INFLUENCE OF CRUMB RUBBER MODIFIER ON PERFORMANCE CHARACTERISTICS OF STONE MASTIC ASPHALT

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ABSTRACT

In asphalt concrete (AC), bitumen as a binder serves two major functions in road pavement; first, to hold the aggregates firmly and secondly, to act as a sealant against water. However, due to several distresses like fatigue failure, the performance and durability of certain characteristics in bitumen are highly affected by changes with time which can lead to the cracking of pavements. To prevent pavement distresses there are various solutions such as adopting new mix designs or utilisation of asphalt additives. Using asphalt additives in highway construction is known to provide virgin bitumen better engineering properties. Scrap tyres lead to grave disposal problems. However, with the use of scrap tyres in asphalt pavements in the form of an additive, they are advantageous in minimising environmental pollution and maximising natural resource conservation. The primary aim of this study was to investigate the effect of adding crumb tyre rubber as a modifier to Stone Mastic Asphalt (SMA) mixture performance properties. This study investigated the essential aspects of modified asphalt binder and mixtures in order to better understand the influence of CRM modifiers on the physical-rheological properties of asphalt binder and fatigue resistance of SMA mixture. In this study, 80/100 penetration grade bitumen was used; modified with crumb rubber (CRM) at five different modification levels namely 6 , 12, 16 and 20% respectively by weight of the bitumen. The physical properties of modified bitumen samples were performed using the classic binder tests namely, penetration test, softening point test, ductility test and elastic recovery test. The rheological properties for asphalt binder were conducted using Superpave tests of Brookfield viscosity and dynamic shear rheometer (DSR). The stiffness and mechanical properties as well as fatigue characteristics of (SMA) mixtures produced with and without the crumb rubber were investigated using IDT and ITFT on varying binder content.

The results of the study concluded that rubber crumb content plays a main role in influencing the performance and rheological properties of rubberised bitumen binders. The increase in rubber crumb content was from 6 - 20% thus indicating a linear increase in softening point, viscosity, elastic recovery and complex shear modulus. The resilient modulus (Mr) of modified SMA samples including different percentages of CRM was obviously higher in comparison with that of unmodified samples. In addition, crumb rubber content significantly enhanced durability of crumb rubber modified bitumen and led to better aging resistance. Thus, the crumb rubber modified bitumen was less susceptible to temperature susceptibility. With the presence of crumb rubber, the fatigue life of crumb rubber modified bitumen significantly improved. The resistance of crumb rubber in producing horizontal tensile stresses attenuated the production of vertical cracks and deterred these cracks from diffusing along the diameters of the asphalt samples. This in turn improved the fatigue life of reinforced samples. Besides aspiring to decrease piling up of waste materials in the environment, utilising waste tyre rubber has improved the characteristics of engineering structure and materials in asphalt production and comparable industries. Consequently, it has also reduced construction rehabilitation and maintenance costs.

ABSTRAK

Di dalam konkrit berasfalt (AC), bitumen sebagai pengikat bertindak dengan dua fungsi utama untuk turapan jalan raya; pertama, untuk mengikat kukuh aggregat dan kedua, bertindak sebagai pelekat terhadap air. Walau bagaimanapun, disebabkan oleh beberapa tegasan (kegagalan) seperti kegagalan kelesuan, prestasi dan ketahanan beberapa ciri-ciri dalam bitumen adalah sangat dipengaruhi dengan perubahan masa yang boleh membawa kepada retakan dalam turapan. Untuk mencegah tegasan (kegagalan) dalam turapan, terdapat pelbagai penyelesaian seperti mengamalkan rekabantuk campuran yang baru atau penggunaan bahan tambah asfalt. Penggunaan bahan tambah asfalt di dalam pembinaan lebuh raya telah diketahui akan menjadikan bitumen segar dengan sifat-sifat kejuruteran yang lebih baik. Tayar sekerap membawa kepada masalah pelupusan. Walau bagaimanapun, dengan penggunaan tayar sekerap di dalam turapan berasfalt dalam bentuk bahan tambah, ia sangat bermanfaat dalam meminimakan pencemaran alam sekitar dan memaksimakan pemuliharaan sumber semulajadi. Tujuan utama kajian ini adalah untuk menyiasat kesan penambahan remah tayar getah sebagai bahan tambah ke atas prestasi sifat-sifat campuran SMA. Kajian ini menyiasat aspek penting pengikat asfalt terubahsuai dan campuran untuk memahami lebih baik kesan pengubahsuai CRM ke atas sifat-sifat fizikal-reologi bagi pengikat asfalt dan rintangan kelesuan bagi campuran SMA. Di dalam kajian ini, bitumen bergred penusukan 80/100 telah digunakan, kemudian diubahsuai dengan remah getah pada lima aras modifikasi berbeza iaitu 6,12,16 dan 20% daripada berat bitumen. Sifat-sifat fizikal sampel bitumen terubahsuai dinilai menggunakan ujian pengikat iaitu ujian penusukan, ujian penentuan titik lembut, ujian kemuluran, ujian pemulihan elastik dan ujian kelikatan Brookfield. Sifat-sifat reologi untuk pengikat asfalt dijalankan dengan menggunakan rheometer ricih dinamik (DSR).Sifat-sifat kekakuan dan mekanikal iaitu ciri-ciri kelesuan campuran SMA dibuat dengan menggunakan remah getah tayar dan juga tidak

menggunakan remah getah tayar. Sifat-sifat ini dikaji dengan menggunakan IDT dan ITFT ke atas kandungan pengikat yang berubah. Keputusan daripada penyelidikan ini merumuskan bahawa kandungan remah tayar getah memainkan peranan yang utama dalam prestasi dan sifat-sifat reologikal pengikat bitumen getah. Peningkatan kandungan remah tayar getah daripada 6 ke 20% menunjukkan peningkatan linear dalam ujian penentuan titik lembut, kelikatan, pemulihan elastik dan modulus ricih kompleks. Modulus ketahanan (Mr) adalah tinggi bagi sampel-sampel SMA terubahsuai yang berlainan peratusan CRM jika dibandingkan dengan sampel-sampel yang tidak terubahsuai (asal). Tambahan pula, kandungan remah tayar getah meningkatkan ketahanan bitumen terubahsuai CR dan menunjukkan rintangan jangka hayat yang lebih baik.Oleh itu, remah getah bitumen terubahsuai adalah kurang terdedah kepada kecenderungan suhu. Dengan kehadiran remah getah, jangka hayat kelesuan CR bitumen terubahsuai meningkat dengan ketara. Rintangan getah tayar sekerap dalam menghasilkan tegangan melintang melemahkan penghasilan retak menegak dan menghalang retak ini dari meresap di sepanjang diameter sampel asfalt. Ini seterusnya meningkatkan jangka hayat kelesuan sampel bertetulang.

Selain bercita-cita untuk mengurangkan penimbunan bahan-bahan buangan dalam persekitaran, penggunaan semula sisa getah tayar telah meningkatkan ciri-ciri struktur kejuruteraan dan material dalam pengeluaran industri asfalt dan setanding dengannya. Oleh itu, ia juga telah mengurangkan pemulihan dalam kerja-kerja pembinaan dan kos penyelenggaraan.

UNIVERSITI MALAYA
ORIGINAL LITERARY WORK DECLARATION

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Field of Study: Pavement materials

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My parents for their love,
My first love, NUHA, for her endless help and continuous support,
My kids,
for their sweet smiles that have inspired me life and energy towards my work.

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LIST OF ABBREVIATIONS AND SYMBOLS

AC : Asphalt concrete

ASTM : American Society of Testing Materials

AMWFA: Artificial marble waste fine aggregate

CRM : Crumb rubber modifier

CTR : Coefficient of Temperature Susceptibility

DSR : Dynamic shear rheometer

FHWA Federal Highway Administration

G* : Complex shear modulus

G’ : Storage shear modulus

G’’ : Loss shear modulus

HMA : Hot mix Asphalt

IDT: Indirect Tensile Test

ITFT : Indirect Tensile Fatigue Test

PAV ; Pressure aging vessel.

PI : Penetration Index

Mr: Resilient Modulus

RTFOT : Rolling thin film oven test

SAMI’s : Stress Absorbing Membrane Interlayer’s

SBR : Styrene Butadiene Rubber

SBS : Styrene Butadiene Styrene

SHRP : Strategic Highway Research Program

SMA : Stone Mastic Asphalt

δ : Phase angle

CHAPTER 1

INTRODUCTION

1.1 Introduction

Roadways are considered as one of the most important elements of infrastructure. They play an essential role in our daily lives for the present and in the future. Thus, highway construction engineers must consider the primary user's requirements of safety as well as the economy. To achieve this goal, highway construction designers should take into account three fundamental requirements which include; environmental factors, traffic flow and asphalt mixtures materials. In specific, there are two main factors which affect the performances of asphalt mixture which are the selected binder and mix composition (Mashaan, 2012). In asphalt mixture, bitumen as a binder serves two major functions in road pavement; first, to hold the aggregates firmly and second, to act as a sealant against water. However, due to some distresses like fatigue failure, the performance and durability of bitumen are highly affected by changes with time in terms of its characteristics which can lead to the cracking of pavement (Mahrez, 1999).

In general, road pavement distresses are related to binder and asphalt mixtures properties. Moisture damage, rutting and fatigue cracking are among the major distresses that lead to permanent failure of the pavement surface. The dynamic properties and durability of conventional binder, however, are deficient in resisting pavement distresses. Hence, the task of current asphalt researchers and engineers is to look for different kinds of polymer modified bitumen that has good rheological properties which directly affects the asphalt pavement performance (Mashaan, 2012).

The use of commercial polymers such as Styrene Butadiene Styrene (SBS) and Styrene Butadiene Rubber (SBR) in pavement construction will increase the construction cost as they are highly expensive materials. However, with the use of alternative materials such as crumb rubber modifier (CRM), it will definitely be environmentally beneficial, and not only it can improve the bitumen binder properties and durability, but it also has the potential to be cost-effective (Mashaan, 2012). As illustrated in a number of studies, the use of crumb rubber modifier shows enhancing in the fatigue resistance (Raad and Saboundjian, 1998; Soleymani *et al*., 2004; McGennis, 1995; Biliter *et al*., 1997a,b; Hamed, 2010). The improved performance of bitumen with CRM compared with conventional bitumen has mainly resulted from improved rheological properties of the rubberised bitumen. Aflaki and Memarzadeh (2011) have investigated the effects of rheological properties of crumb rubber on fatigue cracking at both low and intermediate temperatures using different shear methods. The results displayed that the high shear blending has more effect than the low shear blend on low temperatures properties.

Stone matrix asphalt (SMA) is a gap-graded asphalt mixture that has gained popularity world-wide. SMA was first developed in Germany during the mid 1960s to provide maximum resistance to rutting caused by the studded tyres on road (Brown and Hemant, 1993). Earlier in the 1990s, SMA technology was widely used in United State, however, most researchers' reports highlighted the mixtures great possibility in rutting resistance, but ignored any potential fatigue resistance of SMA (Ratnasamy *et al*., 2006). Due to the nature of SMA mixes (gap-graded) and the relatively large proportion of asphalt content, stabilisation is required to inhibit draindown of bitumen. These requirements can be achieved by adding fibre or polymer modifier, since commercial polymer is not economical in terms of usage (Hamed, 2010), therefore using recycled polymer such as CRM to the mixture has been found to be more economical and environmental- friendly (Mashan,2012).

1.2 Problem Statement

The cause of damage to road surfacing is quite often traced to the adhesion failure. The weather conditions in Malaysia, leads to variation of temperature of about 55^{o}C at the surface to 25^{o}C at the subgrade during hot days. Further, the moisture content is approximately 20 % between the verge and the subgrade on rainy days. As a result, the presence of the moisture and the infiltration of water in the pavement are major causes for the deterioration even with the absence of traffic loading (Abdullah, 1996). Many researchers concur that there is a strong reason for higher quality bituminous materials to overcome this issue (Mahrez, 2008; Hamed, 2010). Due to an increase in service traffic density, axle loading and low maintenance services; road structures have deteriorated and are therefore subjected to failure more rapidly. To minimise the damage of pavement such as resistance to rutting and fatigue cracking, asphalt mixture modification is required.

Virgin polymer offers the possibility of producing mixtures that can resist both rutting and cracking. Thus, using recycled polymer such as crumb rubber is a good alternative and inexpensive. Also, it is considered as sustainable technology, i.e. "*greening asphalt*" which would transform unwanted residue into a new bituminous mixture highly resistant to failure. Thus, utilising crumb rubber obtained from scrap automobile tyre is not only beneficial in terms of cost reduction but also has less ecological impact in keeping the environmental clean and to achieve better balance of natural resources.

1.3 Study Aim and Objectives

The primary aim of this study is to investigate the effect of adding crumb tyre rubber as an additive on SMA mixture performance properties. Therefore to achieve the main aim of this study, the following objective tasks were performed:

(i) To investigate the influence of crumb rubber modifier content on the physical and rheological properties of bitumen binders.

(ii) To investigate the fatigue resistance of modified binder in term of fatigue factor (G^* sin (δ)) at intermediate temperatures after long service life (after PAV test).

(iii) To assess the resilient modulus properties of Stone Mastic Asphalt (SMA) mixtures produced with and without the crumb rubber.

(iv) To investigate the fatigue resistance properties of SMA mixture reinforced with crumb rubber.

1.4 Scope of Study

The scope of the study can be outlined as below:

(i) Preparation of rubberised bitumen binder using five concentrations of crumb rubber (6, 12, 16 and 20%), respectively by binder weight.

(ii) Preparation of rubberised SMA mixtures using a wet mixing process.

(iii) Testing of the physical properties of all rubberised bitumen binder samples for different CRM content, by penetration test, softening point test, ductility test, elastic recovery test and Brookfield viscosity test.

(iv) To determine and test the fatigue factor (G^* sin δ) of rubberised bitumen after long term of service i.e10 years and above (after PAV test) at intermediate temperatures using the DSR test.

(v) Testing of the rheological properties of all samples for different content of CRM by temperature sweep test using the DSR data.

(vi) Testing of rubberised SMA mixture - Marshall test, indirect tensile resilient modulus test (IDT), indirect tensile fatigue test (ITFT).

1.5 Organisation of the Dissertation

- Chapter 1: This chapter intends to introduce the reader to the topic and title of the researcher, as well as the problem statement and the motives behind the study. The main objectives of the study are also presented in this chapter.
- Chapter 2: In this chapter, literature review on the use of crumb rubber in reinforcement of stone mastic asphalt (SMA) will be presented and illustrated. It will also, include review on the effects of CRM on the stiffness and fatigue resistance of road pavement construction.
- Chapter 3: This chapter illustrates the basic experimental and the detailed test approaches used in this research study to investigate the performance properties of crumb rubber modified bituminous mixes. Testing was advocated on the bitumen binder with and without CRM as well as on the bituminous mixtures.
- Chapter 4: This chapter presents the results and analysis of rubberised asphalt binder physical and rheological properties and reinforced SMA mixtures performance properties as well.
- Chapter 5: This is the final chapter, which concludes the discussion by presenting a summary of the main points discussed in the previous chapters and provides the major results from the study, which are supported by the relevant literature employed for substantiation of the claims along with the results of the experiments and tests.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Roadways are an integral aspect of roadway transportation. Hence, the construction and maintenance of road pavements should be long-lasting as they have a significant impact on the economic vitality of a nation. The primary reasons for the deteriorated conditions of roads include the increase in overall traffic and poor asphalt binder quality due to high-tech refining processes and climatic changes (Peralta, 2009; Hamed, 2010).

In road pavement construction, the use of crumb rubber in the reinforcement of asphalt is considered as a smart solution for sustainable development by reusing waste materials. It is believed that crumb rubber modifier (CRM) could be an alternative polymer material in improving asphalt mixture performance properties. In this chapter, literature review on the use of crumb rubber in reinforcement of stone mastic asphalt (SMA) will be presented and illustrated. It will also, include review on the effects of CRM on the stiffness and fatigue resistance of road pavement construction.

2.1.1 Asphalt Concrete Pavement

The design of bituminous mixture involves the selection and proportioning of materials to obtain the desired properties in the finished product. Asphalt mixture is designed to resist rutting, fatigue, low temperatures cracking and other distresses. The serious distresses associated with flexible pavements are cracking, which occurs at intermediate and low temperatures and permanent deformation, which occurs at high temperatures. These distresses reduce the services life of the pavement and elevate the maintenance costs (Hamed, 2010). The overall objective for the design of bituminous paving mixes is to determine an economical blend and gradation as well as bitumen that will yield a mix having sufficient bitumen to ensure a durable pavement, sufficient stability, sufficient voids in the total compacted mix to allow for a slight amount of additional compaction under traffic loading without flushing and sufficient workability to permit efficient placement of the mix without segregation (Mahrez, 2008).

Asphalt cement (bitumen) binds the aggregate particles together, enhancing the stability of the mixture and providing resistance to deformation under induced tensile, compressive and shear stresses. The performance of asphalt mixture is a function of asphalt cement, aggregate and its volumetric properties. In recent years, there has been a rapid increase in using additives in asphalt mixtures to improve its properties (Mahrez, 2008). Current research focuses on increasing the fatigue resistance of asphalt concrete mixtures. Polymer modification is perceived to be a potential alternative to improving the fatigue resistance of asphalt binder and mixtures.

2.2 Bituminous Materials Analysis and Chemical Components

2.2.1 Bitumen definition

Bitumen is a dark black semisolid material, obtained from the atmospheric and vacuum distillation of crude oil during petroleum refining which is then subjected to various other processes (Croney and Croney, 1992). It is considered as a thermoplastic visco-elstic adhesive which is used for road and highway pavement engineering, primarily because of its good cementing power and waterproof properties (Rozeveld *et al*., 1997). The analysis of bitumen indicates that the mix is approximately 8-11% hydrogen, 82-86% carbon, 0-2% oxygen, and 0-6% sulphur by weight with minimal amounts of nitrogen, vanadium, nickel and iron. In addition, bitumen is a complex mixture of a wide variety of molecules: paraffinic, naphthenic and aromatics including heteroatoms. This complexity makes the prediction of bitumen properties particularly difficult (Rozeveld *et al*., 1997).

2.2.2 Bitumen production

Most producers use atmospheric or vacuum distillation to refine the bitumen. While there is some solvent refining and air blowing utilised, they are clearly of secondary importance (Youtcheff and Jones,1994). Based on chemical analysis, crude oil may be predominantly paraffinic, naphthenic or aromatic with combinations of paraffinic, naphthenic being most common. There are approximately 1500 different crudes produced globally. According to the yield and quality of the resultant product, only a few of these, presented in Figure 2.1 (compositions are in percentage of weight and represent the +210 °C fraction), are considered appropriate for the manufacture of bitumen (Read and Whiteoak, 2003; McLean and Kilpatrick, 1997).

Refining bituminous and mixed base crude oil results in petroleum bitumen distinguishing the nature and the grades of the desired bitumen. The most commonly used method and probably the oldest method is the atmospheric vacuum distillation of suitable crudes which produce straight-run residual bitumen. The air blowing process is done to give oxidized or semi-blown products, which inherently upgrades of low-grade bitumen. Crude heavy fractions (that constitute bitumen) are defined as molecules containing more than 25 carbon atoms (C25), which increases with the boiling point (Figure 2.2) as well as the molecular weight, the density, the viscosity, the refractive index (aromaticity) and the polarity (contents of heteroatoms and metals) (Merdrignac and Espinat, 2007). These fractions are enriched in highly polar compounds such as resins and asphaltenes. When compared to the crude or lighter fractions, the highly polar compounds are composed by various chemical species of different aromaticity, functional heteroatoms and metal contents (Merdrignac and Espinat, 2007; Altgelt and Boduszynski, 1994).

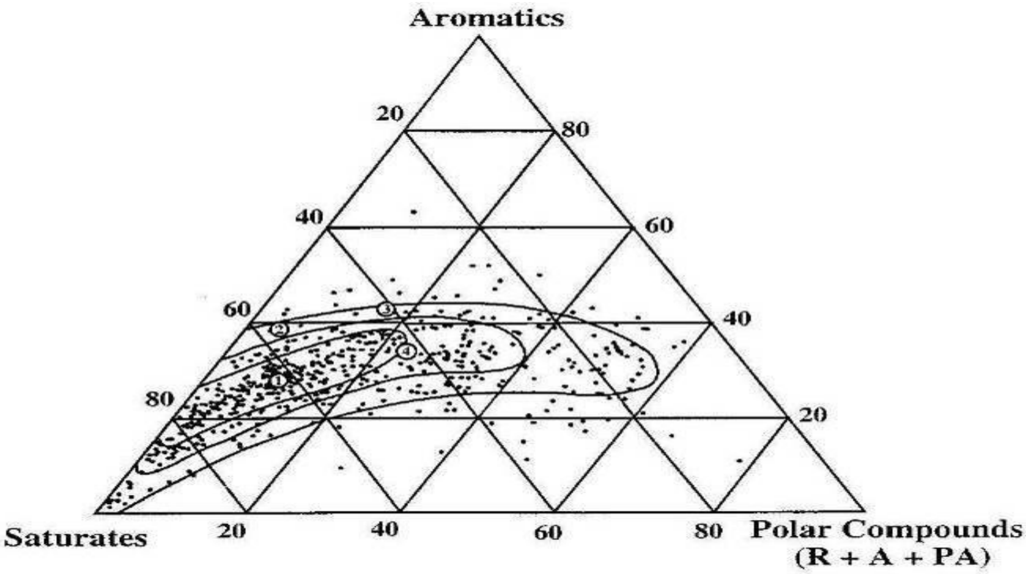


Figure 2.1: Compositional representation on ternary diagram of 640 different crudes (McLean and Kilpatrick, 1997)

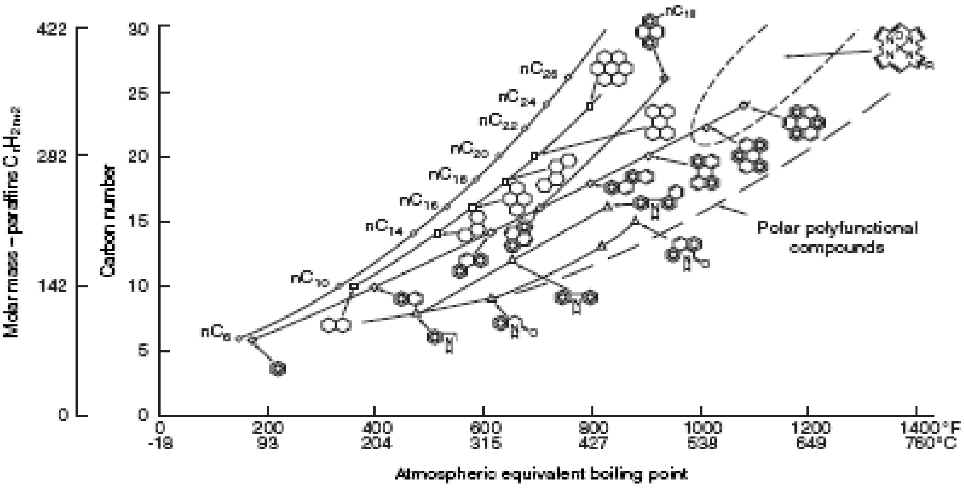


Figure 2.2: Evolution of molecular weights and structures as a function of the boiling point (Altgelt and Boduszynski, 1994).

2.2.3 Bitumen chemical component

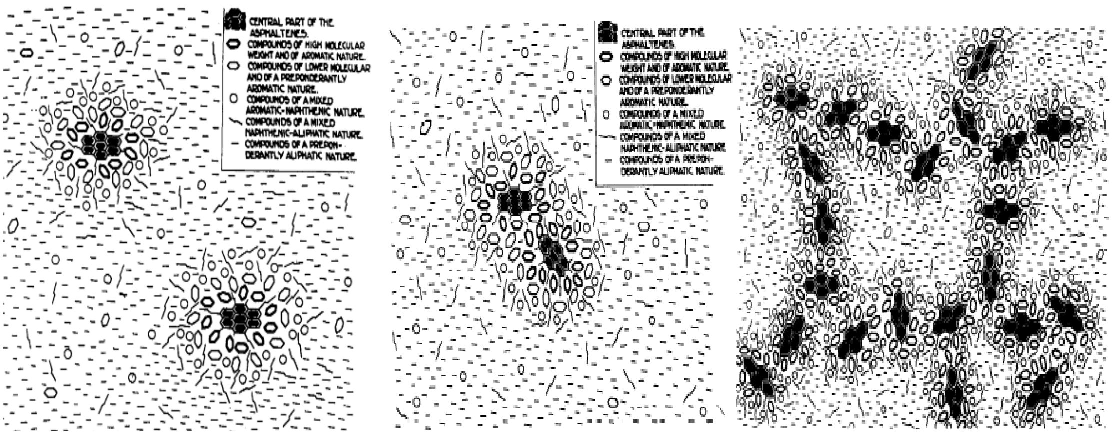
The chemical component of bitumen can be identified as asphaltenes and maltenes. The maltenes can be further sub-divided into three groups of saturated, aromatic and resins. The polar nature of the resins provides the bitumen its adhesive properties. They also act as dispersing agents for the asphaltenes. Resins provide adhesion properties and ductility for the bituminous materials. The viscous-elastic properties of bitumen and its properties as a paving binder are determined by the differing percentages between asphaltenes and maltenes fraction (Navarro, 2002; Lewandoweski, 1994; Dongre *et al*., 1996).

The complexity, content of heteroatom, aromatic and increase of molecular weight are in the order of S< A< R< A (**S**aturates < **A**romatics < **R**esins < **A**sphaltenes) (Claudy *et al*., 1991). A study by Loeber *et al*. (1998) illustrated the rheological properties related to bitumen colloidal behaviour. Bitumen possesses a strong temperature dependence on rheological properties organised by the interaction of individual constitution (asphaltenes, resins, aromatics, saturates). Loeber *et al*. (1998) reported that an increase in one of these constitutions would change the structure and rheological behaviour of bitumen. Thus, bitumen with high asphaltenes/resins ratio, would lead to a network structure with more rigidity and elasticity (low in phase angle and high in complex shear modulus), unlike the case of bitumen with high resins/asphaltenes ratio which leads to high viscous behaviour. Bituminous materials with high asphaltenes content will have higher softening points, higher viscosities and lower penetrations.

2.2.4 Bitumen Polarity

Bitumen has another important property which is polarity, which is the separation of charge within a molecule. Polarity is an important factor in the bitumen system because it refers to molecules managing themselves into preferred orientations.

According to Robertson (1991), most of the naturally occurring heteroatoms, nitrogen, sulfur, oxygen and metals are strongly dependent on polarity within these molecules. Also, oxidation products upon aging are polar and further contribute to the polarity of the entire system. The physicochemical properties have an obvious significant effect on bitumen and each reflects the nature of the crude oil used to prepare it. Pfeiffer and Saal (1940) suggested that bitumen dispersed phases are composed of an aromatic core surrounded by layers of less aromatic molecules and dispersed in a relatively aliphatic solvent phase. However, they do not point out that there are distinct boundaries between bitumen dispersed and solvent phases, as found in soap micelles. However they suggest that it ranges from low to high aromaticity that is from the solvent phase to the centres of the entities constituting the dispersed phase (Figure 2.3).



(a) (b) (c)

(a)(Sol-type bitumen), (b) Flocculated asphaltene micelles, (c) Gel structure of an asphaltic bitumen

Figure 2.3: Bitumen colloidal model (Pheiffer and Saal, 1940)

According to Robertson, R.E. (1991) and Jones and Kennedy (1992) bitumen is a collection of polar and non-polar molecules:

a. The polar molecules are associated strongly to form organised structures and represent a more stable thermodynamic state.

b. The non-polar model has the ability to dissociate the organised structure, but again there are possible variations from bitumen sources and its viscous behaviours are highly dependent on the temperature, as shown in Figure 2.4

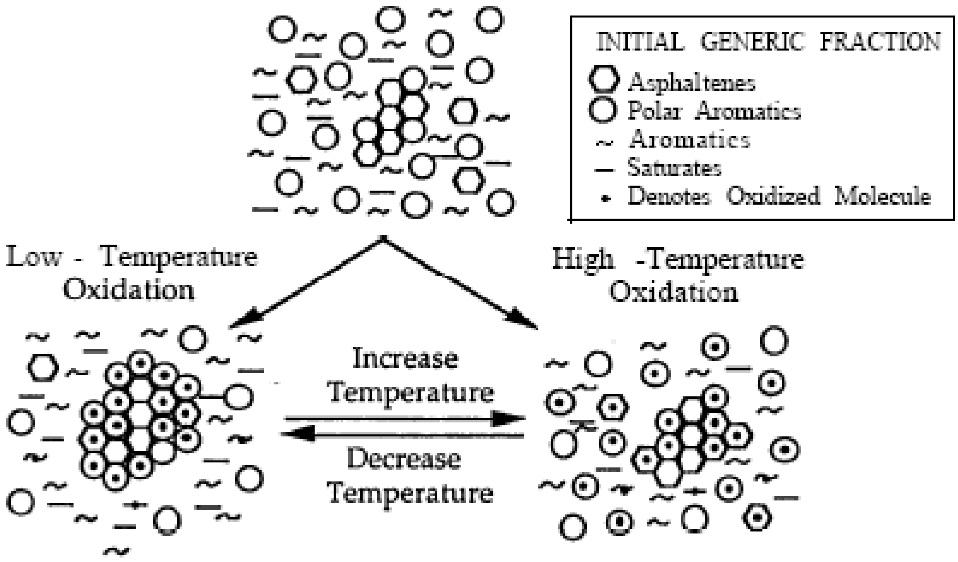


Figure 2.4: Bitumen model and the chain building (Jones and Kennedy, 1992).

Using current day technology, the morphology of bitumen has been studied in order to verify the bitumen structure. Thus, Figure 2.5 presents the topographic atomic force microscopy (AFM) images of two different grads of bitumen, showing a flat background where another phase is dispersed (Masson *et al*., 2006).

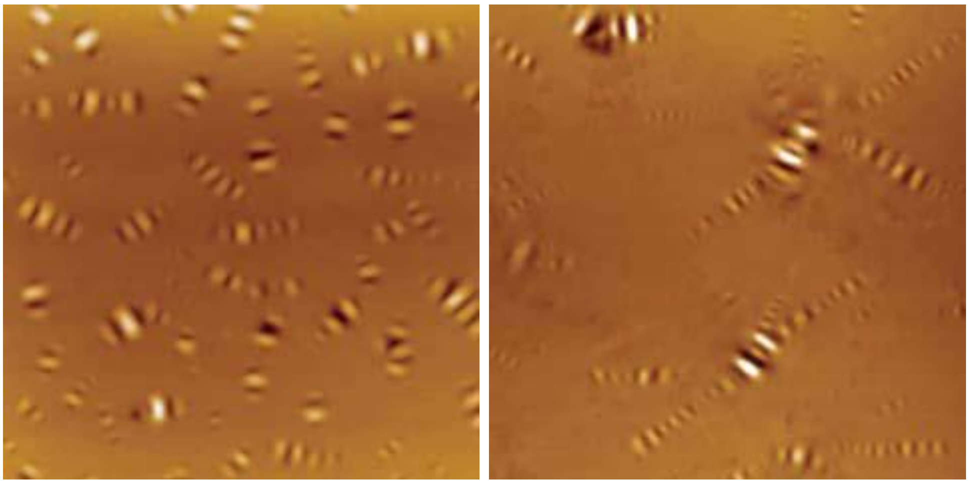


Figure 2.5 : Topographic AFM images of two bitumen (Masson *et al*., 2006)

In the image of the left side of Figure 2.5, the dispersed phase displays a range of pale and dark lines frequently regarded as “bees” or “bee structures”. However with the image on the right side where the bee-like structures are not independent of one another, they are substituted by “multiarm star-shapes” (Masson *et al*., 2006).A dispersed phase, with a “bee-like” appearance as presented in Figure 2.5, is attributed to asphaltenes; however, results of this study support the finding of previous study (Pauli *et al*.,2001). However, no correlation was found between the AFM morphology and the composition made up of asphaltenes, polar aromatics, naphthene aromatics and saturates (Masson *et al*., 2006).

2.3 Bituminous Modification and Reinforcement

Road surface undergo functional deterioration once they are open to traffic. Most common amongst these are cracks, which if not provided early care may lead to a more severe structural deterioration which would require more extensive treatment of the problem and may even lead to a reconstruction. A simple solution in overcoming functional failures in pavement is to overlay the surfacing. The installation of an overlay is usually preferred to reconstruction, not only because it is a cheaper option, but also because an overlaid highway can be put back in service in a matter of hours with the final product looking excellent (Mahrez, 2008). At this juncture, an important question may be raised about an alternative solution besides overlaying to overcome pavement cracking failure. The solution may in fact be pavement reinforcement.

The term reinforced pavements refers to the use of one or more reinforcing layers within the pavement structure. Another application of pavement reinforcement is the use of reinforcement elements in bituminous overlays to provide an adequate tensile strength to the bituminous layer and to prevent failures of the pavement such as reflection cracking. Thus the difference between the two applications is that the first application is used as measure to overcome the distress failure which has already occurred in the pavement, while the second application is used as measure to prevent the existence of such failure (Mahrez, 2008). Worldwide, there are many additives used as reinforcing material into the bituminous mixes, such as styrene butadiene styrene (SBS), synthetic rubber- styrene- butadiene (SBR), natural rubber, fibre and crumb rubber modifier (CRM) (Mashaan ,2012 ; Mahrez, 2008).

2.4 Crumb Rubber in road pavement , analysis, history and use

Crumb rubber or waste tyre rubber, is a blend of synthetic rubber, natural rubber, carbon black, anti-oxidants, fillers and extender type of oils which are soluble in asphalt mixture. Rubberised asphalt is obtained by the incorporation of crumb rubber from ground tyres in asphalt binder at certain conditions of time and temperature using either dry process (method that adds granulated or crumb rubber modifier (CRM) from scrap tires as a substitute for a percentage of the aggregate in the asphalt mixture, not as part of the asphalt binder as shown in Figure 2.6, or wet processes (method of modifying the asphalt binder with CRM from scrap tires before the binder is added to form the asphalt concrete mixture) as shown in Figure 2.7.

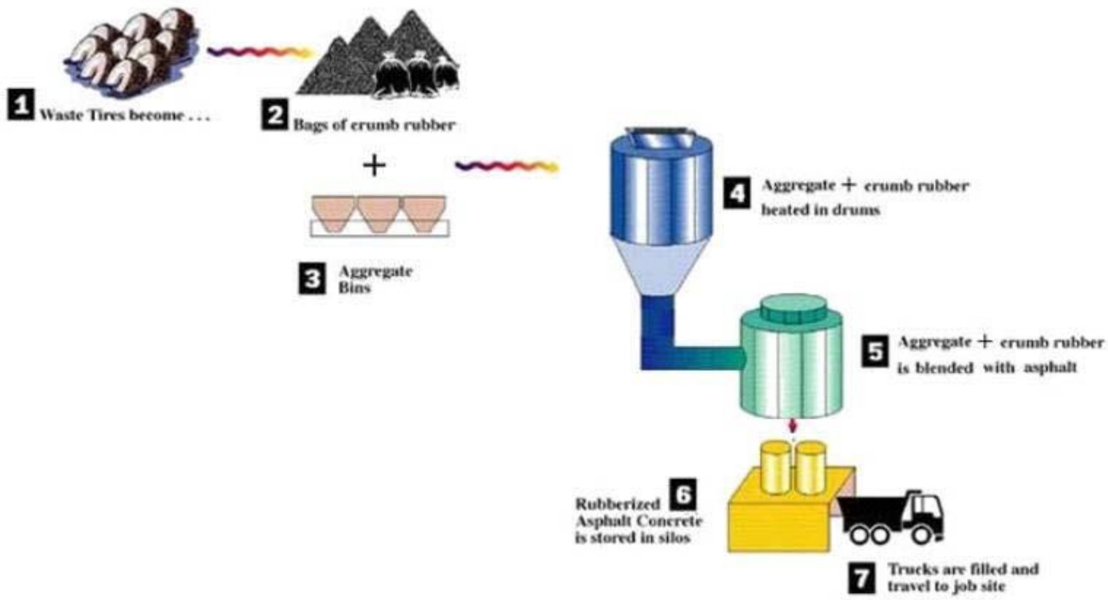


Figure 2.6: The main stages of the crumb rubber dry process (Field blend) (Source: Utah Department of Transportation, 2003).

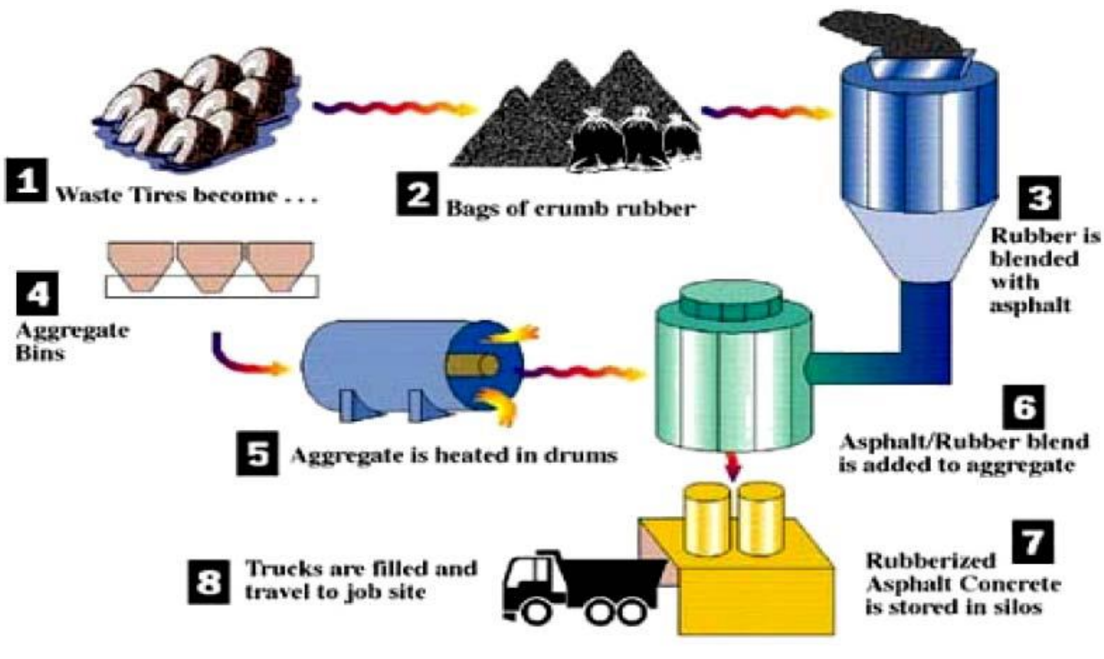


Figure 2.7: Illustration of the main stages of the crumb rubber wet process (Field blend) (Source: Utah Department of Transportation, 2003).

There are two different methods in the use of tyre rubber in bitumen binders; first, is by dissolving crumb rubber in the bitumen as binder modifier. Second, is by substituting a portion of fine aggregates with ground rubber that does not completely react with bitumen (Huang *et al.,* 2007). In 1840s, the earliest experiments had involved incorporating natural rubber into asphalt binder to increase its engineering performance properties. The process of asphalt modification involving natural and synthetic rubber was introduced as early as 1843 (Thompson, 1979). In 1923, natural and synthetic rubber modifications in bitumen were further improved (Charania *et al*., 1991; Isacsson and Lu 1999; and Yildrim, 2007). According to Yildrim, (2007) the development of rubber-bitumen materials being used as joint sealers, patches and membranes began in the late 1930s. The first attempt to modify bituminous binders by adding rubber was made in 1898 by Gauedmberg , who patented a process for manufacturing rubber-bitumen. France was then given credit for installing the first road with a rubberised bituminous surfacing material (Mahrez, 1999).

In 1950, the use of scrap tyre in asphalt pavement was reported (Hanson *et al*., 1994). In the early 1960's, Charles Mc Donald working as head Material Engineer for the city of Phoenix, Arizona, he found that after completing the mixing of crumb rubber with the conventional bitumen and allowing it to blend for mix duration of 45 – 60 minutes, there were new material properties produced. There was swelling in the size of the rubber particles at higher temperatures allowing for higher concentrations of liquid bitumen contents in pavement mixes (Larsen *et al.,* 1988; Huffman, 1980).

The application of rubber-modified asphalt pavement started in Alaska in 1979. Placement of seven rubberised pavements totalling 4 lane-km using the Plus Ride dry process between 1979 and 1981 was reported. The performance of these sections in relation to mixing, compaction, durability, fatigue, stability and flow, and tyre traction and skid resistance were described. Rubberised bitumen using the wet process was first applied in Alaska in 1988 (Raad and Saboundjian, 1998).

Around 1983 in the Republic of South Africa, rubberised asphalt and seals were first introduced. Over 150 000 tons of asphalt were paved over the first 10 years. From the evaluation, it was concluded that the bitumen rubber stress absorbing membrane interlayer's (SAMI's) and asphalt performed above expectations. The bitumen rubber overlays out-performed the conventional asphalts, under identical conditions, by a large margin. Bitumen rubber asphalts and SAMI's are especially suited for highly trafficked roads with pavements in structural distress and where overlays will eliminate re-working options in congested traffic situations (Katman, 2006). Lundy *et al*. (1993) presented three case studies using crumb rubber with both the wet process and dry process at Mt. St. Helens Project, Oregon Dot and Portland Oregon. The results showed that even after a decade of service, crumb rubber products have excellent resistance to thermal cracking. Although, rubberised asphalt can be built successfully, quality control ought to be maintained for good performance.

Rubber Pavement Association found that using tyre rubber in open- graded mixture binder could decrease tyre noise by approximately 50%. Also, in spray applications, rubber particles of multiple sizes had better sound absorption (Zhu and Carlson, 2001). Moreover, another advantage of using asphalt rubber is to increase the life- span of the pavement. However, recommendations were made to assess the cost effectiveness of rubberised asphalt (Huang *et al.*, 2007).

The benefits of using crumb rubber modified bitumen are as listed below:

a- Lower susceptibility to varying temperature on a daily/ seasonal basis.

b- More resistance to deformation at higher pavement temperature.

c- Improved age resistance properties.

d- Higher fatigue life for mixes, and better adhesion between aggregate and binder.

Ever since then, the use of crumb rubber has gained interest in pavement modification as it is evident that crumb rubber can improve the bitumen performance properties (Brown *et al*., 1997; Maupin, 1996; Stroup- Gardiner *et al*., 1996; Hanson *et al*., 1994; Charania *et al*., 1991).

2.4.1 Crumb Rubber Grinding Process

Crumb rubber is made by shredding scrap tyre, which is a particular material free of fibre and steel. The rubber particle is graded and found in many sizes and shapes (see Figure 2.8). To produce crumb rubber, initially it is important to reduce the size of the tyres. There are two techniques to produce crumb rubber: ambient grinding and the cryogenic process (Becker *et al*., 2001). In the crumb rubber market, there are three main classes based on particle size:

a. Type 1 or Grade A: 10 mesh coarse crumb rubber;

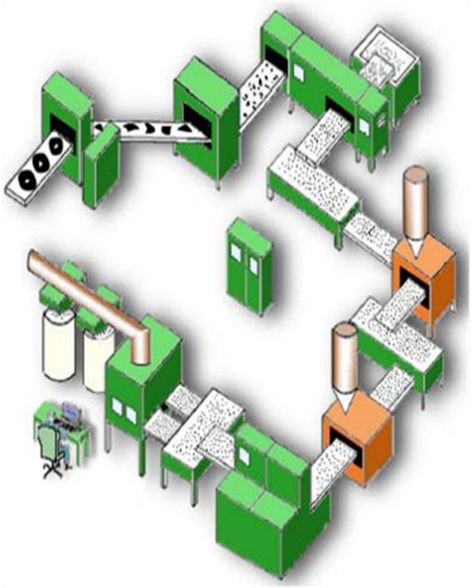
b. Type 2 or Grade B: 14 to 20 mesh crumb rubber;

c. Type 3: 30 mesh crumb rubber.

Mesh size designation indicates the first sieve with an upper range specification between 5% and 10% of material retained.

Figure 2.8: Different type of crumb rubber based on particle size (Bandini, 2011).

Ambient grinding process can be divided into two methods: granulation and cracker mills. Ambient describes the temperature when the waste tyres rubber size is reduced. The material is loaded inside the crack mill or granulator at ambient temperature. Whereas, cryogenic tyre grinding consists of freezing the scrap tyre rubber using liquid nitrogen until it becomes brittle, and then cracking the frozen rubber into smaller particles with a hammer mill. The resulting material is composed of smooth, clean, flat particles. The high cost of this process is considered a disadvantage due to the added cost of liquid nitrogen (Mashaan, 2012). Figure 2.9 displays the particular CRM resulted from ambient grinding process and cryogenic grinding process.



(a) Waste tyre (b) Grinding processes

(b) CRM from ambient grinding (d) CRM from cryogenic grinding

Figure 2.9: CRM production steps using waste tyre rubber; a,b,c,d (Mashaan, 2010)

2.4.2 Properties of Tyre Rubber

The main characteristics of rubber is its property of high elasticity which allows it to undergo large deformations from which almost complete, instantaneous recovery is achieved when the load is removed (Beaty,1992). This property of high elasticity derives from the molecular structure of rubber. Rubber belongs to the class of materials known as polymers and is also referred to as an elastomer.

The properties of an elastomer rubber are:

a. The molecules are very long and are able to rotate freely about the bonds joining neighbouring molecular units.
b. The molecules are joined, either chemically or mechanically, at a number of sites to form a three dimensional network. These joints are termed cross-linked.
c. Apart from being cross-linked, the molecules are able to move freely past one another, i.e the Van der Waal’s forces are small.

Similar to bitumen, rubber is a thermoplastic, visco-elastic material, whose deformation response under load is related to both temperature and rate of strain. However, the deformation of rubber is relatively incentive to temperature change where at both low rates of strain and at temperature well above the ambient, the material remains elastic. The wider range of elastic behaviour of rubber compared to that of bitumen largely results from the cross-linking of the long rubber molecules. Rubber is also much more ductile than bitumen at low temperatures and high loading rates.

2.4.3 Mechanism of Dispersal of Rubber in Bitumen

Previous researchers found that when incorporating the rubber powder into bitumen, the crumb rubber will degrade and its effectiveness is reduced on prolonged storage at elevated temperatures (Mahrez, 1999). The improvements effected in the engineering properties of rubberised bitumen depend largely on the particle dispersion, the molecular level dissolution and the chemical interaction of rubber with bitumen. Temperature and time of digestion are highly important factors affecting the degree of dispersion for slightly vulcanized and unvulcanized natural rubber. For instance, the optimum digestion time for a slightly vulcanized rubber powder is 30 minutes at 180ºC and 8h at 140ºC (O’Flaherty, 1988). On the other hand, unvulcanized rubber powder requires merely 10 minutes digestion time at 160ºC to achieve the same results. The easy dispersion of unvulcanised powder is because of the state of the rubber and fineness of the powder (95 per cent passing 0.2mm sieve). Vulcanised powders are harder to disperse because they are coarser (about 30 per cent retained on 0.715mm sieve and 70 per cent retained on 0.2mm sieve) and also due to vulcanization.

2.4.4 Effect of Heating on Rubberised Bitumen

At a temperature above 120˚C, it is found that there is a reduction in effectiveness of rubberised binders. Depending on the type of material, the effect of heating on rubberised binders is therefore of practical importance (Mahrez, 1999). The change in effective rubber content is found to be variable due to the concurrent degradation of the rubber and vulcanisation which boost increased effectiveness and high elasticity. The reduction in effectiveness reaches a fairly steady value after some time probably due to the absence of further oxygen.

2.4.5 Temperature Susceptibility of Rubberised Bitumen

The temperature susceptibility was defined by Dobson (1969), quoted by Katman (2006), directly as a ratio of Newtonian viscosities at 25 ºC and 60ºC. The binder content in the bituminous mix is usually less than 7% but it plays a very significant role in the overall properties of the composite material. It strongly affects both the load spreading capability and resistance to distortion under heavy traffic. The deformation response of a binder in a mix under load depends on its temperature sensitivity; the temperature range is subjected to, rate of strain and the geometry of binder between the aggregate particles. Therefore, it is logical to use a binder with lower temperature susceptibility, particularly, when the range of working temperatures is very high (Mahrez, 1999).

The concept of the penetration index (PI) was introduced by Pfeiffer and Van Doormal (1936) to measure both, the binder's temperature susceptibility and, in particular, its rheological type in terms of deviation from Newtonian behaviour. PI is obtained from the relationship:

d log pen/dT = (20-PI / 10+PI) / 50 ....(1) (Pfeiffer and Van Doormal, 1936)

Normal road paving asphalts have a PI value between -1 and +1. Paving asphalts with PI below -2 are substantially Newtonian and characterised by brittleness at low temperature. Paving asphalts with PI above +2 are far less temperature susceptible, less brittle at low temperature, indicate marked time dependent elastic properties and show deviations from Newtonian behaviour, especially at large strain rates (Fernando and Guirguis, 1984). The coefficient of temperature susceptibility (CTS) based on viscosity measurements in the temperature range 60º-80ºC were used to assess the behaviour of rubberised binder with temperature.

CTS = log (log η_{T1} / log η_{T2}) / log (T_2 / T_1)................(2) (Fernando and Guirguis, 1984)

Where T = Temperature (º F); η_{T1} and η_{T2} are viscosities measured at temperatures T_1 and T_2 (º F). In 1984, a research study found that 4% rubber is effective in reducing the temperature susceptibility of virgin binders by a factor of at least two. Hence, rubberised bitumen is more resistant to rapid changes in temperature (Fernando and Guirguis, 1984).

2.4.6 Physical Behaviour of Rubberised Bitumen

Mahrez (1999) investigated the properties of rubberised bitumen prepared by physical blending of bitumen 80/100 penetration grade with different crumb rubber content and various aging phases. The results of penetration values decreased over the aging as well as before aging by increasing the rubber content in the mix. Also, the modified binders showed lower penetration values than unmodified binders. Another study by Kumar *et al*. (2009) on penetration change was conducted using bitumen 80/100 and 70/100 penetration grade mixes with different crumb rubber percentage. The results showed a significant decrease in the penetration values of modified binder due to high crumb rubber content in the binders.

According to Jensen and Abdelrahman (2006), elastic recovery property is very important in both fatigue and rutting resistance selection and evaluation. Elastic recovery is a property that indicates the quality of polymer components in bitumen binders. Oliver (1981) concluded from his study, that the elastic recovery of rubberised bitumen binders leads to an increase as the rubber particle size decreases.

It was found that rubber types could affect the force ductility properties at 4 ºC (Rosner and Chehovits, 1982). Bitumen-rubber modification resulted in a better rutting resistance and higher ductility. However, the modified binder was susceptible to decomposition and oxygen absorption. There were problems of low compatibility because of the high molecular weight. Furthermore, it was found that recycled tyre rubber decreases reflective cracking, which in turn increases durability. During compaction or mixing, low viscosity has been observed to resulting in lower stability values. Softening point refers to the temperature at which the bitumen attains a particular degree of softening (Mashaan, 2012).

Mahrez and Rehan (2003) claimed that there is a consistent relationship between viscosity and softening point at different aging phases of rubberised bitumen binder. Also, it is reported that the higher crumb rubber content leads to higher viscosity and softening point.

2.5 Durability and Aging of Pavement Materials

In the paving design mixture, the general practice is to arrive at a balanced design among a number of desirable mix properties, one of which is durability. Durability is the degree of resistance to change in physical-chemical properties of pavement surface materials with time under the action of weather and traffic. The life of a road surfacing will depend primarily on the performance of the binder provider, the mix design and construction techniques (Mahrez, 1999). Asphalt hardening can lead to cracking and disintegration of the pavement surface. The rate of hardening is a good indicator of the relative durability. Many durability tests are based on the evaluation of resistance to asphalt hardening.

Aging is a key factor influencing the performance and characteristics of bitumen binder. There are several other factors which contribute to the hardening of the bitumen such as oxidation, volatilisation, polymerisation and thixotropy. Bitumen being an organic compound is capable of reacting with oxygen found in the environment. With the reaction of oxidation, the bitumen composite develops a rather brittle structure, termed as age hardening or oxidative hardening (Peterson, 1984).

2.6 Failure of road pavement: Cracking and deformation

Two kinds of loading are of specific importance in tandem with the performance of bituminous surfacing. One is due to vehicles loads passing over the road surfacing, while the second is due to thermal contraction in relation to temperature changes (Oliver, 1981).

Vehicle loading can lead to distress at either end of the range of pavement surface temperatures. At increased pavement temperatures, the binder can be extremely fluid and probably will not resist the plucking and shearing action of vehicle tyres. At low pavement temperatures, the binder can be so hard (particularly after a long period of service) that vehicle loading causes brittle fracture of the binder films. The explanation to this phenomenon is thought to be due to the theory of “Normal Stresses” (Wiesenberger effect) which applies to visco-elastic material such as a bitumen/scrap rubber mixture. This theory covers normal stress differences, which are forces that develop normal (i.e. perpendicular) to the direction of shear (Mahrez, 2008; Hamed 2010).

According to the theory, a visco-elastic material forced through an open tube expands normal to the axis of the tube on leaving the tube. In a cracked pavement, the vertical loads are applied by the vehicle wheels which compel the bitumen binder to expand normally to the applied vertical load (horizontally) and thus fill up the cracks. Another reason is that if this bitumen’s mixture is stirred while hot with a stick in a container, the material will climb up the stick, rather than form a vortex as found in Newtonian type fluids (Oliver, 1981).

2.7 Performance of Modified Binder in Asphalt Concrete

The main objective of using modified binders in asphaltic mixture is to provide a cost effective solution in improving the resistance to permanent deformation of the surfacing materials at high temperature and under extreme loading conditions. Beaty (1992) has summarised the advantages of using rubberised asphalt which are:

(1) To improve low temperature ductility and hence the resistance of the bituminous mixture to brittle fracture.

(2) To improve the stiffness of the bituminous mixture at high temperatures so as to reduce permanent deformation and rutting under traffic loading.

(3) To improve the adhesion of the binder to the aggregate and thus improve resistance to stripping.

(4) To reduce the temperature susceptibility of the binder, i.e to decrease the loss of viscosity at high temperature and hence to reduce bleeding.

(5) To reduce or prevent reflection cracking when the material is laid over a cracked pavement structure.

(6) To improve the fatigue resistance of the bituminous mixture.

(7) To increase the strength of the bituminous mixture.

(8) To improve resilience and elastic recovery.

2.8 Marshall Stability Characteristics and Rubberised Asphalt

In relation to the plastic behaviour of materials, the stability of an asphaltic paving mixture is influenced by its internal friction, cohesion and inertia. The friction component of stability in turn is governed by size, shape, gradation and surface roughness of aggregate particles; inter granular contact, pressure due to compaction and loading, aggregate interlock caused by angularity and viscosity of the binder. The cohesion depends on variables such as the rheology of the binder, number of contact points, density and adhesion (Fernando & Mesdary 1988).

The results of Marshall Test by Samsuri (1997) indicated that incorporation of rubber increases the Marshall stability and quotient. The increase varied with the form of rubber used and the method of incorporating the rubber into bitumen. The Marshall stability for mixes containing rubber powders was increased more than two folds and the Marshall quotient increased by nearly three folds compared to the normal unmodified bituminous mix. Mixes produced using bitumen pre-blended with fine rubber powders showed the greatest improvement rather than mixes produced by direct mixing of rubber with bitumen and aggregates. Thus, pre-blending of bitumen with rubber is a necessary step in order to produce an efficient rubberised bitumen binder probably due to adequate and efficient rubber dispersions in the bitumen phase. The optimum binder content was selected based on Marshall Mix design method as recommended by (Asphalt Institute, 1990) uses five mix design criteria:

a. lower Marshall stability,
b. an acceptable average of Marshall flow,
c. an acceptable average of air void,
d. Percent voids filled with asphalt (VFA),and
e. Lower value of VMA.

2.8.1 Influence of Aggregate Gradation on Marshall Test

The mineral aggregate is bituminous concrete constituting about 95 percent of the mixture on a weight basic and about 85 percent on a volume basic. Characteristics of aggregate contributing to the properties of bitumen mixture would be gradation, particle surface texture, particles shape, cleanliness and chemical composition (Wayne and Roberts, 1988).

Investigations showed that the effect of aggregate maximum size on the modified Marshall test results resulted in mixtures with aggregate maximum size of 19mm leading to higher modified Marshall stability values and slightly decreased Marshall flow values than mixtures with aggregate maximum size of 38mm. However, the disparity between the results for the two mixtures was minimal. Also, the modified Marshall flow did not present any specific trend for the two mixtures (Wood and Mamlouk, 1981).

The aggregate maximum size had a marked effect on the amount of air voids and on the specific gravity of the specimens. Small percentages of air voids and higher values of air-cured specific gravity were obtained for mixture with 38mm of aggregate maximum size compared to mixture with 19mm of aggregate maximum size (Wood and Mamlouk, 1981). On the other hand, binder emulsion content showed a significant effect on the air voids and the specific gravity of the specimens. Increasing the binder emulsion content in the mixture filled the voids among aggregate particles and also allowed for more occurrence of compaction due to lubrication (Wood and Mamlouk, 1981).

2.8 .2 Influence of Compaction on Marshall Test

The stability values of the various mixes obtained using gyratory compaction were two to three times greater than the values obtained with Marshall Compaction .The flow values of the mixes obtained using gyratory compaction correlated with the stability values, where the maximum stability occurred the lowest with regards to the flow, while those obtained using the Marshall compaction were not consistent in this respect (Brennen, *et al*., 1983).

2.9 Rheology and Viscoelastic Properties of Rubberised Asphalt

Rheology is the study of deformation and flow of materials. It is the science knowledge that is related to all aspects of deformation of material under the influence of external stresses (Ferguson and Kemblowski, 1991).

Bitumen being a viscoelastic material has rheological properties which are highly sensitive to temperature as well as to the rate of loading. The most prominent problems of road pavement are rutting, fatigue cracking and thermal cracking, with respect to temperature. Zaman *et al*. (1995) found that the viscosity of asphalt increased with the addition of rubber. With the decrease of rubber, rubber-modified asphalt samples showed higher resistance against loading and were more uniformed. The degrees of shear-thickening and shear-thinning behaviour decreased with the increase in rubber in asphalt cement. Moreover, the liner viscoelastic functions (dynamic viscosity and storage modulus) were also elevated. The results indicated that the elastic behaviour of asphalt modified with 7.5% rubber improved yet there was minimal difference in the elasticity of asphalt modified with 5% and 7.55% rubber.

Piggott *et al*. (1977) stated that the vulcanized rubber had an enormous effect on the viscosity of the asphalt. The viscosity, measured at 95ºC, increased by a factor of more than 20 when 30% vulcanized rubber was added to the mixture. However, the devulcanized rubber had a rather minimal effect. The viscosity test also showed that when rubber was mixed with hot asphalt cement, there was no effect of gel formation.

Navarro *et al*. (2002) conducted a study on the rheological characteristics of ground tyre rubber-modified bitumen. The experiment compared the viscoelastic behaviour of five ground tyre rubber-modified with unmodified bitumen and polymer-modified

(SBS) bitumen. It was performed in a controlled-stress Haake RS150 rheometer. The study showed that rubber-modified bitumen improved viscoelastic characteristics and therefore resulted in higher viscosity compared to unmodified binders. Consequently, the ground tyre rubber-modified bitumen was expected to have better resistance to permanent deformation or rutting and low-temperature cracking. The study also found that the viscoelastic properties of rubber-modified bitumen with 9% weight are being to SBS-modified bitumen having 3% weight SBS at-10ºC, and 7% weight at 75ºC.

2.9.1 Correlation between rheological properties of asphalt binder and performance asphalt mixture

An extensive research program conducted by Claxton et *al*. (1996) to investigate the benefits of using fundamental binder rheological measurements to predict asphalt pavement performance included:

i. Pavement deformation (rutting) at high service temperatures

ii. Fatigue at intermediate service temperatures

iii. Brittle fracture at low service temperatures

At high service temperatures, rutting resistance tests were measured as a function of some binder parameters (viscosity, ductility recovery, non-recoverable creep compliance, complex shear modulus G* and parameter specified by SHRP G*/sinδ). It was concluded from the parameters considered, for this range of binders, only the SHRP G*/sinδ gives the most reliable prediction of rut resistance. The SHRP recommended frequency (1.6Hz) was found to correspond closely to the frequency of the wheel tracking test used for rutting resistance experiments. This parameter includes both a measure of the stiffness of the binder (its ability to resist deformation when a load is applied) and its ability to recover any deformation with the removal of the load. The frequency selected for the binder measurements has was to have a significant impact on the quality of the correlation obtained and should be maintained close to the frequency of loading applied to the mix (Claxton *et al*., 1996).

At intermediate pavement service temperatures a reasonable correlation was found between one aspect of mix fatigue performance (**ε**), and the binder loss modulus (G* sin δ), again measured under the same temperature and loading as the mix testing. However, above certain binder stiffness, due to machine compliance being significant at high mix stiffness the variation in measured fatigue life was minimal. Binder rheology alone is not adequate to accurately predict and explain mix fatigue life. At low pavement service temperatures a binder limiting stiffness temperature (LST) in this case based on G*=300Mpa at 1000s provides a good indicator of the fatigue temperature of the mix (Claxton *et al*., 1996).

2.9.2 Rheological Properties and Fatigue Resistance of CR Bituminous

Bahia and Davies (1994) used the rheological properties as indicators for the pavement performance. At high temperature the rheological properties were related to the rutting performance of pavements. The rheology at intermediate temperatures had an impact on the fatigue cracking of pavements. The low temperature properties of the binder are related to the low-temperature thermal cracking of the pavement. Temperature additionally is a vital factor that is correlated with the rate of loading. At elevated temperatures, or slow rates of loading, bitumen becomes a viscous material. However, at decreased temperatures or higher rates of loading, bitumen then becomes a highly elastic material. In fact at intermediate temperatures, bitumen has two different characteristics; i.e. an elastic solid and a viscous fluid (Van der Poel, 1954).

A study by Aflaki and Memarzadeh (2011) investigated the effects of rheological properties of crumb rubber on fatigue cracking at low and intermediate temperature using different shear methods. The results showed that the high shear blending has more effect on improvement at low temperatures than the low shear blend.

Bahia and Anderson (1993) conducted a time sweep test using dynamic shear rheometer. The test is a simple method of applying repeated cycles of stress or strain loading at selected temperatures and loading frequency. The initial data under repeated loading in shear showed that the time sweeps are effective in measuring binder damage behaviour. One of the advantages of the time sweep test is that it can be used to calculate fatigue life of asphalt binder based on dissipated energy approaches. Bahia and Anderson (1995a,b) presented a description of the purpose and scope of the dynamic shear rheometer test. The dynamic shear rheometer (DSR) was used to characterise the viscoelastic behaviour of bituminous material at intermediate and high service temperatures. Stress-strain behaviour defines the response of materials to load. Asphalt binder exhibit aspects of both elastic and viscous behaviours; hence they are referred to as viscoelastic materials

2.9.3 Stress and Strain within Flexible Asphalt Concrete

Asphalt road pavements are defined as asphalt layers built bound over a granular base. Due to this, the total pavement structure deflects due to traffic loads, thus these types of pavements are known as flexible pavements. A flexible pavement structure is composed of various layers of materials. Basically, the pavement structure is divided into three layers namely: bituminous surfacing (surface course), road base (base course) and sub-base (Hamed, 2010) (see Figure 2.10).

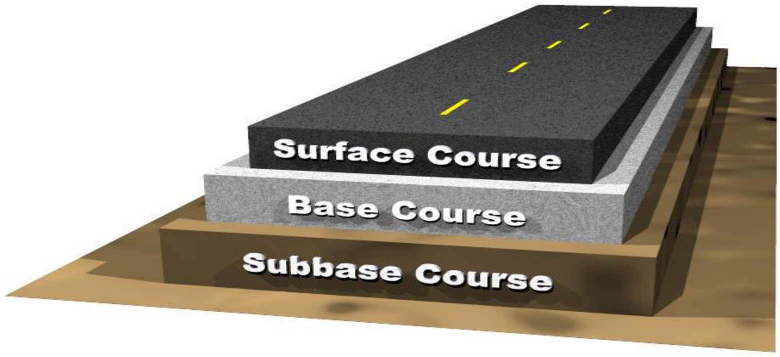
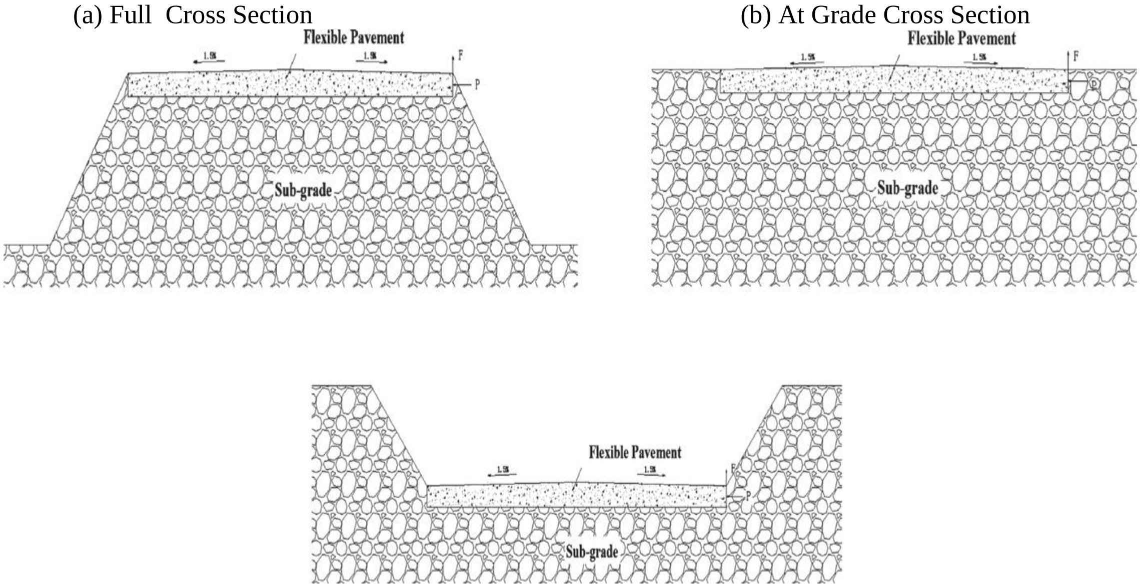


Figure 2.10: Typical flexible pavement structure

Flexible pavements could have one of the three typical cross section geometries as shown in Figure 2.11. At the pavement edge, between the pavement edge and adjacent soil two forces exist which are; vertical friction, F, and lateral passive pressure, P. The friction force (F) relies on relative movement, coefficient of friction and the lateral passive pressure. Lateral passive pressure (P) varies depending on soil type and weight of the soil subjected to the pavement. As illustrated in Figure 2.11- a, the soil wedge is small and the two force (F and P) can be ignored. On the other hand, as shown in Figures 2.11- b and c, the friction and passive forces may be significant and the pavement edge can move laterally and vertically (Wang *et al*., 2012).

(a) Full Cross Section

(b) At Grade Cross Section



(c) Cut Cross Section

Figure 2.11: Flexible pavement typical cross section geometries (Wang *et al*., 2012)

Asphalt mixture should have high stiffness to be able to resist permanent deformation. On the other hand, the mixtures should have enough tensile stress at the bottom of the asphalt layer to resist fatigue cracking after many load applications. Figure 2.12 presents the orientation of principal stresses with respect to position of rolling wheel load.

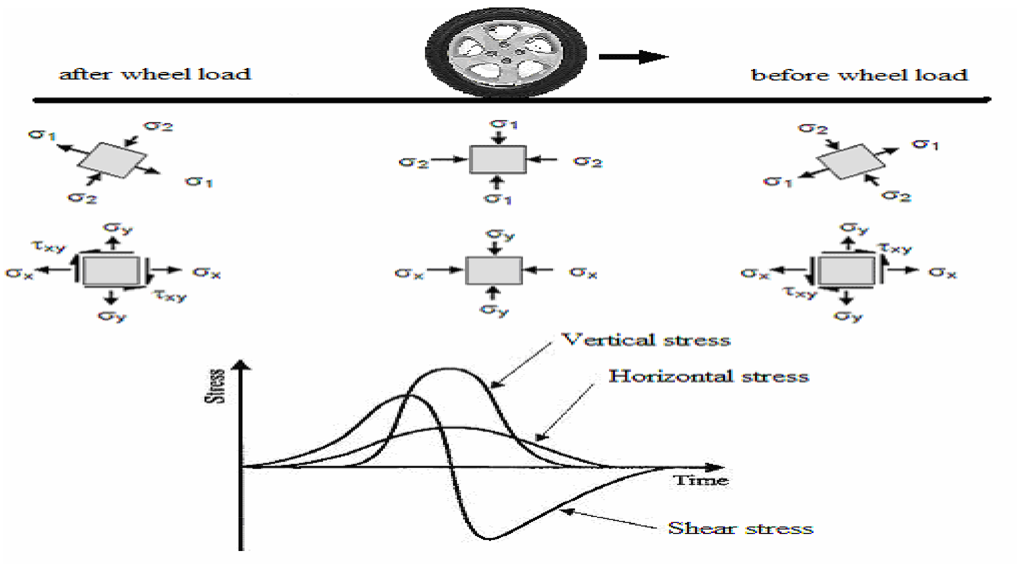


Figure 2.12: Stress beneath a rolling wheel load after (Shaw, 1980)

2.10 Asphalt Mixtures Characterisation

Different tests and approaches have been used to evaluate asphalt concrete mixtures properties. Several material properties can be obtained from fundamental, mechanistic tests that can be used as input parameters for asphalt concrete performance models. The main aspects, which can be characterised using indirect tensile test, are resilient elastic properties, fatigue cracking and the properties related to permanent deformation. The elastic stiffness of the asphalt mixtures can be measured using the indirect tensile test (IDT) (Hamed, 2010; Hadley *et al*. 1970).

2.10.1 Indirect Tensile Strength Test

The indirect tensile strength of a sample is calculated from the maximum load to failure. According to Witczak *et al*. (2002), the indirect tensile test (IDT) has been extensively used in the structural design of flexible pavements since the 1960s. Strategic Highway Research Program (SHRP) (1994) recommended indirect tensile test for asphalt concrete mixture characterisation. The popularity of this test is mainly due to the fact that the test can be done using marshal sample or cores from filed. This test is easy, quick, and characterised as less variable. Guddati *et al*. (2002) have also indicated that there is good potential in predicting fatigue cracking using indirect tensile strength results.

Othman *et al.* (2007), quoted by Hamed (2010), was conducted a research to evaluate the performance of Polyethylene (PE) modified asphaltic mixtures based on physical and mechanical properties. Physical properties were evaluated in terms of penetration and softening point. The mechanical properties were evaluated based on the indirect tensile strength. The result presented that PE enhanced both physical and mechanical properties of modified binder and mixtures.

2.10 .2 Resilient Modulus Test

The dynamic stiffness or 'resilient modulus' is a measure of the load-spreading ability of the bituminous layers; it controls the levels of the traffic-induced tensile strains at the underside of the lowest bituminous bound layer which are responsible for fatigue cracking, as well as the stresses and strains induced in the subgrade that can lead to plastic deformations (O'Flaherty, 1988).

The dynamic stiffness is computed by indirect tensile modulus test, which is a quick and non-destructive method. In general, the higher the stiffness, the better is its resistance to permanent deformation and rutting (Samsuri, 1997). Research on rubberised bitumen by Eaton *et al*. (1991) showed that the resilient modulus increased or the mix behaved in a stiffer manner (the mix become stronger) with a decrease in temperature; also, as the load time increased, the resilient modulus decreased or yielded more under a longer loading time.

Indirect tensile resilient modulus test is widely used as a routine test to evaluate and to characterise pavement materials. Little *et al.* (1990) defined the resilient modulus as the ratio of the applied stress to the recoverable strain when a dynamic load is applied. In this test, a cyclic load of constant magnitude in the form of haversine wave is applied along the diametric axis of a cylindrical specimen for 0.1 seconds and has a rest period of 0.9 seconds, thus maintaining one cycle per second. Wahhab *et al.* (1991) conducted a resilient modulus test on unmodified and modified asphalt concrete mixtures using Marshall specimen. A dynamic load of 68 kg was applied and stopped after a 100 load repetition. The load application and the horizontal elastic deformation were used to compute the resilient modulus value. Two temperatures were used, 25 ^{o}C and 40 ^{o}C. The modified asphalt mixtures with 10 % percent crumb rubber showed an improved modulus compared to the unmodified asphalt concrete mixtures.

2.10.3 Indirect Tensile Fatigue Test

There are different test methods used throughout the world to measure fatigue resistance for asphalt concrete mixtures. Read *et al*. (1996) investigated the fatigue life of asphalt concrete mixtures using the indirect tension fatigue test. During the indirect tension fatigue, the horizontal deformation was recorded as a function of load cycle. The test specimen was subjected to different levels of stress, in order for a regression analysis on a range of values. This allows the development of the fatigue relationship between the number of cycles at failure (N_F) and initial tensile strain (ε_t) on a log-log relationship. Fatigue life (N_f) of a specimen is number of cycles to failure for asphalt concrete mixtures. The fatigue life is defined as the number of load cycling application (cycles) resulting in either disintegration or a permanent vertical deformation. Fatigue test procedure is used to rank the bituminous mixture resistance to fatigue as well as a guide to evaluate the relative performance of asphalt aggregate mixture, to obtain data and input for estimating the structural behaviour in the road. During the fatigue test, modulus value decreased as indicated in Figure 2.13. Three phases were distinguished (Castro and Jose´, 2008):

i. Phase I: Initially there is a rapid diminution of the modulus value.

ii. Phase II: modulus variation is approximately linear.

iii. Phase III: rapid decrease of the modulus value.

Damage is defined as any loss of strength that takes place in a specimen during a test.

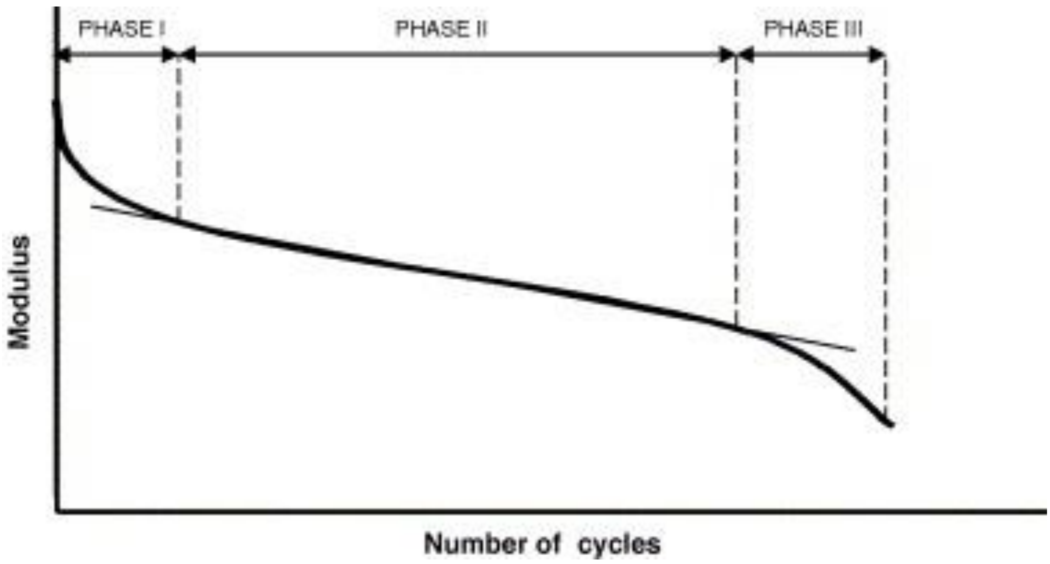


Figure 2.13: The Three phases of Fatigue test (Castro and Jose´, 2008)

Stephen and Raad (1997), quoted by Hamed (2010), was investigated the fatigue behaviour of the different mixes using controlled-strain third-point flexural beam tests. Controlled-strain flexural fatigue testing indicated that the incorporation of CRMs in mixes can enhance their fatigue resistance. The magnitude of improvement appears to depend on the degree and type of rubber modification. Multilayer elastic analysis combined with fatigue test results for typical Alaskan conditions also indicated the enhanced fatigue behaviour of CRM mixes. However, condition surveys at both conventional and CRM sections revealed no longitudinal or alligator type of cracking, suggesting similar field fatigue performance for both materials.

2.11 Fatigue Cracking of Rubberised Asphalt

Fatigue is one of most important distresses in asphalt pavement structure due to repeated load of heavy traffic services which occur at intermediate and low temperatures as shown in Figure 2.14. The use of crumb rubber modified with bitumen binder seems to enhance the fatigue resistance, as illustrated in a number of studies (Raad and Saboundjian, 1998; Soleymani *et al.*, 2004; Read, 1996; McGennis, 1995; Biliter *et al.*, 1997a; Hamed, 2010). The improved performance of bitumen rubber pavements compared with conventional bitumen pavements has partly resulted from improved rheological properties of the rubberised bitumen binder.

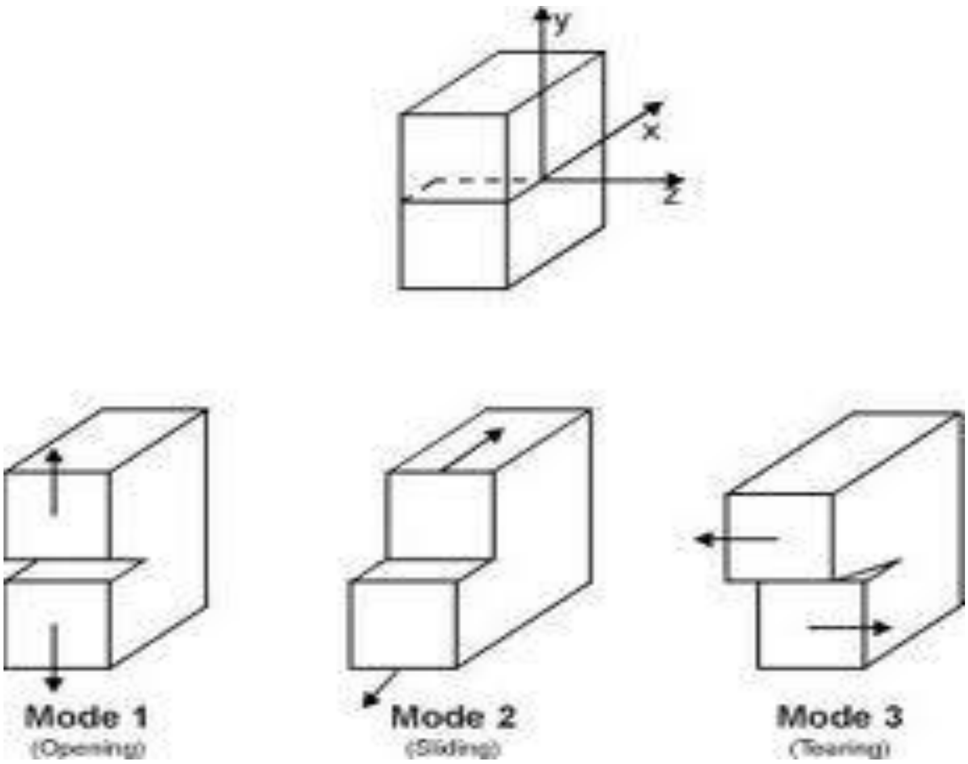
Figure 2.14: Fatigue Cracking (Source: Asphalt institute , 2009)

Cracking is normally considered to be low temperature phenomena while permanent deformation is considered the predominant mode of failure at elevated temperatures. Cracking is mainly categorised into thermal cracking and load-associated fatigue cracking. Large temperature changes that occur in pavement usually result thermal cracking. This type of failure occurs when the thermally induced tensile stress, together with stresses caused by traffic exceeds the tensile strength of the materials. It is often characterised by transverse cracking along the highway at certain intervals. Load-associated fatigue cracking is the phenomenon of fracture as a result of repeated or fluctuated stresses brought about by traffic loading. Traffic loads can cause a pavement structure to flex and the maximum tensile strain will occur at the base of the bituminous layer. If this structure is inadequate for the imposed loading conditions, the tensile strength of the materials will be exceeded and cracks are likely to initiate, which will be manifested as cracks on the surface of the pavement (Mahrez, 2008).

This resistance of bituminous mixtures to cracking is essentially dependent upon its tensile strength and extensibility characteristics. These can be achieved by simply increasing the bitumen content of the mix. However such an attempt may have an adverse effect on the mix stability. The use of softer bitumen can also improve the mix flexibility but this can only be achieved at the expense of the tensile strength and stability of the mix (Mahrez, 2008).

In the fracture mechanics approach, fatigue cracking process of pavement systems is considered to develop in two distinct phases involving different mechanisms. These phases consist of crack initiation and crack propagation before the material experience failure or rupture. Crack initiation can be described as a combination of micro-cracks within the mix forming a macro crack as a result of repeated tensile strains. This occurrence usually creates gradual weakening of the structural component (Majidzadeh, 1983). These micro-cracks become more visible as the stress concentrations at the tip of the crack increase and cause further crack propagation. Crack propagation is the growth of the macro-crack through the material under additional application of tensile strains. The actual mechanism of crack initiation and propagation involves fracture of the overlay when the tensile stresses exceed the tensile strength under the particular conditions (Mahrez, 2008).

For an accurate determination of the crack propagation, the magnitude of the stress intensity factors over the overlay thickness should be available for each fracture mode. In general, the mechanisms of cracking propagation can follow one or more of the three fracture modes which are directly related to the type of displacement induced (Joseph *et. al.*, 1987). This is shown in Figure 2.15:



***Mode 1(Opening),   Mode 2 (Sliding),   Mode 3(Tearing)**

Figure 2.15: Modes of crack displacement (Joseph *et al*., 1987)

i. Mode I loading (opening mode) results from load applied normally to the crack plane (normal tension). This mode is associated with traffic loading and in the case of thermally induced displacement.

ii. Mode II loading (sliding mode) results from in plane / normal shear loading, which leads to crack faces sliding against each other normal to the leading edge of the crack. This mode is usually associated with traffic loading or differential volume changes.

iii. Mode III loading (tearing mode) results from out of plane shear (parallel shear) loading, which causes sliding of the crack faces parallel to the crack loading edge. This mode may occur under lateral displacement due to instability, if the crack plane is not normal to the direction of traffic.

2.12 Stone Mastic Asphalt

2.12.1 History of Stone Mastic Asphalt

Stone Mastic Asphalt (SMA) is a hot asphalt paving mixture, developed in Germany during the mid 1960's (AASHTO report, 1990; Brown and Hemant, 1993) to provide maximum resistance to rutting caused by the studded tyres on European roads. Once the use of studded tyres was no longer allowed, it was found that SMA provided durable pavements which exhibited such high resistance to rutting by heavy truck traffic and proved to be extremely effective in combating wear. "Strabag", a large German construction company, led to the development of SMA. In Europe, it is primarily known as "Splittmastixasphalt," revealing its German origin (Split-crushed stone chips and mastic-the thick asphalt cement and filler). Over the years this term was variously translated and has been referred as Split Mastic Asphalt, Grit Mastic Asphalt, or Stone Filled Asphalt (AASHTO report, 1990; Brown and Hemant, 1993).

In recognition of its excellent performance a national standard was set in Germany in 1984. Since then, SMA has spread throughout Europe, North America and Asia Pacific. Several individual Countries in Europe now have a national standard for Stone Mastic Asphalt, the European standards body, is in the process of developing a European product standard. Today, SMA is widely employed in many countries in the world as an overlay or surface course to resist load induced and its popularity is increasing amongst road authorities and the asphalt industry (Mahrez, 2008).

2.12.2 Composition of Stone Mastic Asphalt

SMA is characterised by a gap-graded aggregate gradation and high stone content. It consists of up to 80% by weight of coarse aggregate and up to 13% by weight of filler (Mahrez,2008). This high stone content ensures stone-on-stone contact after compaction. The gap-graded aggregate mixture provides a stable stone-on-stone skeleton that is held together by a rich mixture of asphalt mastic. Aggregate interlock and particle friction are maximised and gives the structure its stability and strength.

The necessity to fill the voids between the aggregate requires a high amount of mortar. The SMA mixtures are usually designed to have air voids content between 2-4%; this prevents overfilling of the voids and ensures particle friction and stone-to-stone contact. The remaining voids of the structural matrix are filled with high viscosity bituminous mastic, which is a mixture of bitumen, filler, sand and stabilising additives (Susanne, 2000).

SMA mixes have a bitumen content of minimum 6% of the total mix. The bitumen is stabilised during the mixing process, through the addition of stabilising additives. Stabilising additives can be organic or mineral fibres, or less often, polymers. They stabilize the asphalt mortar and tend to thicken or bulk the bitumen to prevent binder run-off from the aggregate. Thus, they ensure the homogeneity of the mixture (Bernd, 1996).

2.12.3 Performance Characteristics of Stone Mastic Asphalt

The development of modern pavement technology is needed to accelerate significant improvement of pavement of highways, airport runways and urban roads. The performance characteristics of SMA showed that SMA meets the following demands upon an asphalt pavement (Bernd, 1996):

i. Good stability at high temperatures: SMA mixtures have a self-supporting stone skeleton of crushed high quality coarse aggregate, which provides an increase in internal friction and shear resistance and hence it is extremely high.

ii. Good skid resistance: SMA pavement achieves a better level of skid resistance because of the macro-texture of the road surface and the use of coarse aggregates with a high Polished Stone Value. However, the skid resistance may be lessened during approximately the first month of service until the initial thick binder film on the surface wears off under traffic, exposing the rough edges of the coarse aggregate particles.

iii. Good flexibility at low temperature: SMA mix has a binder rich, mastic, mortar which has superior properties over dense graded asphalt in resisting thermal cracking.

iv. High wearing resistance: SMA mix has low air voids, which make the mix practically impermeable, and provide satisfactory ageing resistance, moisture susceptibility and durability.

v. High adhesive capacity between the stone granules and the bitumen: with the increase of the amount of filler, fibres are added as stabiliser. The use of fibre assists the bitumen to maintain a high viscosity, thickens the bituminous film and improves the bitumen/aggregate adhesion.

vi. A mix with no tendency to separate: An efficient stabilisation of mastic in order to prevent its segregation from the coarse particles.

vii. Reduced weather spray: due to its greater texture depth, there is less water spray, and at night there is less glare reflected from the road surface and better visibility of road markings.

viii. Lower traffic noise: SMA reduces noise emission considerably due to the macrotexture properties of the road surfaces which absorb traffic noise. Due to its noise absorptive property, this surface is highly suitable for access roads in residential areas and on estates.

2.12.4 Effect of Compaction on the SMA Mix

Brown and Hemant (1993) reported that all the mix designs for SMA construction have been performed using the 50 blow Marshall hammer. Even though these mixtures are used on heavy duty roads, 75 blows compaction should not be used since it tends to break down the aggregate further and will not result in a significant increase in density over that provided with 50 blows. SMA mixes have been more easily compacted on the roadways to the desired density than the effort required for conventional HMA mixes (Brown and Hemant, 1993, Brown *et al*., 1997).

2.12.5 Effect of Mastic Asphalt content on the SMA Properties

SMA relies on a stone-on stone skeleton. The stone skeleton is filled and held together with mastic asphalt. The mastic is rich and has fewer voids so it provides a durable surface that is resistant to cracking. The stone skeleton must accommodate all the mastic without disrupting the point-to point contact of the coarse aggregate particles (AASHTO, 1990). Too much mastic will spread the coarse aggregate apart, leading to a pavement that is susceptible to shear. Too little mastic will create an unacceptably high air voids content, expose the bitumen to accelerated aging and moisture damage, and lead to a poorly-bound, distress-prone pavement (AASHTO, 1990).

2.12.6 Cost Effectiveness of SMA mixture

The European asphalt study tour team during their visit to Sweden, stated in their report that the price difference between SMA and the standard asphalt concrete mix was difficult to tie down. Comments indicated, however, that SMA is somewhere between 10 and 12 percent more expensive (Davidson and Kennepohl, 1992).

CHAPTER 3

METHODLOGY

3.1 Introduction

The experimental program in this study aims to investigate the effect of CRM on the rheological characterises of rubberised bitumen and mechanical properties of SMA mixtures. This chapter illustrates the basic experimental and the detailed test approaches used in this research study to investigate the performance properties of crumb rubber modified bituminous mixes. Testing was advocated on the bitumen binder with and without CRM as well as on the bituminous mixtures.

The engineering properties of the mixtures were determined from the relevant laboratory tests in compliance with the American Society of Testing Materials (ASTM), the American Association of State Highway and Transportation Officials (AASHTO) and the British Standards (BS).

The testing methodologies involved in this study can be categorised into two main tasks (see Figure 3.1) :

(i) The first task will focus on the rheological properties of crumb rubber modified bitumen binder.

(ii) While the second task will focus on the performance of crumb rubber modified SMA mixtures.

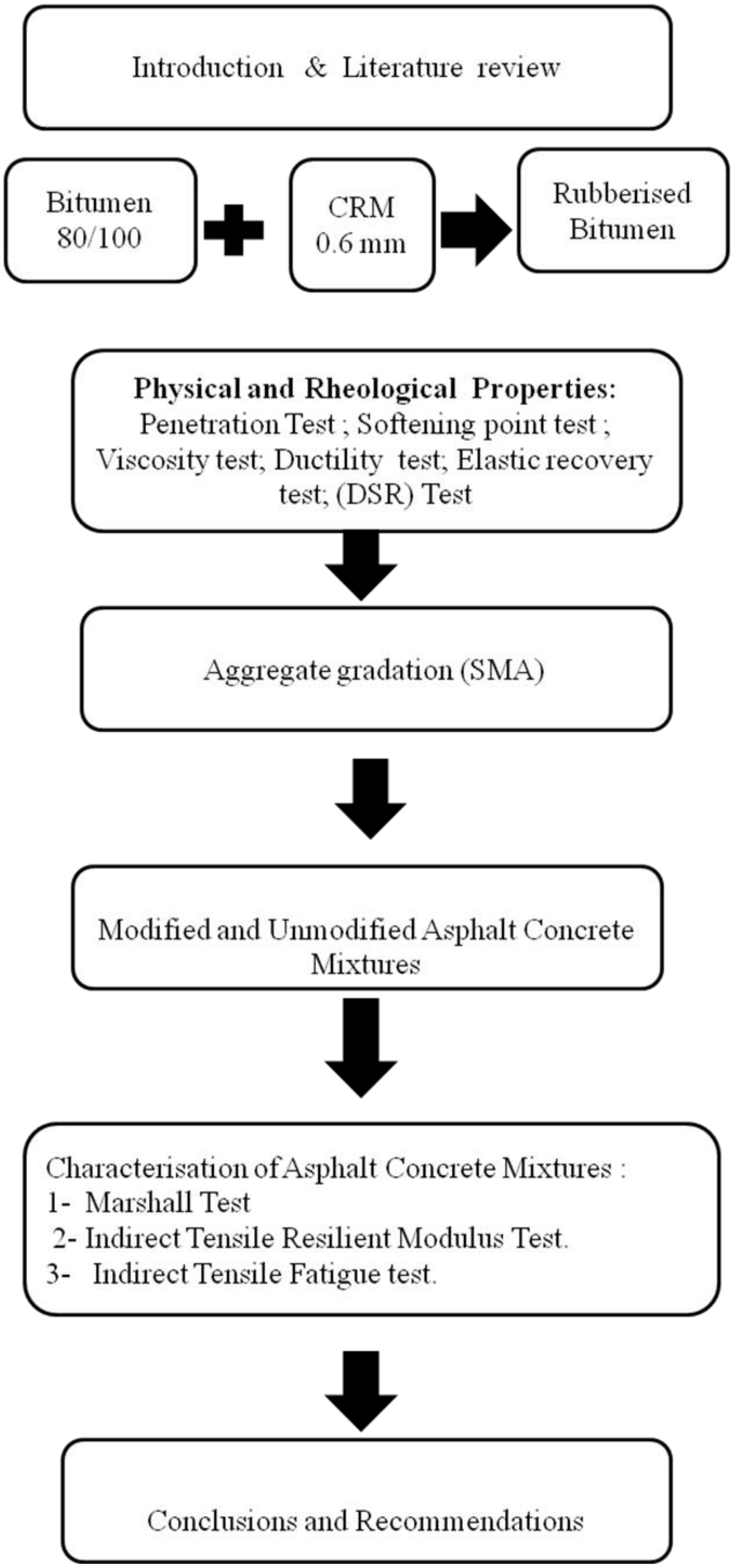


Figure 3.1: Flow chart of laboratory testing program

3.2 Materials

3.2. 1 Bitumen

For the purposes of this research investigations, bituminous binder of 80/100 penetration grade was used .Table 3.1 shows the characteristics of the bitumen used in this research.

Table 3.1: Properties of Base Binder Grade 80/100 Penetration

Test properties	Test result
Viscosity @135 ˚C (Pas)	0.65
G*/ sin δ @ 64˚C (kpa)	1.35
Ductility @ 25 ˚C	100
Softening point @ 25 ˚C	47
Penetration @ 25 ˚C	88

3.2.2 Aggregate selection

The crushed granite aggregates supplied by the Kajang quarry was used throughout the study. The SMA 20 aggregate gradations were adopted, characterised by 20mm as shown in Table 3.2.

Table 3.2 : SMA 20 aggregate gradation

SMA 20					
	% Passing				
B.S Sieve	**Min.**	**Max.**	**Mid.**	**% Retained**	**Weight (G)**
19	100	100	100	0	0
12.5	85	95	90	10	110
9.5	65	75	70	20	220
4.75	20	28	24	46	506
2.36	16	24	20	4	44
0.6	12	16	14	6	66
0.3	12	15	13.5	0.5	5.5
0.075	8	10	9	4.5	49.5
pan	0	0	0	9	99
				100	**1100**

3.2.3 Crumb Rubber Modifier (CRM)

For the purpose of maintaining consistency of the CRM throughout the entire study, one batch of crumb rubber obtained from one source was solely used as displayed in Figure 3.1. In this study, in order to decrease segregation, fine crumb rubber size 30 # (0.6 mm) was selected (Liu *et al*., 2009), with specific gravity equivalent to 1.161.

3.3 Rubberised Bitumen Binders Fabrication

By mixing 80/100 penetration grade bitumen with different percentages of fine crumb rubbers, rubberised bitumen was produced. The rubber particle was passed through a 30 mesh sieve (0.6 mm). In preparing the rubberised bitumen, a propeller mixer and the wet process was utilised. Binder mixing was conducted at the velocity speed of 200 rpm at blending temperatures 160 °C and blending time 30 minutes. The steps involved in the preparation of CRM bitumen samples are follows (Mashaan, 2012).

3.4 Bitumen Binder Testing

The tests undertaken comprised the penetration test (ASTM D5), softening point test (ring and ball) (ASTM D36) and Brookfield viscosity (ASTM D4402), Ductility test (ASTM D113) and elastic recovery test (ASTM D 6084- 97), respectively.

3.4.1 Softening Point Test (Ring and Ball) (ASTM D 36)

According to the specification test, softening point is the temperature at which the bitumen reaches a particular degree of softening. This was tested using a ring and ball apparatus as shown in Figure 3.2. A brass ring containing test sample of bitumen was suspended in liquid such as water or glycerine at a specific temperature. A steel ball was put into the bitumen sample and the liquid medium was heated at 5˚C per minute. Temperature was recorded when the softened bitumen touched the metal plate which was at a specified distance. Often, higher softening point indicates lower temperature susceptibility which is typically found in hot climates (ASTM D 36-06; Mahrez, 2008).

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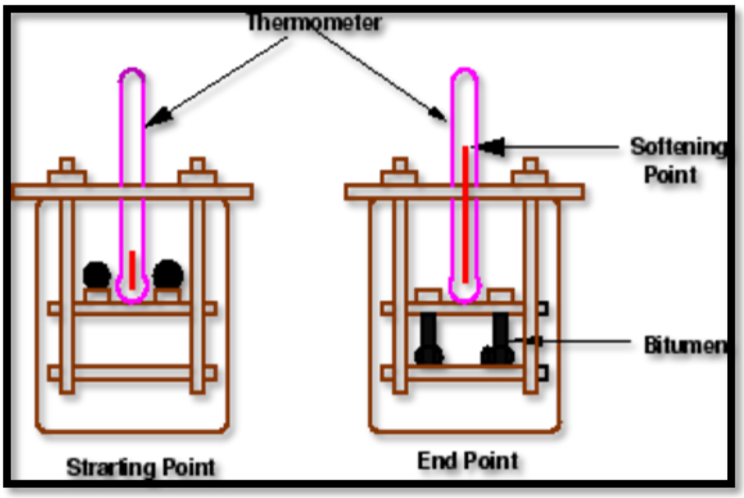

Figure 3.2: Softening Point Test Setup (Branco *et al*., 2005)

3.4.2 Penetration Test (ASTM D5)

The purpose of this test is to examine the consistency of bituminous materials by measuring the distance (tenths of a millimetre) when a standard needle is vertically penetrated into the bitumen sample under known conditions (5 sec, 25 ˚C and 100g) as shown in Figure 3.3. The penetration apparatus consists of a needle, needle holder, sample container, water bath, transfer dish, timing device and thermometer. The penetration test is a common test for the purpose of defining the various grades of bitumen. Form past research, it is evident that reduction in penetration value indicates resistance of binder to permanent deformation (ASTM D5- 79; Mahrez, 2008).

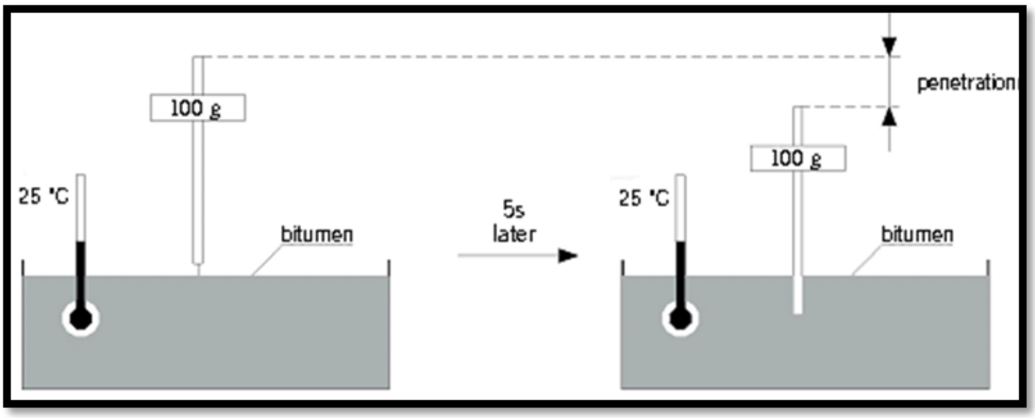

Figure 3.3: Penetration test setup (Branco *et al*., 2005)

3.4.3 Brookfield Viscosity (ASTM D 4402- 87)

The Brookfield Thermosel apparatus is used to measure the apparent viscosity of bitumen from 38 to 260 ˚C (100 to 500 ˚F). Brookfield Thermosel high temperature viscosity measure system comprises spindles, thermoses system; thermo container and sample chamber, controller and graph plotting equipment (see Figure 3.4). The sample was filled into the sample holder to approximately 10 ± 0.5 g. In this research study, the proportional temperature controller was set to desired test temperature of 90 ˚C.   Also, Spindle No. 27 was used in this research study which was inserted into the liquid in the chamber, coupled with the viscometer. Then, the sample was left aside allowed to wait for 15 minutes until it reached the equilibrium temperature of 90 ˚C. The spindle was rotated at a lower speed (10 – 15 rpm) and then increased to 20 rpm when the torque started dropping. The reading for viscosity was only taken when the torque reading was stable (ASTM D 4402- 87).

Figure 3.4: Brookfield viscometer apparatus

3.4.4 Ductility Test (ASTM D113- 99)

The ductility of a bituminous material is measured by the distance of its elongation before breaking when two ends of a briquette specimen of the material are pulled apart at specified speed and temperature. The specified conditions of the tests are temperature of about 25 ˚C ± 0.5 ˚C and the speed of 5 cm/min. This test method provides one measure of tensile properties of bituminous materials and can also be employed to measure ductility for specification requirements. Ductility is considered as an important elasticity property of asphalt mixture, thus would indicate the adhesiveness of asphalt mixture, like the capability of the bitumen binder to resist deformation under high temperature load services. Factors which greatly affect the ductility value include pouring temperature, test temperature, rate of pulling (ASTM D113- 99).

3.4.5 Elastic Recovery Test (ASTM D 6084- 97)

This test method presents the elastic recovery of a bituminous material measured by the recoverable strain determined after severing an elongated briquette specimen of the material. These specimens are pulled to a specified distance at a specified speed and temperature. The elongated distance considered is 10 cm; the test was conducted at temperature of 25 ˚C ± 0.5 ˚C and with a speed of 5 cm/min ± 5 %. This test is the same as the ductility test whereby the same test apparatus is used with the exception for the mould design which is similar to the description in the elastic recovery standard test (ASTM D6084).

3.4.6 DSR test (ASTM D-4 proposal P246)

This proposed standard contains the procedure used to measure the complex shear modulus (G*) and phase angle (δ) of bitumen binders using a dynamic shear rheometer (DSR) and parallel plate test geometry. Test specimens 1 mm thick by 25 mm in diameter are formed between parallel metal plates. The test specimen was maintained at the test temperature to within ± 0.1 ˚C by positive heating and cooling of the upper and lower plates. Oscillatory loading frequencies using this proposed standard ranged from 1 to 100 rad/s using a sinusoidal waveform. Specification testing was performed at a test frequency of 10 rad/s. The complex modulus (G*) and phase angle (δ) were calculated automatically as part of the operation of the rheometer using proprietary computer software supplied by the equipment manufacturer. Two types of testing plate geometries were used with the dynamic shear rheometer. The first specimen geometry was a 25-mm diameter spindle with 1-mm testing gap for intermediate to high temperature. The second specimen geometry was 8-mm diameter spindle generally used at intermediate and low temperatures.

3.5 Aging

3.5.1 Rolling Thin Film Oven Test (RTFOT) (ASTM D 2872-88)

The RTFOT procedure requires an eclectically heated convection oven to be heated to the ageing temperature of 163 ˚C. The oven is inside a vertical circular carriage which can hold up 8 horizontally positioned, cylindrical bottles which may be rotated mechanically around the carriage centre. An air jet blows into each bottle whenever it passes through its position on this carriage during the circulation.

3.5.2 Pressure Ageing Vessel Test (PAV) (ASTM D 6521)

The pressure ageing apparatus consists of the ageing vessel and a temperature chamber. Air pressure is provided by a cylinder of dry, clean compressed air with a pressure regulator, release valve and a slow release bleed valve. The pressure ageing vessel exposes the asphalt to a simultaneous high pressure of 2.1 MPa and high temperatures above 100 ˚C for a period of 20 hours. The vessel must accommodate at least 10 sample pans by means of a sample rack. Figure 3.5 presents the Pressure Ageing Vessel Oven (PAV).

Figure 3.5: Pressure Ageing Vessel Oven (PAV)

3.6 Rubberised Stone Mastic Asphalt Mixture

3.6.1 Experimental design of rubberised SMA Mixture

The binders used in preparation of rubberised bitumen mixture were the same binders prepared in Section 3.3 which contained the bitumen 80/100 penetration and crumb rubber modifier with 6, 12, 16 and 20% binder weight. The binder content utilised in this study are 5, 5. , 6, 6.5 and 7% by weight of the total mix.

3.6.2 Preparation of the SMA Mixture Samples

Marshall Design method was used for the modified and unmodified asphalt concrete mixtures. An impact hammer was used to compact samples in a 101 mm diameter mould to a height of approximately 64.5 mm. The process that was used in SMA mixture samples perpetration was illustrated below:

(i) The appropriate proportion of aggregate was weighed, placed into the oven and heated up to 160 °C for 3 hours. Bitumen required for the specimen was simultaneously heated up to temperature of 160 °C for one hour.

(ii) Once the aggregate and the bitumen reached the required temperature, the needed quantity of heated bitumen was added into the aggregates. Later, the crumb rubber modified bitumen binder and aggregates were mixed together (mixed by hands) at mixing temperature of 160 ^{o}C until the aggregate was coated totally with bitumen.

(iii) The mixture was transferred into a Marshall mould. To avoid the sample and the mould sticking to each other, a filter paper was laid in the base of the marshal mould.

(iv) The mixture was tampered with a spatula ten times around the perimeter and another ten times over the interior of the mould.

(v) The stainless steel thermometer was put in the centre of the mould and mixture was then ready for compaction at temperature of 160 ± 5 ^{o}C.

(vi) Another filter paper was used on the top of the mixture sample and later, the mould assembly was shifted to the Marshal Compactor device.

(vii) All samples were subjected to 50 compacted blows at each side.

(viii) After finishing compaction, each sample was kept to cool at room temperature overnight before being extruded from the mould.

(ix) Samples were removed from Marshall Mould using hydraulic jack and stored at room temperature to be used later for further testing.

3.6.3 Optimum Binder Content

The optimum binder content according to the Marshall method (ASTM D 1559) was chosen based on examining volumetric properties of the specimens as well as their stability and flow test results. The methodology for selecting the optimum binder content was in compliance with the asphalt institute procedure as listed below:

(i) Obtain the average of the binder contents required for maximum stability, maximum density, and midpoint of selecting average of VMA.

(ii) Obtain from the test plots the value of stability, flow, VIM and VMA corresponding to the average binder content calculated in (1).

(iii)Verify that values determined in (2) satisfy the limiting criteria (Asphalt Institute, 1990).

3.7 Indirect Tensile Modulus Test (Resilient modulus)

3.7.1 Scope

This test covered the procedure for testing laboratory or filed recovered cores of bituminous mixtures to determine resilient modulus (M_R) value using load indirect tensile test, under specified conditions of temperature, load and load frequency. The test was conducted by applying compression loads with a prescribed sinusoidal waveform. The load was applied vertically in the vertical dimension plan of cylindrical specimen of bitumen sample. The resulting horizontal deformation of the specimen was measured with an assumed Poisson's ratio to calculate the resilient modulus values.

3.7.2 Testing Parameters

a. Temperature = 5, 25 and 40 ^oC

b. Poisson ratio =0.34

c. Force = 20 x specimen depth

d. Rise time = 70 ms; Pulse period = 1 s.

3.7.3 Test Procedure

The indirect tensile test for resilient modulus of bituminous was carried out according to ASTM D1234 (1987), using the UMATTA (Universal Materials Testing Apparatus).

Initially, the diameter and thickness of the samples were measured using a venire clipper and recorded to the nearest 0.1mm. The sample was placed in the test jig and put inside a temperature control chamber at a specified temperature until test temperature was obtained at the core of the sample (ASTM D1234, 1987).

The sample was loosely fitted into the loading apparatus and the loading strips were positioned to be parallel and centred on the vertical diametric plane. The displacement transducer yoke (LVDT) was placed, the sample laterally central and two loose clamps were tightened firmly to attach the yoke to the specimen. The level display was used to mechanically adjust the LVDT transducer to operate within the electrical range. The indirect tensile test then was started, a pulsed compressive force was applied to the sample and the resulting total recoverable strain was measured by LVDT transducer. Each sample was tested four times at equal distance on diametric plane, and the average of four readings was considered (ASTM D1234 ,1987).

A cylindrical specimen was loaded diametrically across the circular cross section. The dimension of the sample was 101.5 mm diameter and 64.5 mm height with load to failure along the diametrical plane of the sample. Diametric load was applied continuously at the constant rate of deformation until the peak load was reached, at which point the specimen fractured. Marshall sample was subjected to compressive loads between two loading strips, which created tensile stress, along the vertical diametric plane causing a splitting failure as shown in Figure 3.6 (Grätz ,1996 quoted by Hamed, 2010).

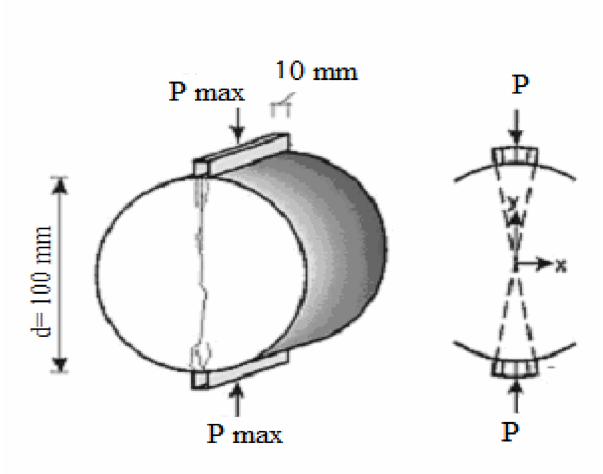

Figure 3.6: Shows indirect tensile test specimen with load balancing (Grätz ,1996 quoted by Hamed, 2010).

3.8 Resistance to fatigue of Bituminous Mixtures Using Repeated Load Indirect Tensile Test (BS EN 12697-24: 2004)

3.8.1 Scope

This test method is according to the standards procedure BS EN 12697-24: 2004. The test was performed on compacted bituminous materials under a sinusoidal loading. Relative applications of compressive load pulse in the vertical diameter of a cylindrical specimen which resulted in permanent deformation of the specimen and induced tensile stresses that are sufficient to eventually split the specimen into two pieces.

3.8.2 Test Procedure

UMATTA was used to determine the repeated load indirect tensile test as a method of assessing the fatigue resistance of bituminous materials. The placement of the sample and test setup is similar to the resilient modulus test where the same loading jig is used. The specimen was exposed to repeated compressive loads with a load signal through the vertical diametrical plane. This loading developed a relatively uniform tensile stress perpendicular to the direction of the applied load, which induced permanent deformation leading to failure of the sample by splitting along the central part of the vertical diameter. The resulting deformation was measured and an assumed Poisson's ratio was used to calculate the tensile strain at the centre of the sample. During the test, the load and horizontal deformation were monitored continually and recorded at the pre-selected intervals using computer data system. The test was stopped when an obvious cracking appeared on the vertical axis. A stiffness reduction of 50% was used to present the sample failure due to fatigue deformation (Mahrez, 2008).

In this research study, three cyclic loading forces were used (2000, 2500 and 3000 N), respectively. Loading cycle width was 100 ms, load cycle repeated time was 500 ms; temperature was 25 °C with axial displacement of about 5- 6 mm (Mahrez, 2008).

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Introduction

This chapter presents the results and analysis of rubberised asphalt binder physical and rheological properties and reinforced SMA mixtures performance properties as well.

4.2 Analysis and discussion of physical binder tests

4.2.1 Penetration Test

The penetration results of various crumb rubber contents are illustrated in Figure 4.1. The results show that the higher crumb rubber content in the mix led to lower penetration values. The results of this research were in tandem with the findings of previous studies (Katman, 2006; Kumar, 2009; Mashaan, 2012). These results are due to the crumb rubber content exhibiting a strong effect on penetration reduction by increasing the stiffness of crumb rubber modified bitumen binder. This makes the binder more resistant to high temperature susceptibility, thus leading to high resistance to permanent deformation like rutting as mentioned by Liu *et al*. (2009) and Hung (2007).

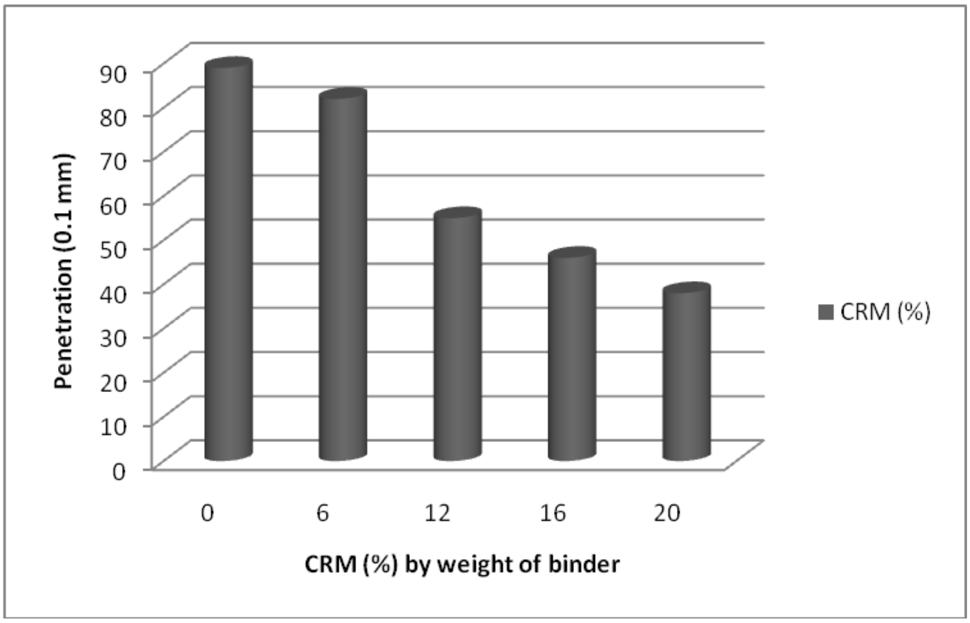

Figure 4.1: Penetration Results vs. different CRM contents

4.2.2 Softening Point Results

Results of the softening point test for various crumb rubber contents are illustrated in Figure 4.2.

Figure 4.2 shows an increase in softening point as the rubber content was increased in the bituminous specimens. The increase in softening point of modified binder samples compared with unmodified binder were approximately about 4 to 70 ^{o}C for 6 and 20% rubber content , respectively. The increase of rubber content in the mix might be related to an increase in the asphaltenes/ resins ratio which enhances the stiffened property and makes the modified binder less susceptible to temperature changes. According to Liu *et al*. (2009) the main factor in the increase in softening point can be attributed to crumb rubber content, regardless of type and size. The increase in softening point led to a stiff binder that has the ability to enhance its recovery after elastic deformation. Moreover, this increase in softening point might have resulted from the increase in binder molecular weight when the crumb rubber interacted with the bitumen binder.

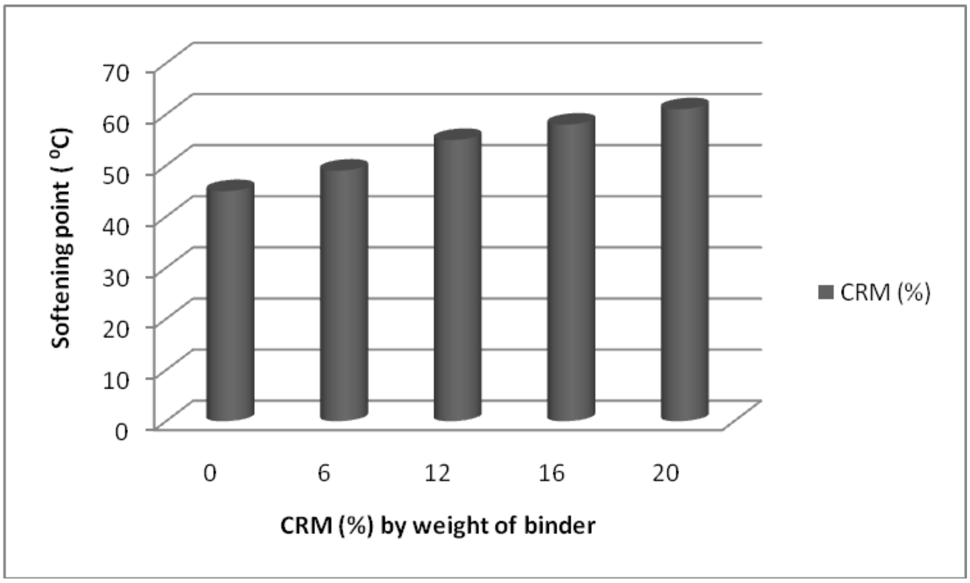

Figure 4.2 : Softening Point Results vs. various CRM contents

4.2.3 Brookfield Viscosity Results

Results from Brookfield viscosity test at 135 °C for the various crumb rubber contents are illustrated in Figure 4.3. An increase in Brookfield viscosity was observed as the rubber content was increased in the bituminous specimens.

The increase in viscosity might be due to the amount of asphaltenes in the bitumen that enhanced the viscous flow of the modified bitumen sample during the interaction process. In general, higher crumb rubber content was found to lead to an increase in the viscosity at 135 °C (Jeong *et al*., 2010). The results of Figure 4.4 show that the viscosity decreased as the temperature of rubberised bitumen binder was increased at different temperatures (90, 100, 120, 135, 150, 160 and 170 °C). The crumb rubber-modified bitumen binder exhibited higher viscosities compared with the unmodified bitumen. The results show that the viscosity of the samples decreased as the temperature was increased. This indicates that the temperature has a direct effect on the viscosity of the modified samples as the results displayed a rapid decrease at temperature 90 °C to 135 °C , however, the results of this research agree the finding of previous studies (Mashaan 2012; Katman, 2006; Mahrez 1999).

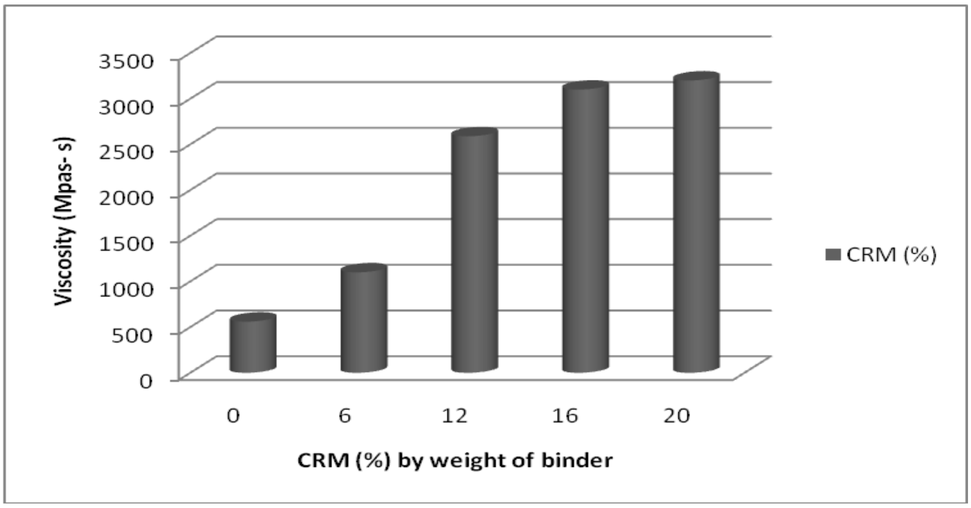

Figure 4.3: Brookfield Viscosity Results at 135˚C vs. various CRM contents

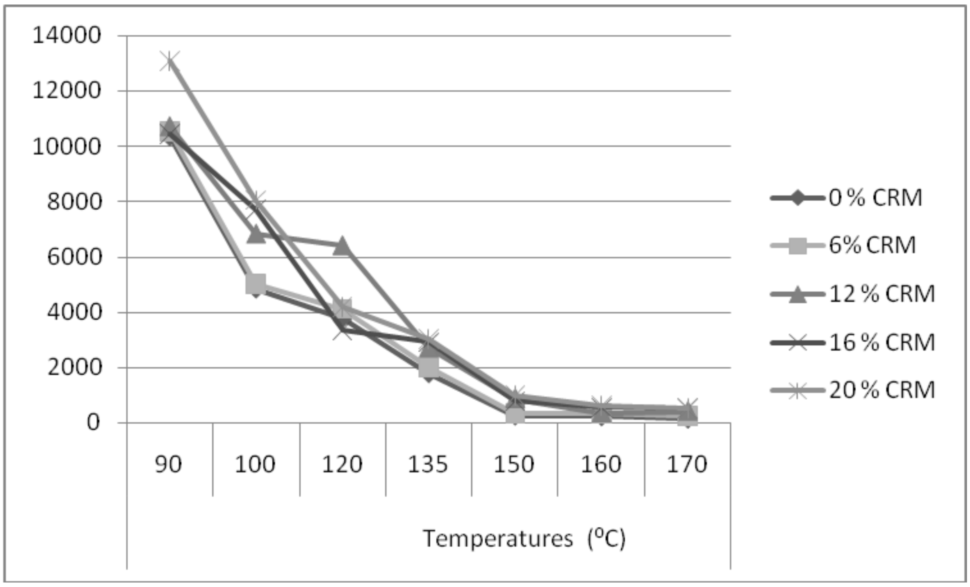

Figure 4.4: Effect of temperature on Brookfield viscosity results

4.2.4 Analysis of Ductility Results

The ductility results for various crumb rubber contents are illustrated in Figure 4.5.

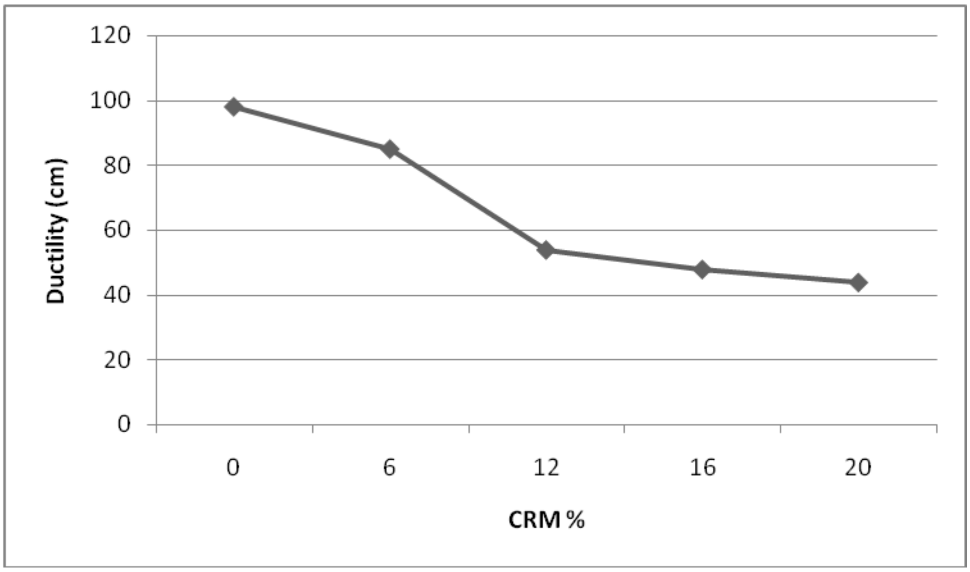

Figure 4.5 : Ductility results vs. various CRM contents

As Figure 4.5 shows, ductility decreased as the rubber content was increased in the bituminous specimens. From the results, the average decrease in ductility of modified binder samples compared with unmodified binder were approximately 20 to 81 % for 6 and 20% rubber content, respectively. Meanwhile, the decrease in ductility value could be attributed to the oily part of the bitumen being absorbed into the rubber powder and the increase in mass of the rubber particles. In effect, the modified binder becomes thicker compared with the unmodified samples (Mashaan, 2012).

4.2.5 Analysis of Elastic Recovery Results

The elastic recovery results at 25 ˚C for various crumb rubber contents are shown in Figure 4.6.

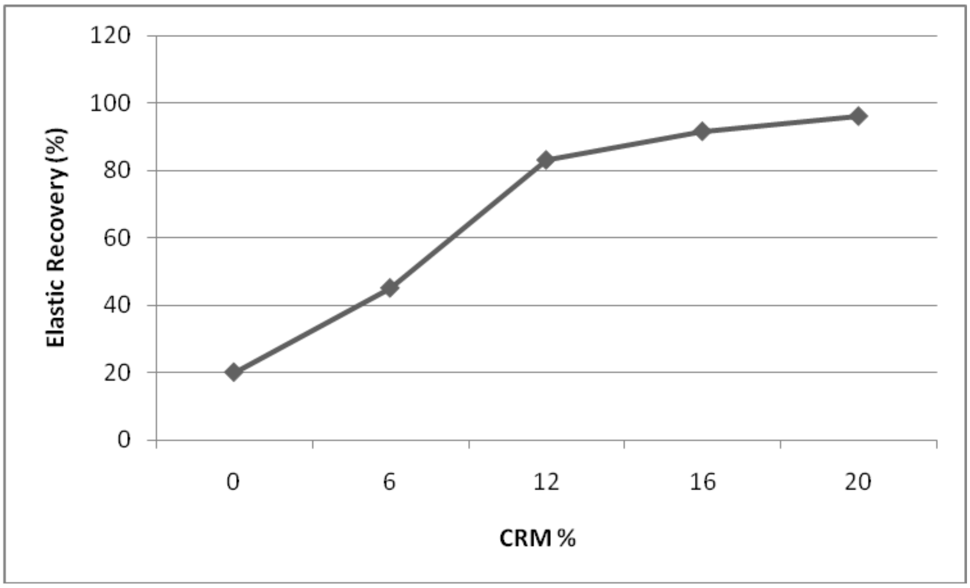

Figure 4.6 : Elastic Recovery results vs. various CRM contents

As displayed in Figure 4.6, an increase in crumb rubber content (CRM) causes the degree of elastic recovery to increase linearly for all modified bitumen samples.

Elastic recovery results were similar to ductility results of rubberised bitumen as it showed significant effect of high content of crumb rubber on elasticity of the modified binder, which presented consistency with binder elasticity and recovery after deformation; hence, improving binder resistance to rutting. In general, the crumb rubber consists of elastomers (natural and synthetic rubber); it is well known that synthetic rubber increases the elastic behaviour of the bitumen modified binder, while the use of natural rubber leads to an increase in thermal behaviour (Mashaan, 2012; Memon and Chollar, 1997).

4.3 DSR Test Results

(DSR) was used to measure and determine the rheological properties of the bitumen binder.

4.3.1 Results of Rheological Parameters

The test results of G*, G', G'' and phase angle (δ) at 76 °C for various crumb rubber contents are shown in Figures 4.7.1 - 4.7.4. Table 4.7 illustrate the test results of G*, G', G'' and phase angle (δ) at 58, 70and 76°C, respectively. Table 4.7 show the effect of different CRM contents on storage modulus G', loss modulus G", complex modulus G* and phase angle. The significant increases in G', G" and G* were clearly affected by higher crumb rubber contents. Rubber mass increased during the swelling process of blend interaction mix, thus, leading to sufficient softening of the asphalt binder, however , the results of this research agree the finding of previous studies (Mashaan, 2012; Mahrez, 1999).

Temperature has enormous effect on asphalt binder properties. The variation in materials stiffness varies with differing temperatures susceptibility. Generally, asphalts has high temperature susceptibility are incapable to alleviate stresses easily at low temperatures which lead to more thermal cracking than softer asphalts. The results of this study show that the crumb rubber-modified binder manifest visco-elastic behaviours, with the binder exhibiting better relaxation upon applied stress on to the asphalt rubber. To sum up, the results confirm that when rubber content is increased, rubber-modified binders become less susceptible to temperature variations. Additionally at high service temperatures, improvements in asphalt performance properties are evident (Mashaan, 2012).

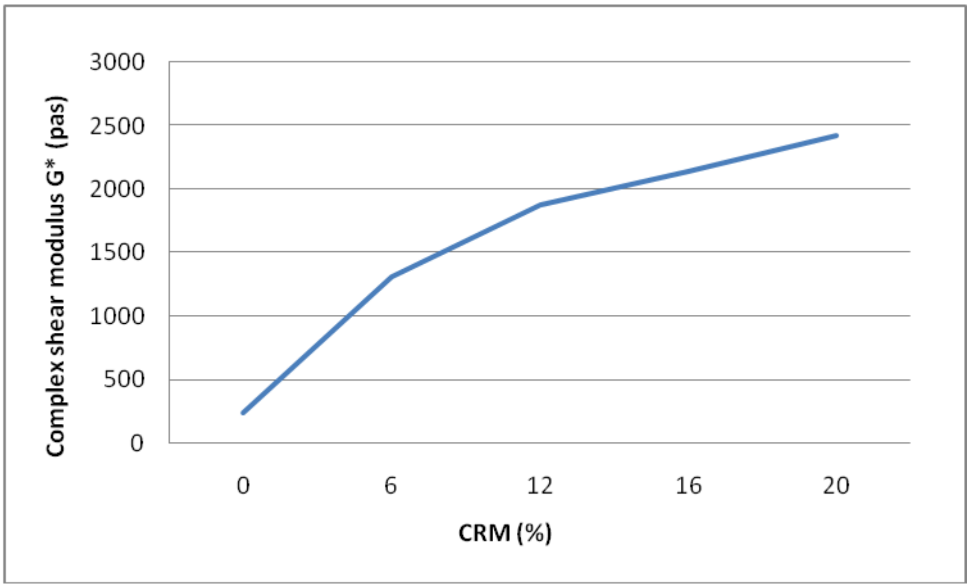

Figure 4.7.1: Shear modulus results vs. different CRM contents

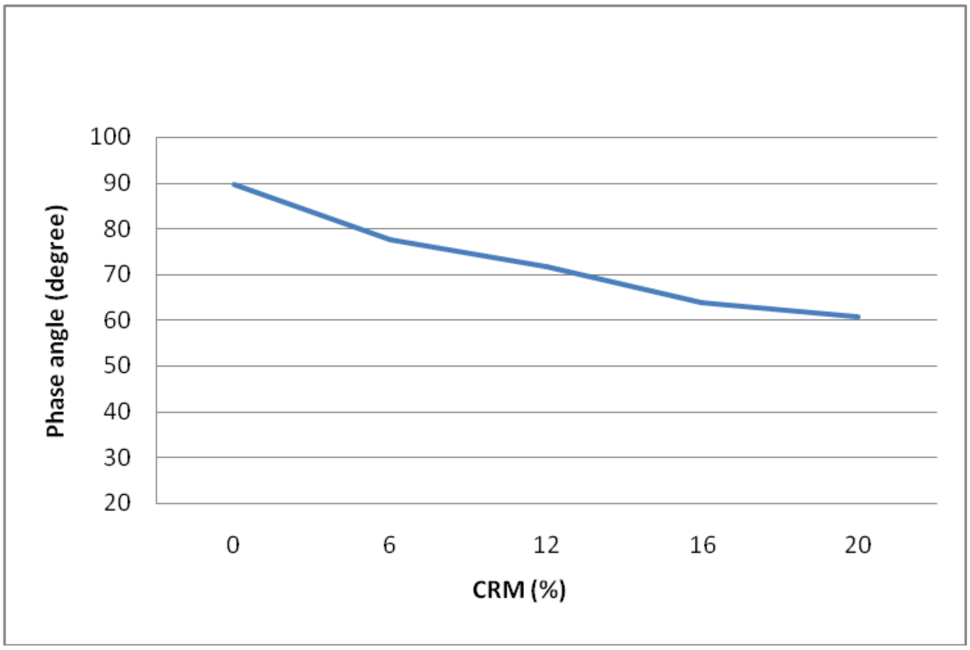

Figure 4.7.2 : Phase angle results vs. different CRM contents

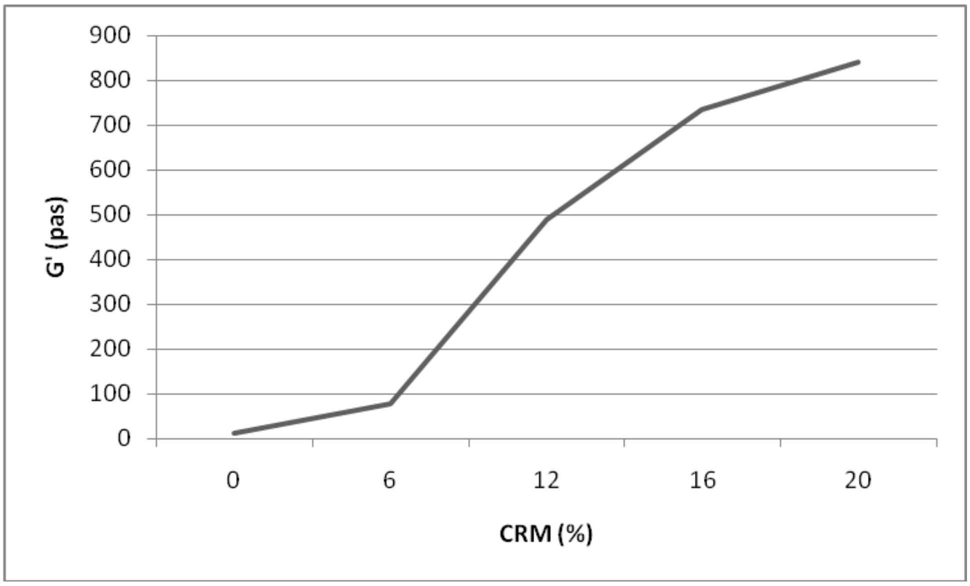

Figure 4.7.3 : G’ results vs. different CRM contents

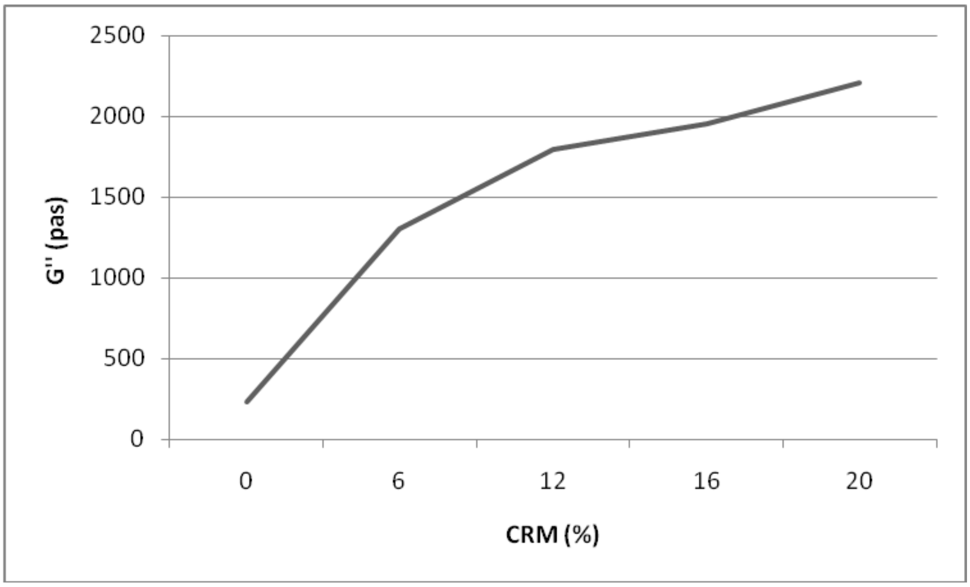

Figure 4.7.4 : G’’ results vs. different CRM contents

Table 4.7 : Viscous-elastic Properties vs. CRM content for various Temperatures

T(ºC)	DSR parameters	Unmodified bitumen	Rubberised Bitumen			
			6%	12%	16%	20%
58	G* (MPa)	235	1313	1872	2135	2423
	δ (degree)	89.23	81.89	73.8	67.8	60.55
	G’ (MPa)	12.6	140	600	721	799
	G´´ (MPa)	229	1300	1798	1977	2110
70	G* (MPa)	297	1252	1725	2185	2267
	δ (degree)	89.67	79.88	71.88	65.5	59.77
	G’ (MPa)	15.12	210	644	740	821
	G´´ (MPa)	297	1430	1830	1989	2132
76	G* (MPa)	308	1208	1700	2090	2120
	δ (degree)	89.67	77.5	71.7	63.78	58.77
	G’ (MPa)	18.45	298	710	785	833
	G´´ (MPa)	308	1570	1910	2003	2199

4.3.2 Temperature Effects on Rheology of CR Modified Bitumen

The rheological properties in terms of complex shear modulus and phase angle are presented in Figures 4.7.5 and 4.7.6, respectively. Complex shear modulus for all mixes is a function of temperature: whereby the higher the testing temperature, the lower the complex shears modulus is. Previously in Chapter 3, test procedures and specification were discussed. In comparison to the complex shear modulus of base bitumen, the complex shear modulus of the crumb rubber modified bitumen is higher; however, the results of this research agree the finding of previous studies (Mashaan, 2012; Hamed, 2010; Mahrez, 1999).

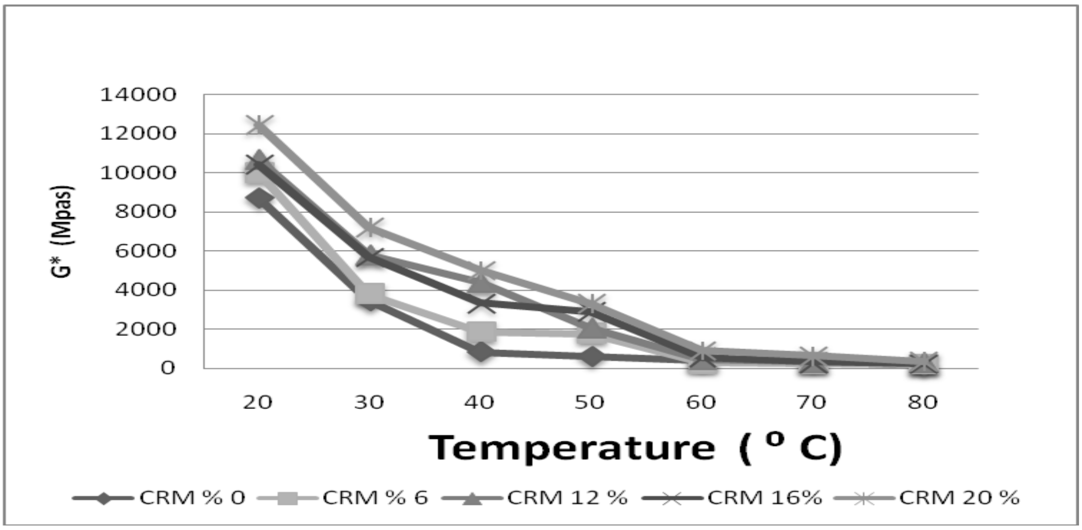

Figure 4.7.5: (G*) versus temperature for CRM bitumen at10 Hz

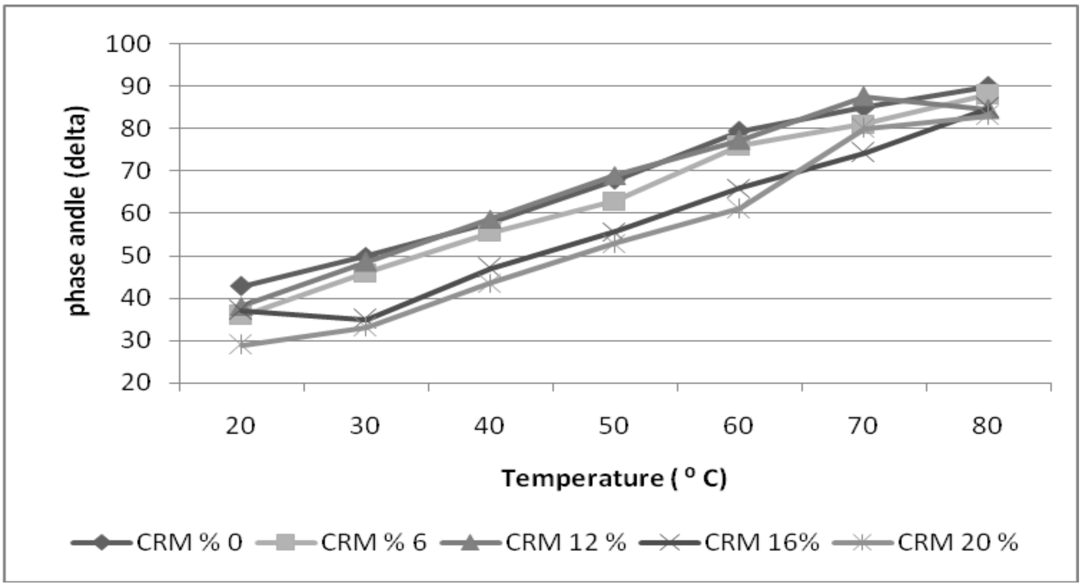

Figure 4.7.6: Phase angle versus temperature for CRM bitumen at 10 Hz

The following conclusions have been drawn from the results of the experimental investigations conducted on virgin asphalt binder and CRM modified asphalt. The change in the rheological properties of the CRM bitumen is evident as there is increased stiffness and elastic response in the bitumen. The primary reason for these changes in the rheological properties of the modified bitumen is due to the modifier absorbers of the lighter fractions of the bitumen which causes the bitumen to become stiffer. The change in rheological properties also has a detrimental effect on the cohesion properties. The properties of the modified bitumen is function of asphalt polymer net work formation, which includes numerous variables such as: asphalt composition; chemical structure of polymer; polymer molecular weight; physical properties of the polymer; the nature of the interaction between polymer and asphalt; asphaltenes content in the bitumen and mechanical history of blending bitumen with polymer such as mixing time shear rate and mixing temperatures (Mashaan, 2012; Mahrez, 1999).

4.3.3 Analysis of Fatigue Performance of Bitumen Binder after PAV

The (SHRP) had a maximum value of 5000 kPa for G* sin (δ) at intermedate temperature, and low values of these parameters are considered as good indicators of fatigue cracking resistance (The Asphalt Institute, 2003). At intermediate temperatures of (31 – 41 ºC) G*, δ and the fatigue resistance parameter, G* sin (δ) values, of the unmodified bitumen binders and rubberised binders after PAV test , which has followed RTFOT aging, were measured using the dynamic shear rheometer (DSR) and the results are presented in Figures 4.8.1 ,4.8.2 and 4.8.3, respectively. These temperature has been seletcted based on previous studies (Mahrez,1999; Hamed 2010). In general, the high crumb rubber content (20%) led to lower G* sin (δ) values of the rubberised bitumen binders which led to better fatigue resistance. Furthermore, the higher crumb rubber content, the lower G* sin δ at 31 ºC after PAV aging, thus leading to higher resistance to fatigue cracking; however , the results of this research agree the finding of previous studies (Mashaan 2012; Hamed 2010).

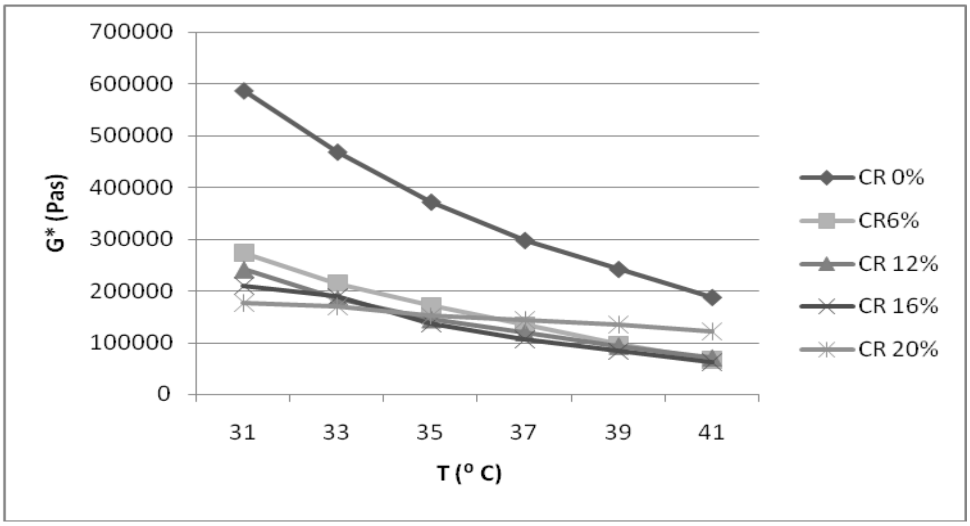

Figure 4.8.1 : G* results vs. temperatures for CRM binders after PAV

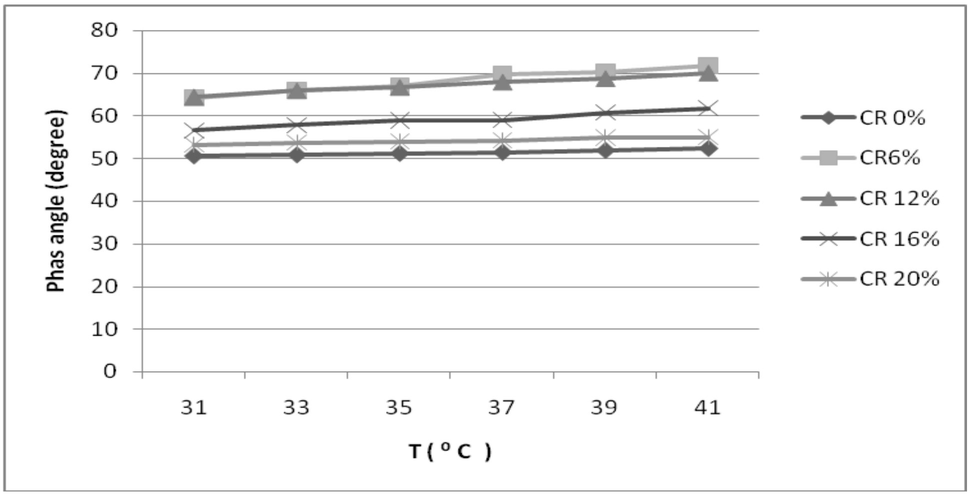

Figure 4.8.2 : δ results vs. temperatures for CRM binders after PAV

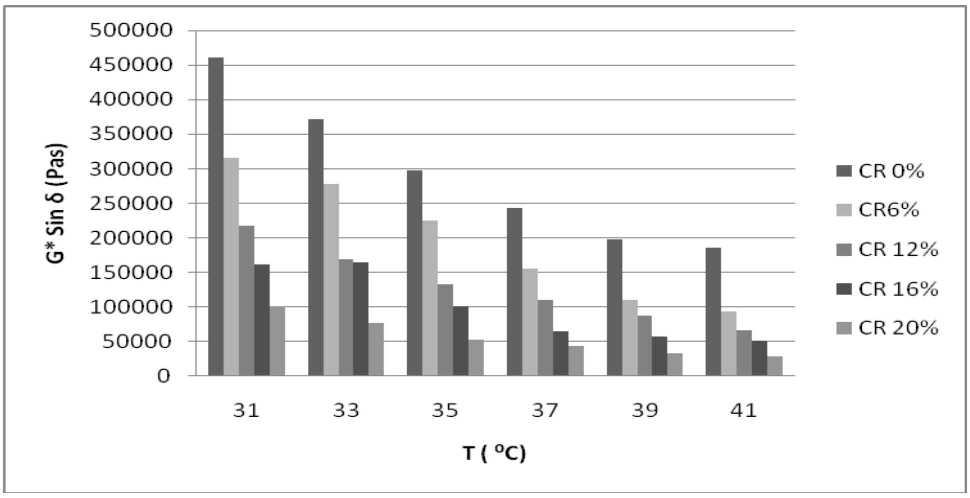

Figure 4.8.3: G* sin δ results vs. Temperature for CRM binders after PAV

4.4 Marshall Test Results

4.4.1 Marshaall sability

The results obtained for various CRM content for each binder contents are shown in Figure 4.9.

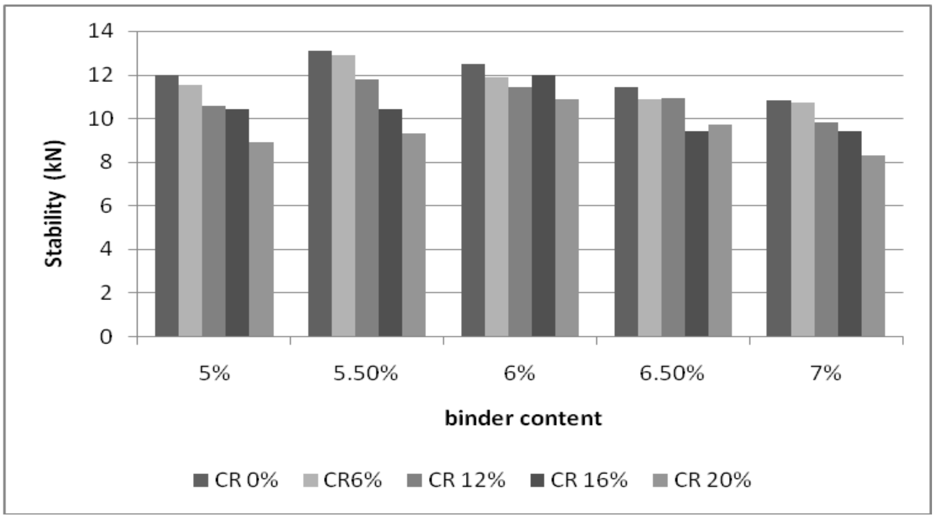

Figure 4.9 : Stability results vs. Binder content

Figure 4.9 illustrates the Marshall Stability value versus CRM content for different binder content. The diagrams show the stability values for the differing binder content varying in tandem with the CRM content. Once CRM is added the stability value elevated until the maximum level, which was approximately 12% of the used CRM. Then it began to decrease. In comparison to the control mix (mix with 0% CRM), the values of Marshall Stability were generally higher. Only mixture with a lower stability value was the mixture with 20% CRM. Stability is improved by adding CRM binders to the stone mix asphalt as better adhesion is developed between the materials in the mix (Mahrez, 1999), however , the results of this research agree the finding of previous studies (Hamed ,2010).

4.4.2 Marshall Flow

The results obtained for various CRM content for each binder contents are shown in Figure 4.10.

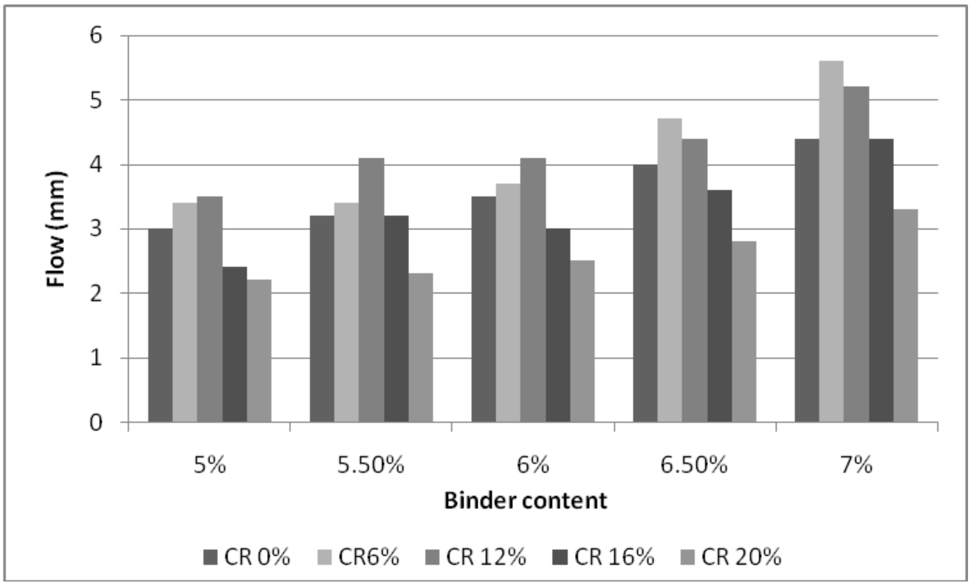

Figure 4.10 : Flow results vs. binder content

Figure 4.10 illustrates the Marshall flow value versus CRM content for different binder contents. The flow results show an increase when the binder content is increased with any specific CRM content. This is due to the percentage of additional bitumen which allows the aggregates to float within the mix resulting in increased of flow. Additionally, Figure 4.10 shows that the increase in CRM content in the SMA mixture does not necessary increase the flow values. Increased CRM content in the mix decreased the stability value. With more crumb rubber being added the stability is lowered. Thus, the addition of more CRM content increased the flow to an optimum level and with further addition of CRM in the mix; it was observed that there was an obvious decrease; however, the results of this research agree the finding of previous studies (Mahrez, 1999; Kumar *et al*., 2009).

4.4.3 Density of the compacted mix (CDM)

The results obtained indicated that binder content influences the compaction characteristics of the SMA mixtures, thus having a significant effect on the mix density. Figure 4.11 show that for any specific binder content, the density of the compacted mix is progressively increased, as the bitumen content of the mix increase. This is due to the bitumen filling in the void space of the aggregate particles.

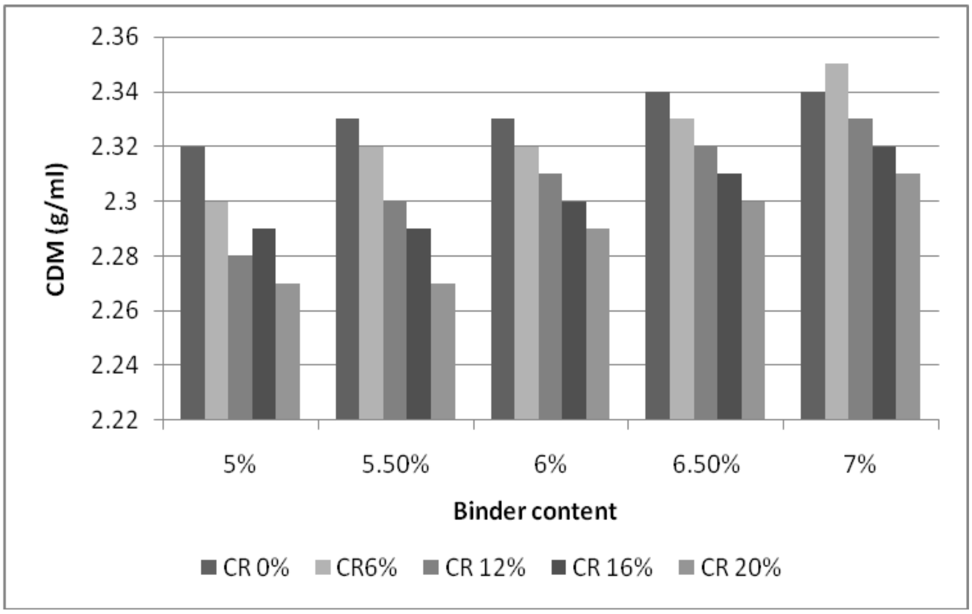

Figure 4.11 : CDM results vs. binder content

The results indicated a lower density for the mixtures with incorporation of crumb rubber. Figure 4.11 show that for any binder content, the density decreased as the crumb rubber is increased in the SMA mixtures. The increase in CRM content related to the increased bitumen being absorbed by the crumb rubber leading to extensive voids space with the aggregate particles, hence a decrease in mix density. An explanation for the varying densities of the mixtures is because of the viscosity effect on the compatibility of the mixtures. The increase in viscosity could be a result of the amount of asphaltenes in the bitumen which improves the viscous flow of the modified bitumen sample during the interaction process. The higher viscosity of the resulting binder provided better resistance during compaction of the mixture, thus resulting in lower density of the modified mix. This is in concurrence with previous finding by Mahrez (2008), which revealed that for ideal paving mixture a good correlation between binder viscosity and the compaction effort is required.

4.4.4 Voids in the Mix (VIM)

The durability of bituminous pavement is a function of the voids of the mix (VIM) or porosity. In general, the lower the porosity, the less permeable is the mixture and vice versa. The effect of the CRM content for different binder content on the porosity of the virgin mixture and SMA mixture showed in Figure 4.12.

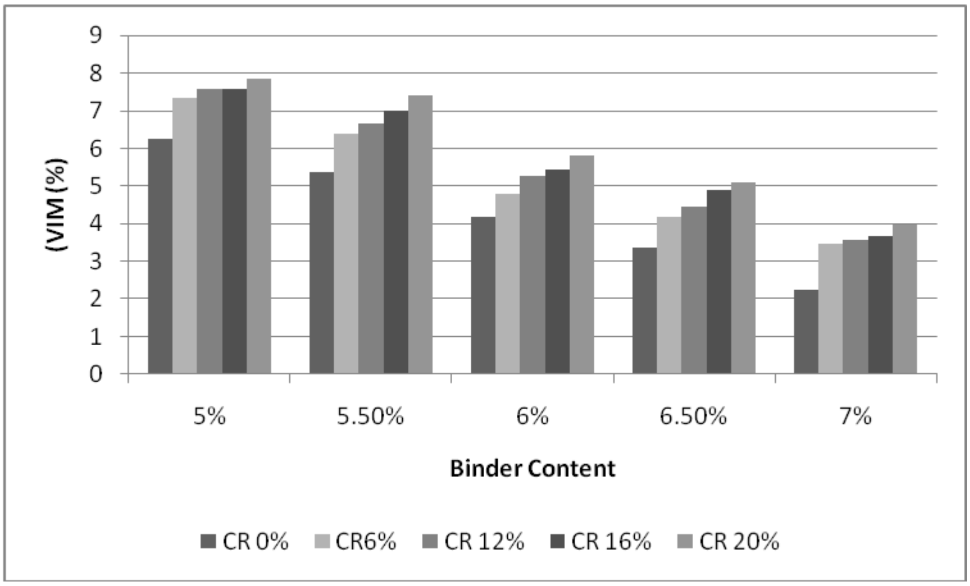

Figure 4.12 : VIM results vs. binder content

Figure 4.12 displays that for any binder content used, the increase in CRM content in the mixture is followed by an increase in the VIM, which is due to the contact point between the aggregates which is lower when the CRM is content increased. The high amount of crumb rubber particle absorbs the binder which is required to encapsulate the aggregate and subsequently fill the voids between aggregates. High porosity in the bituminous mixture means there are many voids providing passageways for the entry of damaging air and water through the mix. On the other hand, with low porosity, flushing accoutres whereby bitumen is squeezed out of the mix to the surface (Mahrez, 2008).

4.5 Indirect Tensile Test Results (Stiffness Modulus)

In order to determine the stiffness modulus, the resilient modulus of specimens was conducted in accordance with (ASTM D 4123). In asphalt samples, as a result of the excess strain, cracks appeared in relation to the tensile strength which was primarily micro-cracks. These cracks were perpendicular to the maximum tensile stress direction; integrating these micro-cracks by increasing the deformation results in a generation of macro-cracks. In tandem with the investigations, these cracks led to a fracture zone in the specimen. The length of this fracture zone can be viewed as a material parameter and can be construed to be a result of the fracture-energy of the material. Temperature and bitumen percentage are the two principal parameters which significantly influence the asphalt characteristics.

Figures 4.13 show the stiffness modulus (Mr) variation against bitumen content for asphalt mixtures reinforced with different contents of CRM and non-reinforced asphalt mixture (containing 0% CRM). As revealed in Figures 4.13, there is a marked variation between the reinforced and non-reinforced samples in the stiffness modulus (Mr). The increased bitumen has a significant impact on the stiffness modulus of specimens with varying CRM contents, due to the effect of the optimum bitumen percent being lower in non-reinforced samples. In reinforced asphalt samples with CRM, the crumb rubber content absorbs a portion of bitumen resulting in the optimum binder percent to increase. As the crumb rubber content is increased, more bitumen is absorbed, which in turn increases the optimum binder content of the mix. It is evident that the stiffness modulus of reinforced asphalt samples is higher compared to the non-reinforced samples; however, the results of this research agree the finding of previous studies (Arabani *et a*l., 2010).

Mixes with higher stiffness suggest that apart from being stiffer, they are more resistant to deformation. However, care must be exercised with very high stiffness mixes due to their lower tensile strain capacity to failure i.e such mixes are more likely to fail by cracking particularly when laid over foundations which fail to provide adequate support (Mahrez, 2008).

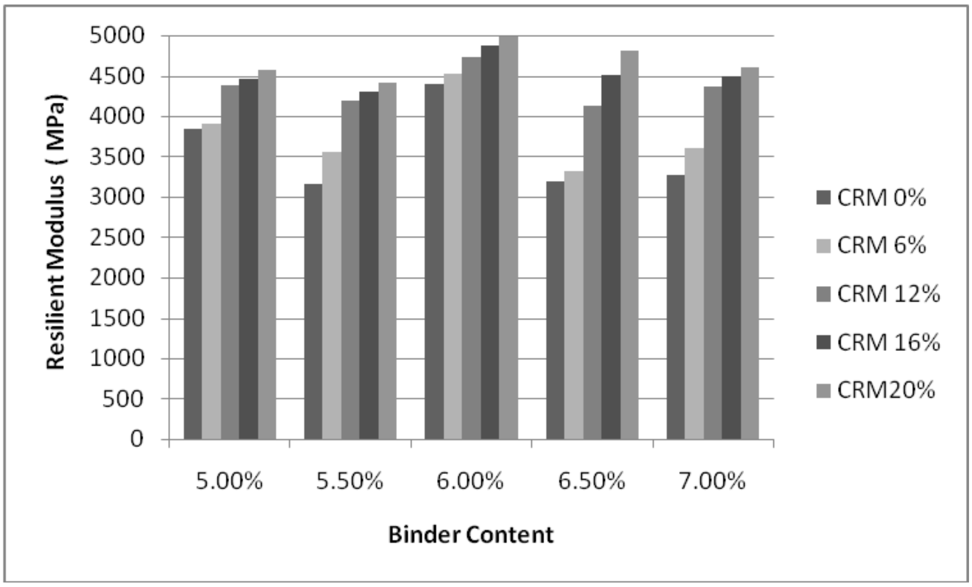

Figure 4.13 : Resilient Modulus vs. Bitumen Content

4.5.1 Effect of Temperature on Stiffness Modulus

Resilient modulus is a primary variable in mechanistic design approaches for improved pavement structures, with regards to dynamic stresses and corresponding strains in pavement response (Hamed, 2010).

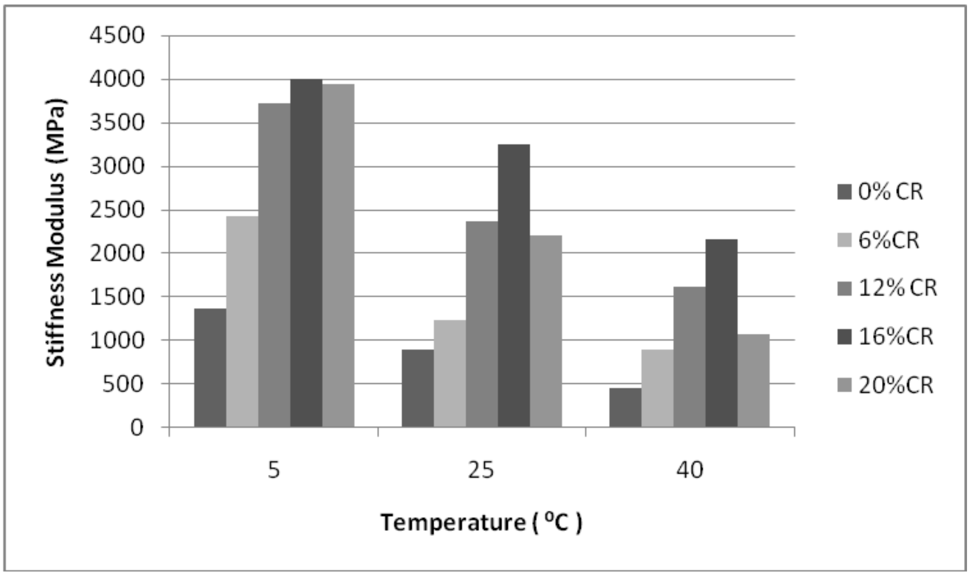

Figure 4.14 : Stiffness modulus versus temperature.

Figure 4.14 illustrates the stiffness modulus variation plotted against temperature for reinforced SMA asphalt samples containing different percentages of crumb rubber and non-reinforced SMA samples. Each sample was prepared with optimum binder content (AC 6%). The results indicate that when temperature is increased, the stiffness modulus of the asphalt samples is decreased. This occurs due to the change in the viscosity of bitumen as a result of the increase temperature which causes particle slippage in asphalt mixtures. This subsequently decreases the stiffness modulus of both the reinforced and non-reinforced samples. However, in comparison to the non-reinforced samples, the stiffness modulus of reinforced samples is found to be elevated as temperature increases with the presence of crumb rubber in the SMA asphalt samples, which can resist particle slippage. This in turn, reduces the stiffness modulus rate of decrease; hence the rate of stiffness modulus is lower in reinforced samples. However, this positive effect is attenuated by an extreme increase in the CRM, and the gap generated between talus material grains causes the stiffness modulus to decrease ; however , the results of this research agree the finding of previous studies (Arabani *et al*., 2010).

IDT results (stiffness modulus) indicate that the increase in CRM content produces an improvement in the elastic properties of the studied mixtures. Modified bitumen improves the resilient modulus of asphalt mixtures compared to the control mixtures, due to higher viscosity and thick bitumen films leading to better resilience properties. Thus modified bitumen produces asphalt concrete mixtures with improved stiffness and subsequently higher load bearing capacity. Furthermore, crumb rubber modified binders indicated lower temperature susceptibility. Mixes with modified binders indicated increased flexibility at decreased temperatures. This is due to the lower resilient modulus and higher stiffness as well as tensile strength at higher temperatures (Hamed, 2010).

4.6 Indirect Tensile Fatigue Test (ITFT)

The fatigue characteristics relating the accumulated strain with the number of cycles to failure for the SMA mixes with and without CRM reinforcement are presented in Table 4.15.1for various stresses (2000 , 2500 and 3000 N).

Table 4.15.1, display that the addition of CRM binder into SMA mixture improved the fatigue life and reduced the accumulated strain. SMA mixture reinforced with 12% CRM resulted in high fatigue life and hence lower strain value. Also, it is appears that the higher the stress, the lower the fatigue life is. At stress 2000 and 2500 N the fatigue life increased by about 25%, 29 %, 35% and 49% with the addition of 6- 20% CRM, respectively. In addition, it seems that SMA mixtures tend to have lower fatigue lives at higher stress levels.

In order to obtain representation of the fatigue life, the regression equation for each mixture along with the regression parameters for various CRM and stress values are illustrated in Tables 4.15.2 and 4.15.3. The basic fatigue life model confirms the aforementioned effects of crumb rubber content and stress levels on fatigue life. By having looked at fatigue model coefficients, some guidance may provide. As strong evidence, the high R^2 values are reasonably indicative of good models accuracy. Meaning, the fatigue life is higher for the mixtures reinforced with crumb rubber as compared with original mixture (without crumb rubber). The relationship obtained is rational in that lower fatigue life as the stress levels are increased.

Also, Table 4.15.1 indicates the variation of cyclic loading on the specimens containing varying percents of crumb rubber modifier. As the loading cycles are increased, the rate of tensile strain generation for both reinforced and non-reinforced specimens is found to be different. Crumb rubber modifier (CRM) leads to sustenance of higher tensile strains in asphalt samples. The high elasticity and tensile strength of crumb rubber allow asphalt samples to deter creep-caused-cracks as well as reduce the generation and propagation rate of micro-cracks. The high tensile strength evident in CRM can deter crack generation and the propagation of micro-cracks in asphalt samples (Arabani *et al*, 2010; Hamed, 2010). However, the number of cycles to failure different e for asphalt samples which contain various percentages of crumb rubber. Reinforced samples tend to have longer fatigue life compared with non-reinforced samples. From Tables 4.15.2 and 4.15.3, the behaviour model for asphalt samples containing various percentages of waste crumb_rubber and the respective correlation coefficients are presented as well. It is observed that deviation from the optimum CRM content decreases the fatigue life of reinforced asphalt samples. The CRM asphalt deters tensile and vertical cracks from being effortlessly formed by horizontal tensile stresses and stops them from propagating.

Table 4.15.1: Fatigue test results

CRM	σ (N)	με	Nf (Cycles)
0%	2000	1185	15,476
	2500	3354	3011
	3000	9893	345
6%	2000	677	19,999
	2500	2735	2354
	3000	6656	490
12%	2000	568	22566
	2500	2354	4657
	3000	4189	678
16%	2000	889	18767
	2500	2890	2890
	3000	6788	543
20%	2000	989	16566
	2500	3567	3567
	3000	7898	488

Table 4.15.2: Regression Equations for Fatigue Life Due to the Variation of stress along with Regression Parameters.

		Equation for	K_1	K_2	R^2
Stress Values	2000 N	$N_f = 2.276 \times 10^5 \left(\frac{1}{\varepsilon}\right)^{0.429}$	3.188×10^5	0.476	0.95
	2500 N	$N_f = 2.222 \times 10^7 \left(\frac{1}{\varepsilon}\right)^{1.112}$	2.624×10^7	1.090	0.92
	3000 N	$N_f = 1.344 \times 10^5 \left(\frac{1}{\varepsilon}\right)^{0.734}$	3.322×10^5	0.866	0.90

Table 4.15.3: Regression Equations for Fatigue Life Due to the Variation of CRM content at OBC along with Regression Parameters

		Fatigue Equations	K_1	K_2	R^2
CRM %	0	$N_f = 2.361 \times 10^9 \left(\frac{1}{\varepsilon}\right)^{1.725}$	2.261×10^9	1.345	0.96
	6	$N_f = 3.102 \times 10^9 \left(\frac{1}{\varepsilon}\right)^{1.631}$	3.002×10^9	1.898	0.92
	12	$N_f = 5.564 \times 10^8 \left(\frac{1}{\varepsilon}\right)^{1.613}$	4.664×10^8	1.212	0.90
	16	$N_f = 2.061 \times 10^9 \left(\frac{1}{\varepsilon}\right)^{1.616}$	2.161×10^9	1.676	0.92
	20	$N_f = 1.687 \times 10^9 \left(\frac{1}{\varepsilon}\right)^{1.699}$	1.487×10^9	1.888	0.94

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

For a long time, the crack potential of pavements under various loading conditions and temperature variations has been an issue for the purposes of application of asphalt pavements. The maintenance and rehabilitation pricing is drastically increased when crack generation in asphalt pavement appears. Two primary solutions which have been put forth by researchers are: first, application of a thicker asphalt pavement and, secondly, producing an asphalt mixture with modified characteristics. To date a wide array of experiments have been conducted to investigate the effects of CRM reinforcement to resolve the issue of the cracking potential of asphalt pavement. For the purpose of this study, the use of crumb rubber modifier (CRM) in reinforcing asphalt pavement has been introduced and investigated thoroughly.

Based on the study conducted, the following conclusions can be derived:

1- According to laboratory binder tests, it is clear rubber crumb content plays a main role in influencing the performance and rheological properties of rubberised bitumen binders. It could further enhance the performance properties of asphalt pavement resistance against deformation during construction and road services. The increase in rubber crumb content was from 6 - 20% thus indicating a linear increase in softening point, viscosity, elastic recovery and complex shear modulus. This phenomenon can explain by the absorption of rubber particles with lighter fraction oil of bitumen, leading to increase in rubber particles during swelling, during the blending process.

2- Dispersion of crumb rubber in bitumen resulted in the creation of a dense elastomeric rubber network within the mastic consequently impeding the mobility of the bitumen. This resulted in increased viscosity of the mastic. The flow behaviour of the resulting mixtures was affected by both the type and volumetric properties of the rubber- bitumen blend.

3- The stiffness modulus of reinforced SMA samples containing various contents of CRM is significantly high in comparison with that of non-reinforced samples. This increased stiffness modulus however is not related to increased brittleness of rein-forced asphalt samples. The stiffness modulus of reinforced samples is in fact less severely affected by the increased temperature compared to the non-reinforced samples.

4- According to the results, durability of rubberised bitumen binder improved significantly with crumb rubber content leading to higher resistance to aging. Thus, the crumb rubber modified bitumen was less susceptible to temperature susceptibility. Furthermore, the higher crumb rubber content, the lower G* sin δ at (31- 35 ^{0}C) after PAV aging, led to higher resistance to fatigue cracking. In addition, at lower rubber content (6%), the behaviour of the modified binders remained close to that of the base bitumen.

5- With the presence of crumb rubber, the fatigue life of CRM reinforced samples is significantly improved. The resistance of waste tyre rubber to generated horizontal tensile stresses decreases the formation of vertical cracks and prevents these cracks from propagating along the diameters of asphalt samples. This in turn improves the fatigue life of reinforced samples.

6- The relationships between fatigue life and stress level are rational; the higher the stress level is, the lower the fatigue life is and the higher the accumulative strain is. Due to the addition of CRM there has been marked improvement in fatigue life thus being more considerable at higher stress level than at lower stress level. Especially when heavy traffic load is applied, the enhancement of crumb rubber reinforced bituminous mix as a fatigue barrier is more remarkable. Regression models (fatigue equation) of fatigue life and accumulated strain due to different CRM content were developed for all samples. It was evident that high R^2 values are reasonably indicative of the model's accuracy.

5.2 Recommendations

Since different conclusions have been drawn from this research project, a list of recommendations is summarised as follows for further investigations in the future:

1- Use of different type of aggregate, aggregate gradation, different mixing methods and different compaction methods.

2- Selection of different bitumen sources with various penetration grade and also the use of other kind of recycled polymer such as waste plastic bottle.

3- A comparative assessment of the cost incurred for pavement constructions utilising various modified asphalt with those constructed using conventional binder.

4- Further rheological characterisation of the studied materials should be carried out using different temperature degrees, different test geometries and configurations and other rheometers, in order to obtain the master curves, the transition temperatures (Tg) and the fatigue resistance of the binders.
5- Use scanning electron microscope images (SEM) to evaluate binder-aggregate adhesion.
6- Conduct more studies for fatigue damage, including more mix variables and different rubber size to evaluate the effect of the particle size and texture of rubber.
7- In order to improve rutting and fatigue resistances as well as low temperature cracking, several different polymer types with same base asphalt blend ought to be tested for example by blending base bitumen with thermoplastic polymer, thermoplastic polymer and high boiling point petroleum oil.

REFERENCES

AASHTO Report, (1990). European asphalt study tour, Washington D.C., p31-61.

Abdullah, S. (1996). *A laboratory investigation of modified binders in bituminous mixes.* MSc. Dissertation, University of Malaya, Kuala Lumpur, Malaysia.

Aflaki, S., & Memarzadeh, M. (2011). Using two-way ANOVA and hypothesis test in evaluating crumb rubber modification (CRM) agitation effects on rheological properties of bitumen. *Construction and Building Materials*, *25*, 2094–2106.

Altgelt, K.H., & Boduszynski, M.M.(1994). Composition and Analysis of Heavy Petroleum Fractions. Marcel Dekker: New York.

Arabani, M., Mirabdolazimi , S.M. , & Sasani, A.R. (2010). The effect of waste tire thread mesh on the dynamic behaviour of asphalt mixtures. *Construction and Building Materials*, 24, 1060–1068.

Asphalt Institute. (2009*). Asphalt in pavement preservation & maintenance* (MS-16) 14th editing, 2009 printing. ISBN:9781934154588. Asphalt Institute, USA.

Asphalt Institute, (1990). *Mix design methods for asphalt concrete and other hot mix types,* Asphalt Institute Manual Series No 2(MS-2).

ASTM ,(2000). *Standard Test Method for Penetration of Bituminous Materials*, D5- 97, Annual Book of ASTM, Vol. 04.03, Easton, USA.

ASTM, (2006). *Standard Test Method for Softening Point of Bitumen (Ring – and – Ball Apparatus),* D 36- 06, Annual Book of ASTM, Vol. 04.04, Easton, USA.

ASTM, (2006). *Standard Test Method for Viscosity Determinations of Unfilled Asphalts Using the Brookfield Thermosel Apparatus,* D4402- 06, Annual Book of ASTM, Vol. 04.04, Easton, USA.

ASTM, (2000). *Determining the Rheological Properties of Asphalt binder for specification Purposes Using a Dynamic Shear Rheometer*, D-4 Proposal P246, Annual Book of ASTM, Vol. 04.03, Easton, USA.

ASTM, (2006). *Standard Test Method for Elastic Recovery of Bituminous Materials by Ductilometer*, D 6084- 06, Annual Book of ASTM, Vol. 04.03, Easton, USA.

ASTM, (1999). *Standard Test Method for Ductility of Bituminous Materials*, D113-99, Annual Book of ASTM, Vol. 04.03, Easton, USA.

ASTM, (2004). *Standard Test Method for Effect of Heat and Air on a Moving Film of Asphalt (Rolling Thin-Film Oven Test),* D2872- 04, Annual Book of ASTM, Vol. 04.03, Easton, USA.

ASTM, (2008). *Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)*, D 6521- 08, Annual Book of ASTM, Vol. 04.03, Easton, USA.

ASTM,(1989). *Test Method for resistance of plastic flow of bituminous mixtures using Marshall apparatus*. D1559- 89, Volume 04.03, Anuual book of ASTM standard, American society for testing and materials (ASTM),Philadelphia, USA.

ASTM,(1987). *Test Method for indirect tensile test for resilient modulus of bituminous mixture*. D 4123- 87,Volume 04.03, Anuual book of ASTM standard, American society for testing and materials (ASTM),Philadelphia, USA.

Bahia, H. U., & Anderson, D. A. (1993). Glass transition behavior and physical hardening of asphalt binders. *Journal of the Association of Asphalt Paving Technologists*, *62*, 93- 129.

Bahia, H. U., & Davies, R. (1994). Effect of crumb rubber modifiers (CRM) on performance- related Properties of asphalt binders. *Journal of the Association of Asphalt Paving Technologists*, *63*, 414-449.

Bahia, H. U., & Anderson, D. A.(1995a). Strategic highway research program binder rheological parameters: background and comparison with conventional properties. *Transport Research Record*, No.1488, 32-39.

Bahia, H. U., & Davies, R. (1995b). Factors controlling the effect of crumb rubber on critical properties of asphalt binders. *Journal of the Association of Asphalt Paving Technologists* , *64*, 130- 162.

Bandini, P. (2011). *Rubberized Asphalt Concrete Pavements in New Mexico: Market Feasibility and Performance Assessment.* Prepared for the New Mexico Environmental Department and the South Central Solid Waste Authority, Department of Civil Engineering, New Mexico State University, USA.
Link:http://www.nmenv.state.nm.us/swb/documents/RubberizedAsphaltConcretePavementsInNM_Final.pdf

Beaty, A. N. S. (1992). Latex- modified bitumen for improved resistance to brittle fracture. Highway and Transportation, p.p 32-41.

Becker ,Y., Mendez ,M.P., & Rodriguez,Y.(2001). Polymer modified asphalt. *Vision Tecnologica*, *9*(1), 39-50.

Bernd S.(1996). *Stone mastic asphalt, pavement technology for the new millennium*, Report from road construction division, J. Rettenmaier & Sohne, Germany.

Billiter,T.C., Davison, R. R., Glover, C. J., & Bullin, J. A. (1997a). Physical properties of asphalt -rubber binder. *Petroleum Science and Technology*, *15*(3-4), 205- 236.

Billiter, T.C., Chun, J.S., Davison, R.R., Glover, C.J., & Bullin, J.A. (1997b). Investigation of the curing variables of asphalt-rubber binder. *Petroleum Science and Technology*, *15* (5-6), 445-469.

Branco, F., Pereira, P., & Picado-Santos, L. (2005). *Pavimentos Rodoviários*. Edições Almedina, Coimbra.

Brennen, M., Tia, M., Altshaeffl, A., & Wood, L. E. (1983). Laboratory investigation of the use of foamed asphalt for recycled bituminous pavements. *Transportation Research Record*, no. 911, p.80.

British Standard, (2004). *Standard test method for resistance to fatigue of bituminous mixture*, BS EN 12697-24:2004.

Brown, E. R. & Hemant, M. (1993). Evaluation *of laboratory properties of SMA mixture*. National Center for Asphalt Technology, Auburn University, NCATR Report no. 93-5.

Brown, D. R., Jared, D., Jones, C., & Watson, D. (1997). Georgia's experience with crumb rubber in hot – mix asphalt. *Transportation Research Record: Journal of Transportation Research Board.* 1583, 45-51.

Castro , M., & Jose´, A. S. (2008). Estimation of asphalt concrete fatigue curves – A damage theory approach. *Construction and Building Materials* , 22 , p.1232–1238.

Charania, E., Cano, J. O., & Schnormeier, R. H. (1991). Twenty- year study of asphalt rubber pavement in Phoenix, Arizona. *Transportation Research Record: Journal of Transportation Research Board* ,1307, 29-38.

Claudy, P., Letoffe, J. M., King, G.N., Planche, J. P., & Brule, B. (1991). Characterization of paving asphalt by differential scanning calorimetry. *Fuel Science and Technology International, 9*(1), 71- 92.

Claxton, M., Lesage, J., & Planc, L. (1996). The use of binder rheological properties to predict the perfprmance of asphalt mixes. Proceedings, ARRB conference, Part 2, p.311.

Croney, D. & P. Croney**.** (1992). *The Design and Performance of Road Pavement* (2nd. ed). London: McGraw-Hill Book Co.

Davidson, J.K., & Kennepohl, G.J. (1992). Introduction of stone mastic asphalt in Ontario. Report for AAPT in Chaleston, South Carolina.

Donger, R., Youtcheff, J., & Anderson, D. (1996). Better roads through rheology. *Applied Rheology, 6*, 75- 82.

Eaton, R.A., Roberts, R.J., & Blackburn, R.R. (1991). Use of scrap rubber in asphalt pavement surfaces. Special report (Cold Regions Research and Engineering Laboratory (U.S.)); 91-27, Vol iii, p.14.

Ferguson, J., & Kemblowski, Z. (1991). *Applied Fluid Rheology*. Essex, New York: Elsevier science publishers Ltd.

Fernando, M. J., & Mesdary, M. S. (1988). *Effect of rubberised asphalt on Portland cement coated aggregates in road construction under severe conditions of traffic and climate.* Pooceddings, Australian Road Research Board (ARRB) conference, vol.14, p. 78.

Fernando, M. J., & Guriguis, H. R. (1984). Natural rubber for improved surfacing. Australian Road Research Board (ARRB). Vol.12, Part 2, p.p 121- 129.

Guddati, M. N., Z. Feng, & Kim, Y.R. (2002). Toward Micromechanics-Based Procedure to Characterize Fatigue Performance of Asphalt Concrete. *Journal of the Transportation Research Board* , l 1789, 142-167.

Hadley, W., Hudson, W., & Kennedy, T.W. (1970). A Method of Estimating Tensile Properties of Materials Tested in Indirect Tension, Center for Highway Research, Texas, 41, 7-98

Hamed, F.K. M. (2010). *Evaluation of fatigue resistance for modified asphalt concrete mixture based on dissipate energy concept*. Phd.Thesis, Technische University Darmstadt, Germany.

Hanson, D.I., Foo, K. Y., Brown, E. R., & Denson, R. (1994). Evaluation and characterization of a rubber-modified hot mix asphalt pavement. *Transportation Research Record: Journal of Transportation Research Board,* 1436, 98-107

Huang, Y., Bird, R.N., & Heidrich, O. (2007). A review of the use of recycled solid waste materials in asphalt pavements. *Resources, Conservation and Recycling* ,52 , 58 – 73.

Huffman, J.E. (1980). *Sahuaro Concept of Asphalt-Rubber Binders*. Presentation at the First Asphalt Rubber User Producer Workshop, Arizona.

Isacsson, U., & Lu, X. (1999). Characterization of bitumens modified with SEBS, EVA and EBA polymers. *Journal of Materials Science,* 34(15), 3737- 3745

Jensen, W., & Abdelrahman, M. (2006). Crumb rubber in performance-graded asphalt binder. Final Report SPR-01.Nebraska Department of Roads University of Nebraska-Lincoln ,(05),P585.

Jeong, K.D., Lee, S.-J., Amirkhanian, S. N., & Kim, K. W. (2010). Interaction effect of crumb rubber modified asphalt binder. *Construction and Building Materials,* 24, 824-831.

Jones, D.R.IV, & Kennedy, T.W. (1992). The Asphalt Model: Results of the SHRP Asphalt Research Program. *SHRP A-001*. Center for Transportation Research University of Texas, Austin, Texas, USA.

Joseph, P.E., Hass, R., Phang, W.A.& Rothenburg, L.(1987). *Low temperature reflecting cracking through asphalt overlays,* Proceeding 6th International conference on structural design of asphalt pavements, Vol.1, p. 13-17.

Katman, H. Y. (2006). *Effects of rubberised bitumen on properties of porous mixes*. MSc. Dissertation, University of Malaya, Kuala Lumpur, Malaysia.

Kumar, P., Mehndiratta, H. C., & Singh, K. L. (2009). Rheological properties of crumb rubber modified bitumen – A lab study. *Journal of Scientific & Industrial Research*, 68, 812- 816.

Larsen, H. J. E., Wohlk, C. J., & Hall, B. A. (1988). *Modified bitumen*. The 14th ARRB Conference, Canberra, Australia. P. 222-229.

Lewandowski, L.H. (1994). Polymer modification of paving asphalt binders. *Rubber Chemistry and Technology , 67*(3), 447- 480.

Little Dallas N., & Kamyar M. (1990). Overview of a Rational Asphalt Concrete Mixture Design for Texas. *Journal of the Transportation Research Board* ,1269, 125-132.

Liu, S., Cao, W., Fang, J., & Shang, S. (2009). Variance analysis and performance evaluation of different crumb rubber modified (CRM) asphalt. *Construction and Building Materials*, 23, 2701-2708.

Loeber, L., Muller, G., Morel, J., & Sutton, O. (1998). Bitumen in colloid science: a chemical, structure and rheological approach. *Fuel*, 77(13), 1443 – 1450.

Lundy, J.R., Hicks, R. G., & Zhou, H. (1993). Ground Rubber Tires in Asphalt-Concrete Mixtures-Three Case Histories. *ASTM Special Technical Publication* **,**STP1193 PP.262-275.

Mahrez, A. (2008). *Properties and performance of stone mastic asphalt reinforced with glass fibre*. Phd. Thesis. Faculty of Engineering, University of Malaya, Malaysia

Mahrez, A, & Rehan, M. (2003) Rheological evaluation of aging properties of crumb rubber- modified bitumen. *Journal of the Eastern Asia Society for Transportation Studied*, 5, 820-833.

Mahrez, A. (1999). *Properties of rubberised bitumen binder and its effect on the bituminous mix.* MSc. Dissertation, Faculty of Engineering, University of Malaya, Malaysia.

Majidzadeh, K., Llves, G. J., & Luther, M.S. (1983). Reflection cracking models: review and laboratory evaluation of engineering fabrics. *Transportation Research Board* ,916, p. 18-25.

Mashaan, N. S. (2012). *The effect of crumb rubber modifier to the properties and rheological behaviour of asphalt binder*. MSc. Dissertation, University of Malaya, Kuala Lumpur, Malaysia.

Mashaan, N.S., Ali A. H., Karim, M.R., & Abdelaziz M .(2010). Influence of blending interaction of crumb rubber modified bitumen on pavement properties. *Proceeding of Malaysian Universities Transportation Research Forum and Conferences 2010 (MUTRFC2010), 21 December 2010, Universiti Tenaga Nasional, Malaysia, ISBN 978-967-5770-08-1.*

Masson, J.-F., Leblond, V., & Margeson, J. (2006) . Bitumen Morphologies by Phase-Detection Atomic Force Microscopy. *Journal of Microscopy*, 221 (1), pp. 17-29.

Maupin Jr, G.W. (1996). Hot mix asphalt rubber applications in Virginia. *Transportation Research Record: Journal of Transportation Research Board*, 1530, 18-24.

McGennis, R.B. (1995). Evaluation of physical properties of fine crumb rubber – modified asphalt binder. *Transportation Research Record :Journal of Transportation Research Board*, 1488, 62-71.

McLean, J.D., & Kilpatrick, P.K. (1997). Comparison of Precipitation and Extrography in the Fractionation of Crude Oil Residua . *Energy Fuels*, 11, pp. 570-585.

Memon, G. M., & Chollar, B. H.(1997). *Thermal characterization of crumb rubber modifier Asphalt.* 24th NATAS Conference, San Francisco.CA.

Merdrignac, I., & Espinat, D. (2007). Physicochemical Characterization of Petroleum Fractions: the State of the Art . *Oil & Gas Science and Technology – Rev*. IFP, 62 (1), pp. 7-32.

Navarro, F.J., Partal, P., Martinez- Boza, F., Valencia, C., & Gallegos, C. (2002). Rheological characteristics of ground tire rubber- modifier bitumens. *Chemical Engineering Journal* , 89 ,53-61.

Navarro, F.J., Partal , P., Garcia-Morales, M., Martin-Alfonso, M.J., Martinez-Boza, F., Gallegos, C., & Diogo, A. C. (2009). Bitumen modification with reactive and non-reactive (virgin and recycled) polymers: A comparative analysis. *Journal of Industrial and Engineering Chemistry*, 15, 458–464

O'Flaherty, C. A. (1988). *Highway Engineering Textbook* (3rd ed.). UK:. Edward Arnold.

Oliver, J. W.H. (1981). Modification of paving asphalts by digestion with scrap tyre. *Transportation Research Record*, no 821, p37.

Pauli, A.T., Branthaver, J.F., Robertson, R.E., Grimes, W., & Eggleston, C.M. (2001). Atomic Force Microscopy Investigation of SHRP Asphalts: Heavy Oil and Resid Compatibility and Stability. *American Chemical Society.Division of Petroleum Chemistry*, San Diego, California, pp. 110-114.

Peralta E. J.F. (2009). *Study of the interaction between bitumen and rubber*. MSc. Dissertation, Faculty of Engineering, University of Minho,

Peterson, J. C. (1984). Chemical composition of asphalt as related to asphalt durability – state of the art. *Transportation Research Record*: *Transportation Research Board* ,999, 13-30.

Pfeiffer, J.P., & Saal, R.N.J. (1940) . Asphaltic Bitumen as Colloid System. *The Journal of Physical Chemistry*, 44 (2), pp. 139–149.

Pfeiffer, J.P., &Van Doormaal, P.M. (1936). The Rheological Properties of Asphaltic Bitumen. *Journal of the Institute of Petroleum Technologists*, 22, p. 414.

Piggott, C.J., Sadler, D.E., & Villiers, E.M. (1977). *Bitumen rubber asphalt experiences in the RSA*. In proceedings: 7th Conference on asphalt pavement for southern Africa.

Raad, L., & Saboundjian, S. (1998). Fatigue Behavior of Rubber Modified Pavements. *Transportation Research Record* 1639, pp 73–82.

Ratnasamy M., & Bujang, .BK. (2006). Laboratory diameteral fatigue performance of SMA with cellulose oil palm fiber. *American Journal of Applied Sciences,* 3(9), 2005-2010.

Read, J. M., & Whiteoak, C.D. (2003). *The Shell Bitumen Handbook.* (5th ed). London: Thomas Telford Publishing.

Read, J. M. (1996). *Fatigue Cracking of Bituminous Paving Mixtures*. PhD. Thesis Department of Civil Engineering, University of Nottingham, UK.

Robertson, R.E. (1991). Chemical properties of asphalts and their relationship to pavement performance. SHRP-A/UWP-91-510. Washington, D.C.: Strategic Highway Research Program. http://onlinepubs.trb.org/onlinepubs/shrp/SHRP-91-510.pdf

Rosner, J.C., & Chehovits, J.G. (1982). Chemical and physical properties of asphalt-rubber mixtures – Part III, vol. 4 – Physical Properties of Field-Mixed Asphalt-Rubber Mixtures and Comparisons of Lab and Field-Mixed Asphalt-Rubbers. Report no. FHWA/AZ-82/159/4, Arizona Department of Transportation.

Rozeveld, S.J., Shin, E., Bhurke, A., France, L., & Drzal, L. (1997). Network morphology of straight and polymer modified asphalt cements. *Microscopy Research and Technique*, 38, 529-543.

Samsuri, A. (1997). *Properties of rubberised bitumen from reclaimed rubber*. In proceeding: Conference on the use of rubberised bitumen in the road construction, Malaysia. P.15-23.

Shaw, P. S. (1980). Stress-Strain Relationships for Granular Materials under Repeated Loading. PhD Thesis, Department of Civil Engineering, University of Nottingham, UK.

SHRP, (1994). Standard method for the test of determining of fatigue life of compacted bituminous mixture subjected to repeated flexural bending, SHRP designation: M-009.

Soleymani, H. R., Zhai, H., & Bahia, H. (2004). Role of modified binders in rheology and damage resistance behavior of asphalt mixture. *Transportation Research Record: Transportation Research Board*, 1875, 70- 79.

Stroup-Gardiner, M., Chadbourn, B., & Newcomb, D. E. (1996). Babbitt, Minnesota: case study of pretreated crumb rubber modified asphalt concrete. *Transportation Research Record: Journal of Transportation Research Board*, 1530, 34-42.

Susanne, O.,(2000). Prediction of performance of stone mastic asphalt, Presented at the Young researchers Forum. By Society off Chemical Industry (SCI) Construction Materials Group. SCI publication, pp.74.

The Asphalt Institute. (2003). *Performance graded asphalt binder specification and testing*. Superpave Series (SP-1).Lexington, KY.

Thompson, D.C., & Hoiberg , A.J.(Eds). (1979). *Bituminous Materials: Asphalt Tars and Pitches*. New York : Krieger Publishing Co.

Utah Department of Transportation (Utah DOT) (2003). *Technical Bulletin MT-03.06*, October 21, 2003. http://www.udot.utah.gov/main/uconowner.gf?n=200511230848151. Accessed April 15, 2011.

Van der Poel, C. (1954). A general system describing the viscoelastic properties of bitumen and its relation to routine test data. *Journal of Applied Chemistry, 4*, 221- 236.

Wahhab, H.& Al-Amri, G. (1991). Laboratory Evaluation of Reclaimed Rubber Asphalt Rubber Asphalt Concrete Mixes. *ASCE Journal of Materials in Civil Engineering* , 3, 189-203.

Wang, H., You, Z., Mills-Beale, J., & Hao, P. (2012). Laboratory evaluation on high temperature viscosity and low temperature stiffness of asphalt binder with high percent scrap tire rubber. *Construction and Building Materials*, 26, pp 583–590.

Wayne, L.K. & Roberts, F. L. (1988). Perliminary framework for a rotational bituminous mix design to mitigate pavement failures. Proceedings, ARRB, vol. 14, p.45.

Witczak, M. W., Kaloush, K. E., Pellinen, T., El-Basyouny, M., &Von Quintus, H. (2002). Simple Performance Test for Superpave Mix Design. National Cooperation Highway Research Program (NCHRP) Report 465, Arizona State University.

Wood, L. E. & Mamlouk M. S. (1981). Effect of aggregate top size on asphalt emulsion mixture properties. *Transportation Research Record*, no. 821, p.44.

Yildrim, Y. (2007). Polymer modified asphalt binder. *Construction and Building Materials*, 21, 66-72.

Youtcheff, J.S., & Jones, D.R.IV. (1994) . Guideline for Asphalt Refiners and Suppliers. Strategic Highway Research Program, National Research Council, Washington, DC.

Zaman, A. A., Fricke, A. L. & Beatty, C. L. (1995). Rheological properties of rubber-modified asphalt. *Journal of Transportation Engineering*, no. 121, p. 6.

Zhu, H., & Carlson, D. (2001). A spray based crumb rubber technology in highway noise reduction application. *Journal of Solid Waste Management* , 27(1), 27-33.

APPENDIX A

Data results of asphalt binder test and SMA mixture

Penetration results

CRM	Penetration (0.1 mm)		
	Mean	Std. Dev.	n
0	89.67	2.08	3
6	79.67	2.52	3
12	54.67	2.52	3
16	50	6.56	3
20	36.67	2.9	3

Softening Point Results

CRM	Softening Point (^{o} C)		
	Mean	Std. Dev.	n
0	45.5	0.7	2
6	48.8	1.1	2
12	55.8	0.4	2
16	57.8	0.4	2
20	60.8	0.4	2

Brookfield Viscosity Results at 135˚C

CRM	Viscosity at 135 ºC		
	Mean	Std. Dev.	n
0	659.3	5.5	3
6	1109.3	9	3
12	2588.7	4.5	3
16	3095.7	4	3
20	3190.7	5	3

Effect of temperature on Brookfield viscosity results

	Temperatures (ºC)						
	90	100	120	135	150	160	170
0 % CRM	10354	4841	3780	659.3	250	220	135
6% CRM	10540	5000	4100	1109.3	325	310	220
12 % CRM	10711	6824	6400	2588.7	865	350	410
16 % CRM	10450	7692	3340	3095.7	762	525	493
20 % CRM	13094	8055	4200	3190.7	983	611	522

Ductility Results

CRM	Ductility at 25 °C		
	Mean	Std. Dev.	n
0	93.0	2.8	2
6	82.5	3.5	2
12	63.5	2.1	2
16	48.5	2.1	2
20	40.0	4.2	2

Elastic Recovery Results

CRM	Elastic recovery at 25 °C		
	Mean	Std. Dev.	n
0	17.5	0.7	2
6	63.5	3.5	2
12	74.0	2.8	2
16	82.0	2.8	2
20	90.3	0.4	2

G* results at intermediate temperatures for different CRM content

	T (^{o}C)					
	31	33	35	37	39	41
CRM 0%	587000	468700	372100	298020	242100	187934
CRM 6%	273111	214500	171200	133800	96680	66880
CRM 12%	241649	185057	145060	119702	93743	70825
CRM 16%	210153	189198	137039	105573	82578	61459
CRM 20%	176292	169793	150943	143335	134588	12746

δ results at intermediate temperatures for different CRM content

	T (^{o}C)					
	31	33	35	37	39	41
CRM 0%	50.65	50.88	51.21	51.45	51.88	52.33
CRM 6%	64.12	65.89	66.89	69.56	70.11	71.67
CRM 12%	64.44	65.88	66.67	67.89	68.7	69.99
CRM 16%	56.56	57.78	58.89	58.99	60.77	61.8
CRM 20%	53.22	53.67	53.99	54.33	54.89	54.99

G* sin δ results at intermediate temperatures for different CRM content

	T (^{o}C)					
	31	**33**	**35**	**37**	**39**	**41**
CRM 0%	461107	371346	297172	243795	198582	185399
CRM 6%	315681	278118	224695	156232	110575	93383
CRM 12%	217999	168899	133199	110899	87339	66549
CRM 16%	162339	165315	101499	64799	57747	51558
CRM 20%	101199	76787	52999	44443	33000	28776

Stability results (kN) for different CRM content

	Binder content				
	5%	**5.5%**	**6%**	**6.5%**	**7%**
CRM 0%	11.99	13.10	12.5	11.40	10.8
CRM 6%	11.50	12.90	11.9	10.89	10.7
CRM 12%	10.55	11.80	11.4	10.90	9.8
CRM 16%	10.40	10.40	11.99	9.4	9.4
CRM 20%	8.9	9.30	10.89	9.7	8.3

Flow results (mm) for different CRM content

	Binder content				
	5%	**5.5%**	**6%**	**6.5%**	**7%**
CRM 0%	3.0	3.2	3.5	4	4.4
CRM 6%	3.4	3.4	3.7	4.7	5.6
CRM 12%	3.5	4.1	4.1	4.4	5.2
CRM 16%	2.4	3.2	3	3.6	4.4
CRM 20%	2.2	2.3	2.5	2.8	3.3

CDM results (g/ml) for different CRM content

	Binder content				
	5%	**5.5%**	**6%**	**6.5%**	**7%**
CRM 0%	2.32	2.33	2.35	2.34	2.35
CRM 6%	2.30	2.32	2.33	2.33	2.35
CRM 12%	2.28	2.30	2.31	2.32	2.33
CRM 16%	2.29	2.29	2.30	2.31	2.32
CRM 20%	2.27	2.27	2.29	2.30	2.31

VIM results (%) for different CRM content

	Binder content				
	5%	**5.5%**	**6%**	**6.5%**	**7%**
CRM 0%	6.24	5.38	4.19	3.36	2.25
CRM 6%	7.34	6.37	4.78	4.19	3.45
CRM 12%	7.56	6.65	5.28	4.45	3.56
CRM 16%	7.57	6.98	5.43	4.89	3.68
CRM 20%	7.83	7.40	5.81	5.10	3.96

Stiffness Modulus results (Mpa) for different CRM content

	Binder content				
	5%	**5.5%**	**6%**	**6.5%**	**7%**
CRM 0%	3850	3160	4400	3200	3270
CRM 6%	3900	3550	4530	3320	3600
CRM 12%	4384	4200	4740	4130	4370
CRM 16%	4470	4310	4870	4510	4489
CRM 20%	4570	4410	4990	4810	4600

LIST OF PUBLICATIONS

Journal papers

- **Asim Hassan Ali**, Mashaan, N.S., M.R. Karim, 2013. Investigations of Physical and rheological Properties of Aged Rubberised Bitumen. *Advances in Materials Science and Engineering*. Volume 2013, Article ID 239036, 7 pages. http://dx.doi.org/10.1155/2013/239036

- Mashaan, N.S., **Asim Hassan Ali**, M.R., Karim and M. Abdelaziz, 2011a . Effect of crumb rubber concentration on the physical and rheological properties of rubberised bitumen binders. International Journal of Physical Science ,6 (4): 684-690.

- Mashaan, N.S., **Asim Hassan Ali**, M.R. Karim and M .Abdelaziz, 2011b . Effect of blending time and crumb rubber content on properties of crumb rubber modified asphalt binder. International Journal of Physical Science, 6 (9): 2189- 2193.

- Mashaan, N.S., **Asim Hassan Ali**, M.R. Karim and M .Abdelaziz, 2012.An overview of crumb rubber modified asphalt. International Journal of Physical Science, 7 (2): 166- 170.

- **Asim Hassan Ali**, Mashaan, N.S., M.R. Karim, 2012. Crumb Tyre Rubber as An Additive for Bituminous Materials– Lab Study. *International Journal of Modern Engineering Science,* 1(1*),17-23.*

Conference Paper

- Mashaan, N.S., **Asim Hassan Ali**, M.R. Karim and M .Abdelaziz, 2010. Influence of blending interaction of crumb rubber modified bitumen on pavement properties. *Proceeding of Malaysian Universities Transportation Research Forum and Conferences 2010 (MUTRFC2010), 21 December 2010, Universiti Tenaga Nasional, Malaysia, ISBN 978-967-5770-08-1.*

- **Asim Hassan Ali**, Mashaan, N.S., M.R. Karim, 2012. The use of waste tyre rubber as additive for asphalt binder. *INTERNATIONAL CONFERENCE ON INNOVATION AND TECHNOLOGY FOR SUSTAINABLE BUILT ENVIRONMENT* **on** 16-`17 April 2012. Universiti Teknologi MARA (Perak), *ISBN 978-967-5741-06-7*.

- Mashaan, N.S., **Asim Hassan Ali**, M.R. Karim, 2012. Improving asphalt resistance to aging by using waste tyre rubber. *INTERNATIONAL CONFERENCE ON INNOVATION AND TECHNOLOGY FOR SUSTAINABLE BUILT ENVIRONMENT* on 16-`17 April 2012. Universiti Teknologi MARA (Perak), *ISBN 978-967-5741-06-7.*