2 and 6 hours age conditioning, 4% and 8% voids content) have been assessed using the NAT and compared to the performance of four conventional (0 and 6 hours age conditioning, 4% and 8% voids content) primary aggregate mixtures. The mechanical properties that were investigated consisted of stiffness modulus, resistance to fatigue cracking, and resistance to permanent deformation. The elemental mechanical testing produced the following conclusions;

- In terms of the load bearing capacity (stiffness modulus), the partial replacement of aggregate with 2 to 8 mm crumb rubber results in a substantial reduction in stiffness modulus of approximately 25% for the 3% CRM asphalt mixtures and 45% for the 5% CRM mixtures.
- The influence of short-term ageing and mixture compaction was found to have a similar effect on mixture stiffness irrespective of whether the mixture had been modified through the addition of crumb rubber. In addition, the short-term conditioning of the mixture increased stiffness irrespective of compaction effort.
- Fatigue resistance for both the 3% and 5% CRM asphalt mixtures showed superior performance to that of the control mixtures, with and without short-term age conditioning and at both the low and high air void contents.
- Permanent deformation of CRM mixtures is dominated by the rubber content. CRM mixtures produced more strain after 3600 load pulses compared to control mixtures. The mean strain rate, which reflects the performance of the material during the initial phase of the test, is much higher compared to control mixtures. This is due to the initial slack of the apparatus and the presence of highly elastic rubber in lower stiffness CRM mixtures. In addition, conditioning at the testing temperature generated extensive cracking which raised concerns over the durability of the CRM mixtures. However, the minimum strain rate, which reflects the performance of the material in the later stage of the testing, was also higher for CRM mixtures in all testing conditions. The results also demonstrate that CRM mixtures are more susceptible to permanent deformation at high temperatures.
- Compared to the control mixtures, the permanent deformation resistance of the CRM mixtures was found to be less affected by compaction effort. In terms of minimum strain rate, the permanent deformation performance of CRM mixtures demonstrated similar performance at both high and low void contents.
- Provided that the CRM asphalt mixtures can be compacted to a sensible air void content (maximum of 8%) and ignoring any adverse durability issues, the material can be considered to demonstrate reasonable mechanical properties relative to a control DBM mixture. The use of the CRM material in a binder

course layer would reduce the impact of the low stiffness modulus of the material while the increased fatigue resistance would be of overall benefit to the pavement.

11.2.8 Durability of CRM Asphalt Mixtures

The NAT has been used to conduct a durability study on continuously graded CRM asphalt mixtures. The results are compared to the performance of conventional, primary aggregate mixtures produced using similar conditioning and compaction regimes. Both control and CRM asphalt mixtures were subjected to two forms of distress, Link Bitutest Test (Scholz, 1995) protocol for moisture induced damage and SHRP methodology (AASHTO PP2, 1994) for age hardening. The stiffness modulus, fatigue and permanent deformation properties were measured after each type of test and compared with the results obtained from unconditioned state testing. The test results were interpreted in terms of the effect of rubber content, short-term ageing and compaction effort on performance. The salient conclusions from durability studies are:

- The percentage saturation increases with increasing rubber content in the mixtures as well as with increasing void content.
- Visual inspection showed that plucking of rubber particles from the CRM specimen after moisture conditioning was dominant especially in mixtures with 5% rubber contents. In addition, the CRM mixtures appeared to be more temperature sensitive than the control mixtures as vertical expansion and extensive cracking was observed following six hours conditioning at 60^0C.
- CRM mixtures are more susceptible to moisture induced damage compared to conventional DBM mixtures. The reduction in stiffness was approximately 30% for mixtures with 3% crumb rubber and as high as 70% for 5%CRM mixtures after only one moisture conditioning cycle.
- Compared to the unaged conditioned state, the fatigue performance was also found to be adversely affected with significant reduction in fatigue lives. The reduction of the overall fatigue performance following moisture conditioning was predominantly due to considerable reduction in stiffness. However, despite the fact that CRM mixtures are adversely affected by moisture

conditioning, their relative performance is still better than similarly conditioned control mixtures.

- The high temperature (60^0C) permanent deformation resistance of the moisture conditioned CRM mixtures was found to be inferior to that of the control mixtures.
- The ageing of the CRM mixtures leads to an increase in stiffness through excessive loss of the lighter fraction of bitumen because of the combined effects of normal oxidation and rubber-bitumen interaction following short-term and long-term ageing. The increase in stiffness generally appears to be lower for mixtures with higher rubber contents indicating that the softening effect of flexible rubber particles may have contributed to compensate the increased effect of bitumen hardening.
- The brittleness of the mixtures increases due to the loss of adhesive and cohesive strength of the material resulting in a reduction in long-term fatigue life and resistance to permanent deformation. However, the predicted strain and fatigue lives calculated from fatigue and strain equations were still better than similarly aged corresponding conventional mixtures. In addition, the compaction effort did not appear to have any more significance on the long-term ageing properties on the CRM mixtures then it had on the control mixtures.

11.2.9 Pavement Modelling

Analytical modelling has been conducted to study the theoretical design life of a typical pavement structure with and without a CRM binder course layer. The pavement model consists of three bituminous material layers, namely, SMA surfacing, CRM and conventional DBM binder course layer and HDM base layer including Type 1 sub-base and relatively weak foundation support. The CRM layer was placed in two different scenarios. In the first scenario the binder course layer was placed in between surface and base layers whereas in the second scenario it was placed in between the base and sub-base layer. The following conclusions regarding the influence of CRM binder layer on pavement life can be drawn:

- The influence of the CRM binder layer on the pavement life in two different scenarios has been assessed using one set of surfacing, base and foundation conditions. When positioned in between the surfacing and base layer, the CRM binder course layer performed equally with conventional DBM mixtures. When positioned in between base and sub-base layer, the pavement with a properly compacted CRM layer showed significantly higher fatigue life without significantly compromising the resistance to permanent deformation.
- As expected, the overall pavement life appears to be reduced by the reduction of the base layer and also in scenario two, CRM layer thickness.

11.3 RECOMMENDATIONS

11.3.1 Recommendations on Constituent Materials

Despite several conclusions being drawn from this research project, a lot is still not known about the material. A list of recommendations is summarised in the following paragraphs:

- Because of the complicated nature of the rubber particles and the interaction of rubber and bitumen, future research should aim at developing a modified binder that could introduce better adhesion between rubber and bitumen. Detailed investigation should be carried out to provide a standard base level of the crumb rubber particles to reduce swelling potential.
- The work carried out on the interaction between the rubber and the bitumen has considered the change of mass of the swelled rubber following different durations of curing. Although this method gives an indication of the swelling potential, the gain of mass of rubber particles may not have the same volume within the bituminous mixture. Therefore, future research should concentrate on measuring the accurate volume changes of the rubber particles following rubber-bitumen interaction.
- The mechanical properties of the residual bitumen in terms of rheology, viscosity and penetration have been studied in this research project. Future research could be extended to determine the low temperature fracture and

tensile properties of the residual bitumen following rubber-bitumen interaction.

- As the asphaltenes content test undertaken in this research project only provides the reduction of percentage of maltenes fraction in relation to asphaltenes content, detailed chemical composition testing would be desirable to obtain a complete fingerprint of each component and thereby adding specific components would counterbalance the interaction. In addition, to finding a method of reducing the activity of the interaction or the effect of the interaction on the performance of the material, it may be necessary to consider the chemistry of the interaction for both bitumen and rubber. Finally, the research could be expanded to establish a relationship between the chemical properties of the bitumen (virgin and residual) with the rheological properties (virgin and residual) to get an indication on how the chemical composition could affect the response of the bitumen.
- Due to the lack of time and resources, only six different batches of crumb rubber from one manufacturer were used to investigate the swelling potential of the crumb rubber. A very limited investigation had been undertaken (not reported in this thesis) to examine the molecular arrangement of the different crumb rubber particles. Infrared Spectroscopy analysis was used for this purpose and although the results were not analysed in detail, it showed that rubber particles used for this research have more or less identical fingerprints. The method requires only a few seconds to perform and therefore future research could be extended to establish a relationship between rubber chemistry and their binder absorption characteristics. A detailed investigation is, therefore, required on a variety of scrap tyre sources including different types of crumb rubber produced using different techniques.
- Although not reported, a limited study was conducted on rubber pre-treatment prior to mixing with bitumen. The objective was to provide a standard base level for the rubber particles and thereby reduce the swelling potential in the interaction process by coating the rubber particles or by increasing the cross-link density by adding a cross-linking agent like sulphur. Various percentages of sulphur were used in the pre-treatment study and it was found that the initial swelling of the rubber particles could be reduced considerably.

Although the results were promising in terms of swelling potential, the experiments were not extended due to the lack of controlled environment for this treatment operation. Future research could carry out a detailed investigation on the pre-treatment of rubber using different types of inexpensive waste material, such as sulphur etc. and their effect on the chemistry of the resulting rubber particles.

- The mechanical interaction of the rubber-bitumen composite mixtures produced using one rubber to bitumen ratio was studied in this research. The research mainly concentrated on the effect of short-term ageing, frequency and test temperature on different mixtures produced using different types of bitumen. In addition, a range of dry process CRM composite mixtures with different rubber to bitumen ratios could also be tested to establish a relationship to predict mixture performance from the composite performance.
- The residual (recovered) bitumen from the rubber-bitumen composite specimen would give a clear indication of the percentage of bitumen absorbed by the rubber at curing stage and help to develop guidelines for how much bitumen is needed to compensate for the expected short-term ageing of the material during transportation, laying and compaction. Various attempts had been undertaken to recover residual bitumen from the composite mixtures. It was found that the rubber particles are affected by the solvent that are conventionally used in the binder recovery method. Further research could be undertaken to find a suitable solvent to recover bitumen without affecting the swelled rubber.
- Another important area to investigate is the binder adhesion with rubber particles and possible modification of the adhesion of rubber and bitumen using different types of additives, binder types, etc.

11.3.2 Recommendations on Mixture Properties

One of the original goals of the research presented in this thesis was to study the elemental mechanical properties of the different dry process CRM asphalt mixtures. Although several conclusions were drawn from this research, there is still a lot to learn from this challenging material before it could be implemented in the UK road network. Based on the experience gained from this study, the recommendations for future research are listed below:

- As this investigation showed that it is possible to produce dry process CRM mixtures by modifying a conventional DBM mixture gradation, future research should investigate a wider range of aggregates, rubber gradations and mixture types. The ultimate goal of any future research should, therefore, be aimed at producing a design chart for different aggregate gradations with different percentages and sizes of rubber particles. In addition, based on the analytical modelling supported by the wide range of laboratory results, a design guideline for CRM layer thickness for different traffic and environmental conditions should be developed. Furthermore, end of life product in terms of recycling of CRM mixtures can also be investigated.
- Although the stiffness modulus of the CRM mixtures was less compared to conventional DBM mixtures, the fatigue life of the CRM mixtures was found to be significantly better. However, the resistance to high temperature permanent deformation was found to be inferior to conventional mixtures. Although not reported, limited creep recovery tests were conducted on control and CRM mixtures following CRLAT tests. It was found that the creep recovery of the CRM mixtures is significantly higher than conventional primary aggregate DBM mixtures. Therefore, future exploration could be on detailed creep recovery tests for different CRM mixtures to understand the potential rebounding effect of the flexible rubber particles.
- As the CRM mixtures showed considerable deterioration in their mechanical properties following moisture conditioning, future research should concentrate on either improving the testing procedures by providing some level of protection of the CRM specimen which could reflect more realistic field conditions or improve rubber-bitumen adhesion to minimise rubber rebounding during high temperature conditioning.
- As the NAT has limitations in applying very low stress levels and is also not capable of performing strain control mode testing, it would be desirable to perform more fundamental tests, such as three-point bending tests for fatigue properties and triaxial testing for the permanent deformation tests. These tests

would be useful in assessing the fatigue and permanent deformation performance of the CRM mixtures.

- The study was mainly limited to elemental laboratory testing. It is recommended that larger scale laboratory testing (wheel tracking tests, pavement testing facility) would be useful to validate stiffness, permanent deformation and fatigue cracking of the CRM mixtures. In addition, immersion wheel tracking tests would be useful to understand the fretting resistance of the CRM mixtures.
- One of the most important recommendations for future research is to perform full-scale field trials using different types of CRM mixtures. This will enable experience to be gained in CRM mixing and construction characteristics and in identifying differences over conventional mixture criteria. In addition, the results obtained from the field trials will enable maintenance and rehabilitation guidelines for CRM material to be produced using the dry process technology.
- The limited analytical study conducted in this research project has suggested that the better fatigue properties of the CRM material can be beneficial for overall pavement life. The modelling could be extended using different combinations of mixtures and foundation conditions where the ultimate goal would be to prepare a design chart in terms of layer thickness and design traffic for the use of dry process CRM technology.

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