

**USE OF WASTE TYRE RUBBER IN ENHANCING THE
MECHANICAL PROPERTIES OF STONE MASTIC ASPHALT
MIX**

NUHA S. MASHAAN

**FACULTY OF ENGINEERING
UNIVERSITY OF MALAYA
KUALA LUMPUR**

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NUHA S. MASHAAN

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Name of Candidate: NUHA S. MASHAAN

Registration/Matric No: KHA 120108

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**USE OF WASTE TYRE RUBBER IN ENHANCING THE
MECHANICAL PROPERTIES OF STONE MASTIC ASPHALT MIX**

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ABSTRACT

An immense problem affecting environmental pollution is the unprecedented increase of waste tyre vehicles. Furthermore, rapid economic and industrial growth generates increasing amounts of waste materials includes millions of waste tyre rubber. The lack of proper controlled use of these waste would lead to added disposal costs, increase illegal dumping or inadequate storage and could increase the risk of fire and environmental damage. In this research, attempts to inspire a green technology which is more environmentally friendly that can produce economic value are a major consideration in the utilization of waste tyre rubber in the form of Crumb Rubber Modifier (CRM) in asphalt concrete. In asphalt concrete technology, there are two method to use CRM for asphalt road construction either as a binder modifier (wet process) or as an aggregate (dry process). The dry process has been a far less popular method as compared to the wet process which contributed to inconsistent field performance. The primary aim of this study was to investigate the effect of adding waste tyre rubber as an additive to Stone Mastic Asphalt (SMA) mixture performance properties using the dry process. The research project had three phases. In the first phase, Marshall test was conducted on mixtures containing different percentages of crumb rubber with different bitumen content. The optimum bitumen content was incorporated to design the CRM-reinforced-SMA. In the second phase, the mechanical properties in term of stiffness modulus, fatigue, creep, drain-down, moisture susceptibility and resistance to rutting of CRM-reinforced-SMA were investigated and compared with non-reinforced- SMA. Finally, in last phase, an assessment and evaluation of the relationship between the stiffness and dynamic creep with respect to rutting deformation and fatigue life of SMA reinforced with crumb rubber. Additionally, predicting asphalt pavement performance is an important matter which

can save cost and energy. To ensure an accurate estimation of performance of the mixtures, new soft computing techniques had been used. The results showed that the resilient modulus of CRM-reinforced-SMA samples were obviously higher in comparison with that of unmodified samples. The resistance of waste tyre 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. The results show that the addition of CRM into the mixture has an obvious significant effect on the performance properties of SMA which could improve the mixture's resistance against permanent deformation. Further, higher correlation coefficient was obtained between the rut depth and permanent strain, and, also higher correlation coefficient between fatigue and resilient modulus. Soft computing methodologies had shown a very good learning and prediction capabilities and indicated the most significant effect on stiffness performance.

ABSTRAK

Satu masalah besar yang memberi kesan kepada pencemaran alam sekitar adalah peningkatan mendadak sisa tayar kenderaan. Tambahan pula, pertumbuhan pesat ekonomi dan perindustrian telah menjana peningkatan jumlah bahan buangan termasuklah jutaan sisa tayar getah. Kekurangan pengawalan yang betul ke atas penggunaan sisa ini akan menyebabkan pertambahan kos pelupusan, peningkatan lambakan haram atau tempat simpanan yang tidak mencukupi dan boleh meningkatkan risiko kebakaran dan kerosakan alam sekitar. Dalam kajian ini, percubaan untuk berinspirasi melalui teknologi hijau yang lebih mesra alam boleh menghasilkan nilai ekonomi adalah pertimbangan utama dalam penggunaan sisa tayar getah. Untuk kajian ini, sisa tayar getah yang digunakan adalah pengubahsuaian remah getah (CRM) dalam konkrit asphalt. Dalam teknologi asphalt konkrit, terdapat dua kaedah untuk menggunakan CRM bagi pembinaan jalan raya asphalt sama ada sebagai pengubahsuaian pengikat (proses basah) atau sebagai agregat (proses kering). Kaedah proses kering telah menjadi kurang popular jika dibandingkan dengan proses basah yang mana menyumbang kepada prestasi tapak yang tidak konsisten. Tujuan utama kajian ini adalah untuk mengkaji kesan penambahan sisa tayar getah sebagai bahan tambahan kepada sifat-sifat prestasi campuran asphalt batu mastik (SMA) menggunakan proses kering. Projek penyelidikan mempunyai tiga fasa. Dalam fasa pertama, ujian Marshall tidak langsung telah dijalankan ke atas campuran yang mengandungi peratusan remah getah yang berbeza dengan kandungan bitumen yang berbeza. Kandungan bitumen optimum telah diperolehi untuk mereka bentuk CRM bertetulang - SMA. Dalam fasa kedua, sifat-sifat mekanikal dari segi modulus kekakuan, kelesuan, rayapan , aliran ke bawah, kecenderungan kelembapan dan rintangan ke atas aluran CRM bertetulang - SMA telah disiasat dan dibandingkan dengan SMA tidak bertetulang. Akhir sekali, penilaian dan penaksiran

hubungan antara kekakuan dan rayapan dinamik berkaitan aliran ubah bentuk dan hayat lesu SMA yang diperkukuhkan dengan remah getah. Di samping itu, sebahagian dari fasa ketiga, meramal prestasi turapan asphalt adalah perkara penting yang boleh menjimatkan kos dan tenaga. Bagi memastikan anggaran yang tepat untuk prestasi campuran, teknik kiraan lembut yang baru telah digunakan. Hasil kajian menunjukkan bahawa modulus yang daya tahan untuk sampel SMA-CRM-bertetulang adalah jelas lebih tinggi berbanding dengan sampel yang tidak diubahsuai. Rintangan daripada sisa tayar getah dalam menghasilkan tegasan tegangan mendatar melemahkan penghasilan retak menegak dan menghalang keretakan ini daripada meresap di sepanjang diameter sampel asphalt. Ini seterusnya memperbaiki hayat lesu sampel bertetulang. Keputusan menunjukkan bahawa penambahan CRM ke dalam campuran telah secara signifikan mempengaruhi prestasi sifat-sifat SMA yang mana boleh meningkatkan rintangan campuran terhadap ubah suai bentuk kekal. Di samping itu, pekali korelasi yang lebih tinggi telah diperolehi antara kedalaman lubang dan terikan kekal. Selain itu, pekali korelasi yang lebih tinggi antara kelesuan dan modulus daya tahan juga telah diperolehi. Kaedah pengkomputeran lembut telah menunjukkan pembelajaran dan ramalan keupayaan yang sangat baik. Kaedah ini juga menunjukkan kesan signifikan yang ketara ke atas prestasi kekakuan.

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TABLE OF CONTENTS

ORIGINAL LITERARY WORK DECLARATION	ii	
ABSTRACT	iii	
ABSTRAK	v	
ACKNOWLEDGEMENT	vii	
TABLE OF CONTENTS	viii	
LIST OF FIGURES	xiv	
LIST OF TABLES	xvii	
LIST OF ABBREVIATIONS AND SYMBOLS	xviii	
CHAPTER 1	INTRODUCTION	1
1.1	Introduction.....	1
1.2	Problem Statement.....	3
1.3	Study Aim and Objectives	5
1.4	Scope of Study	7
1.5	Organisation of the Dissertation.....	8
CHAPTER 2	LITERATURE REVIEW	9
2.1	Introduction.....	9
2.1.1	Background: Asphalt Pavement.....	9
2.3	Historical Experiment of Using CRM in Pavement.....	11

2.3	Key Factors of Asphalt Rubber.....	13
2.3.1	Asphalt properties.....	13
2.3.2	Asphalt chemical components.....	15
2.3.3	Asphalt polarity and morphology.....	16
2.4	Bituminous Modification and Reinforcement.....	17
2.5	Waste Tyre Rubber in Road Pavement.....	18
2.5.1	Crumb Rubber Grinding Process.....	21
2.5.2	Crumb Rubber Properties.....	22
2.5.3	Crumb Rubber Constitutions and Concentration.....	22
2.6	Mechanism of Bitumen -Rubber Interaction	24
2.6.1	Bitumen –Rubber Interaction Process Variables	25
2.6.2	Elasticity of Tyre Rubber.....	26
2.7	Rheological and Physical Characteristics of Asphalt Rubber.....	27
2.7.1	Viscous-Elastic Behaviour (Dynamic Shear).....	27
2.7.2	Viscosity Property (Flow Resistance).....	31
2.7.3	Physical Behaviour and Stiffness Performance.....	33
2.8	Durability and Aging of Asphalt Rubber.....	35
2.9	Correlation Between Rheological and Stiffness Properties	37
2.10	Fatigue Resistance	38
2.11	Rutting Resistance	41
2.12	Summary.....	43
 CHAPTER 3		
	METHODOLOGY	45
3.1	Introduction	45
3.2	Materials.....	47

3.2.1	Bitumen , CRM and Aggregate.....	47
3.2.2	Filler selection.....	48
3.3	Physical Properties of Aggregate	49
3.3.1	Aggregate Impact Value Test	49
3.3.2	Los Angeles Abrasion Test	50
3.3.3	Polished Stone Value	50
3.3.4	Flakiness and Elongation Index Test.....	51
3.3.5	Aggregate Crushing Value (ACV).....	51
3.4	Physical properties of Bitumen	52
3.4.1	Softening Point Test (Ring and Ball)	52
3.4.2	Penetration Test (ASTM D5- 97)	52
3.4.3	Brookfield Viscosity (ASTM D 4402).....	53
3.4.4	Flash and Fire Point Test.....	53
3.4.5	Ductility Test (STM D113- 99).....	54
3.4.6	Elastic Recovery Test (ASTM D 6084- 97).....	54
3.4.7	Rolling Thin Film Oven Test (RTFOT) (ASTM D 2872).....	55
3.4.8	Pressure Ageing Vessel Test (PAV) (ASTM D 6521)	55
3.5	Dynamic Shear Rheometer (DSR) (ASTM D-4 proposal P246).....	55
3.6	Marshall Mix Design	56
3.6.1	Marshall Samples Preparation	57
3.6.2	Theoretical Maximum Density.....	60
3.6.3	Void in Mix	60
3.6.4	Marshall Stability and Flow	60
3.6.5	Optimum Binder Content	61

3.7 SMA Mix Performance Test.....	62
3.7.1 Indirect Tensile Test (IDT).....	62
3.7.2 Dynamic Creep Test (AAMAS 338).....	63
3.7.3 Wheel Tracking Test.....	63
3.7.4 Moisture Induced Damage Test.....	64
3.7.5 Binder Drain Down Test.....	64
3.7.6 Indirect Tensile Fatigue Test	65
3.8 Adaptive Neuro-Fuzzy Application.....	66
3.8.1 Evaluation of Model Performances.....	68
3.9 Summery	69
CHAPTER 4 RESULTS AND DISCUSSION	70
4.1 Introduction	70
4.2 Materials Test Results.....	71
4.2.1 Bitumen Test Results.....	71
4.2.2 Aggregate Test Results.....	71
4.3 Marshall Test Results.....	72
4.3.1 Marshaall Sability.....	72
4.3.2 Marshall Flow.....	74
4.3.3 Density of Compacted Mix (CDM).....	76
4.3.4 Voids in the Mix (VIM).....	77
4.3.5 Analysis of Variances (ANOVA) Test of Marshall Test.....	78
4.3.6 Marshall Results and ANFIS.....	79
4.4 Optimum Binder Content (OBC) Results.....	81

4.5 Performance Test Results	82
4.5.1 Indirect Tensile Test Results	82
4.5.2 Indirect Tensile Fatigue Test	86
4.5.3 Dynamic Creep Test Results.....	91
4.5.4 Wheel Tracking Test Results.....	92
4.5.6 Drain Down Results	94
4.5.7 Moisture Susceptibility Test	95
4.6 Relationship Between Rutting, IDT and Creep.....	96
4.7 Relationship between Fatigue, Stiffness and Creep	99
4.8 ANFIS Model Analysis.....	102
4.8.1 Resilient Modulus	102
4.8.2 Creep Test.....	104
4.8.3 Fatigue Test	107
4.9 Summary.....	108
CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS	109
5.1 Conclusion	109
5.2 Recommendations.....	111
REFERENCES	112
APPENDIX A	122

LIST OF FIGURES**PAGE**

Figure 2.1: Representative structures of asphalt fractions.....	16
Figure 2.2 : General behaviour of elastic, viscous and viscoelastic material under constant stress loading.....	31
Figure 2.3: Consolidations Rutting of Flexible Pavement.....	42
Figure 2.4: Instability Rutting of Flexible Pavement.....	43
Figure 3.1: Flow chart of Laboratory testing program.....	46
Figure 3.2: SMA 20 aggregate gradation.....	47
Figure 3.3: ANFIS structure with two inputs.....	66
Figure 4.1 : CRM content and Marshall Stability relationships.....	73
Figure 4.2 : CRM content and Marshall Flow results	75
Figure 4.3 : CDM results using different CRM content.....	76
Figure 4.4 : VIM results using different CRM content.....	77
Figure 4.5: ANFIS model used to predict values of Marshall test	81
Figure 4.6: Resilient modulus results using various CRM content.....	84
Figure 4.7 : Effect of different temperature on Stiffness modulus.....	84
Figure 4.8: Fatigue life and Strain at 2000 N stress.....	89
Figure 4.9: Fatigue life and. Strain at 2500 N stress.....	90
Figure 4.10: Fatigue life and Strain at 3000 N stress.....	90
Figure 4.11: dynamic creep for various CRM content.....	92
Figure 4.12: Rut depth plot for various CRM content.....	94
Figure 4.13 : Drain down test results.....	95
Figure 4.14: moisture susceptibility results.....	96
Figure 4.15: TSR results vs. CRM content	96
Figure 4.16: Relationship between rut depth and dynamic stiffness.....	98
Figure 4.17: Relationship between rut depth and creep permanent strain.....	98

Figure 4.18 : Fatigue and stiffness and creep at 2000 N stress.....	100
Figure 4.19: Fatigue & stiffness and creep at 2500 N stresses	101
Figure 4.20: Fatigue & stiffness and creep at 3000 N stresses	102
Figure 4.21: ANFS decision service of resilient modulus results.....	103
Figure 4.22: ANFIS model used to predict values of stiffness modulus	104
Figure 4.23: ANFIS decision surface for creep estimation.....	106
Figure 4.24 : ANFIS model used to predict values of creep test.....	106
Figure 4.25 : ANFS decision services for fatigue life.....	108
Figure 4.26 : ANFIS model using predicted values of fatigue life.....	108

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LISTOF TABLES**PAGE**

Table 3.1: The physical properties of Crumb rubber (CRM).....	48
Table 3.2 : Chemical Components of CRM (Mahrez, 1999)	48
Table 4.1: Properties of Base Binder Grade 80/100 Penetration.....	71
Table 4.2 : Physical properties of the crushed aggregate.....	72
Table 4.3 : Performance statistics of the ANFIS model in Marshall test estimation.....	80
Table 4.4: The OBCs for the mixes with different CRM content	82
Table 4.5: Fatigue results and fatigue prediction equations.....	89
Table 4.6: Regression coefficient between fatigue strain and creep strain and stiffness using SPSS analysis.....	100
Table 4.7: Statistical parameters for data sets.....	103
Table 4.8: Performance statistics of the ANFIS model in stiffness modulus estimation.....	104
Table 4.9: Performance statistics of the ANFIS model estimation.....	105
Table 4.10: Performance statistics of the ANFIS model in fatigue life estimation.....	107

LIST OF ABBREVIATIONS AND SYMBOLS

AC	: Asphalt Concrete
ASTM	: American Society of Testing Materials
AMWFA	: Artificial Marble Waste Fine Aggregate
CRM	: Crumb Rubber
PET	: Poly Ethylene Terephthalate
CTS	: 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
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

1.1 Introduction

Waste tyres consider a main part of the world's solid management issue. Currently, the volume of polymeric waste such as waste tyre rubber has witnessed an unprecedented increase. It has been reported by Torgal *et al.* 2012, that about 1000 million tyres reach the end of their useful lives every year and 5000 million more are expected to be discarded in a regular basis by the year 2030. In Malaysia, production of waste tyres is about 10 million pieces per annum, and it is estimated to be 20 million unites of tyres by 2020, and unfortunately they are being disposed in an environmentally unfriendly manner (Ali *et al.*, 2013; Mashaan, 2012; Ibrahim *et al.*, 2009). Generally, tyre is large with open space; this space is a problem for many reasons. First, gases like methane can gather in the space, provoking a dangerous fire hazard when tyres are in landfill. Moreover, in case of fire, the tyre is very difficult to extinguish and release toxic smoke and chemicals. Second, stagnant water that might collect in the space, providing a breeding ground for insects such as dengue mosquitoes, which is commonly known in Malaysia, that carry their own dangers in the form of fatal disease presented in dengue fever (Mashaan, 2012).

According to the recycling global market there are only two options have shown the possibility to use high number of waste tyres, (i) fuel for combustion and (ii) Crumb Rubber Modified (CRM) material for asphalt. First option is not an ideal environmental solution. The only remaining option is using waste tyre rubber as CRM material for asphalt paving (Mashaan, 2012). However, in Malaysia the use of waste tyre rubber in asphalt pavement as additive is not sufficiently significant, which is believed to be due to the insufficient number of studies on the evaluation of the potential waste tyre.

There are three different processes involved to produce bitumen- rubber that are; the wet process, dry process and terminal blending (Takallou and Takallou, 2003). In the wet process CRM binders are prepared when finely ground crumb rubber is mixed with bitumen at high temperatures before mixing with the aggregate. During the interaction process of wet process, the rubber particles swell in the bitumen by absorbing a percentage of the lighter fraction of the bitumen forming a viscous gel leading to physical and compositional changes. As for the dry process, granulated crumb rubber is used as a replacement for a small portion of the fine aggregate (typically 1-3 percent by mass of the total aggregate in the mixture). The rubber particles are blended with the aggregate prior to the addition of the bitumen (Hamed, 2010).

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). 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 & Bujang, 2006). Due to the nature of SMA mixes (gap-graded) and the relatively large proportion of asphalt content, stabilisation is required to inhibit drain down 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 (Mashaan, 2012).

However, the concept of SMA was introduced to Malaysia road authorities since 1990's. Several studies and projects were launched for the objective of studying the durability and stability of SMA. However, in Malaysia the acceptability of SMA is less enthusiasm among the road authorities. As a result Malaysia has yet to produce its own standard specification for the design mix of SMA (Mashaan *et al.*, 2013b).

1.2 Problem Statement

The high frequency of traffic loads can result in structural damage to asphalt pavement layers in the form of cracking, and rutting along the wheel tracks, which, in turn cause, stripping and other kinds of road surface deterioration. This, would lead the countries, especially in hot and tropical climates, such as Malaysia, to spend millions of dollars on rehabilitation and maintenance (Kamaluddin, 2008). 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 55⁰C at the surface to 25⁰C 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). To minimise the distress of asphalt like rutting and fatigue, asphalt needs to be modified with selected polymer. Primary polymer has the potential to produce asphalt that can resist rutting and fatigue, however, primary polymer is expensive. Thus, using waste tyre rubber is a valid alternate and cheap. It is consider as sustainable technology "greening asphalt" that might transform wasted residue into a new asphalt mixture substantially resistance to pavement's deformation. Thus, utilising crumb rubber obtained from scrap automobile tyre in SMA, not only cost reduction but also less ecological impact to keep the environmental clean and to achieve better natural resources balance (Mashaan, 2012; Mashaan *et al.*, 2014b).

As mentioned before, the wet process has the significant role in improving the binder properties although it needs special equipment for blending bitumen and rubber. Further, the dry process is a less popular method because of the high costs, construction difficulties and poor reproducibility (Amirkhanian, 2001; Fager, 2001; Hunt, 2002). On the other hand however, the dry process has the ability to use larger numbers of recycled crumb rubber contrasted to the wet process leading to better environmental benefits. Moreover, the dry process is logistically easier than the wet process and, thus, the dry process is significantly available to a larger market. In Malaysia, however, research into the dry process is limited and it's worthy to notice that it mostly had focused on dense and porous mixture (Ibrahim *et al.*, 2009; Katman *et al.*, 2005a; Katman *et al.*, 2005b), whereas negligible attention of (SMA), which is highly recognised of resistance to rutting.

To sum up, this current research focused on using CRM-SMA-dry process as an environmentally-friendly method for producing asphalt mixture wearing course with high resistance to structural failure in Malaysia. Laboratory studies were carried out to understand the mechanical and engineering mixture properties. Whether the results of CRM-dry mix show enhancing in the mechanical properties of the SMA mixture, employment of such waste material will be very remuneration. Furthermore, this would assist work to decrease of waste scrap tyres and save the environment.

1.3 Study Aim and Objectives

Roads and highways are considered as one of the most important elements of infrastructure and they play an essential role in our daily lives (Mashaan *et al.*, 2012a). The design of asphalt mixes involves the accurate selection and proportioning of the component materials in a way that the final product will have the optimal properties. However, the main objective for the design of asphalt paving mixes is obtaining an economical mixture of bitumen and gradation yielding a mixture with the better performance properties.

The aim of this study is to investigate the way in which waste tyre rubber modifies the mechanical performance of the SMA mixture following the dry process. According to Malaysia road specification, “Bitumen wearing course shall meet both structural (resistance to stress and strain imposed by traffic load) and functional performance requirement; the later include adequate durability (resistance to disintegrating effects of climate), good frictional characteristics and smoothness” (JKR, 2013, p.3).

An extensive laboratory investigation on CRM mixtures, designed by modifying a conventional Malaysia Stone Mastic Asphalt (SMA), to study the mechanical properties in terms of stiffness modulus, dynamic creep, fatigue life and resistance to permanent deformation. Therefore, to achieve the main aim of the study, the following objectives were stated:

- 1- To better understand the influence of CRM modifiers on Marshall properties, and resilient modulus (stiffness) properties of reinforced SMA in according to Malaysia road specification.

- 2- To address and determine the significant effect of using Malaysian materials of CRM-reinforced-SMA, using different stress level in terms of increasing pavement resistance to fatigue.
- 3- To evaluate the significant use of dry mix and determine to which extent pavement distresses could be controlled or prevented in term rutting performance of SMA using the dry mix.
- 4- To evaluate the relationship between the laboratory test parameters of indirect tensile test and repeated load dynamic test with respect to the rutting performance and fatigue life of SMA reinforced with CRM.
- 5- To stimulate and estimate the mechanical properties of CRM-SMA dry with adaptive neuro-fuzzy inference system (ANFIS). The process of estimating the accuracy and prediction model, which simulates the stiffness, rutting and fatigue.

1.4 Scope of Study

The study investigates the impact of the utilisation of waste tyre rubber, commonly known as CRM as an additive in SMA using the dry process. The CRM is mixed with the aggregate before adding the bitumen and the mixtures are produced at 155°C to 160°C.

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) and Malaysian Association for Road Work (JKR (Jabatan Kerja Raya)) to check material and mixture parameters and ensure achievement of optimal and satisfactory long- term mixture performance.

The fundamental quality tests for aggregate and bitumen, Marshall test, resilient modulus test, wheel tracking test, indirect tensile fatigue test, drain down test, moisture induced damage test and dynamic creep test were carried out on the prepared samples. The experiments were conducted at the centre of transportation research laboratory at Civil Engineering Department, Faculty of Engineering, University of Malaya, Malaysia.

1.5 Thesis Organisation

- 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 SMA is presented and illustrated. It 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 SMA mixes.
- Chapter 4: This chapter presents the outcomes and engineering properties of the CRM- SMA mixes achieved in this current study. Discussion and analysis of the obtained outcomes and findings and their correlation with the collected data forms the major portion of chapter four.
- 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

An immense problem affecting environmental pollution is the increase in waste vehicles tyres. In an attempt to decrease the magnitude of this issue, Crumb Rubber Modifier (CRM) obtained from waste tyre rubber has gained interest in road asphalt reinforcement (Mashaan & Karim, 2013a). Roadways are an integral aspect of transportation infrastructure. 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 (Hamed, 2010). Thus, the use of crumb rubber in the reinforcement of asphalt is considered as a smart solution for sustainable development by re-using waste materials (Mashaan, 2012; Mashaan *et al.*, 2012a). In this chapter, a critical review on the use of crumb rubber in reinforcement of asphalt pavement is presented and discussed. It also include a review on the effects of CRM on the stiffness, rutting and fatigue resistance of road pavement construction.

2.1.1 Background: Asphalt pavement

The content of this section is part of published review paper by Mashaan *et al.* (2014a). Roadways are an integral aspect of transportation infrastructure. Road construction engineers must consider the primary user's requirements of safety as well as the economy. To achieve this goal, designers should take into account three fundamental requirements which include; environmental factors, traffic flow and asphalt mixtures materials (Mashaan *et al.*, 2014a). In Asphalt Concrete (AC), 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 asphalt binder (bitumen) and asphalt mixture properties. 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 asphalt, 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 asphalt such as crumb rubber” (Mashaan, 2012). 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 asphalt overlays to provide an adequate tensile strength to the asphalt 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 already occurred in the pavement, while the second application is used as measure to prevent the existence of such failure. Modification/reinforcement of asphalt binder is possible during different stages of its usage, either in between binder production and mix processes or before paving mix production (Richard & Bent, 2004) (Mashaan et al., 2014a). According to Larsen *et al.* (1988) the bitumen modification provides binders with:

- Sufficient increase in consistency at the highest temperature in pavements to prevent plastic deformation.
- An increase in flexibility and elasticity of binders at low temperature to avoid crack deformations and loss of chippings.
- An improvement of adhesion to the bitumen into aggregates.

- Improved homogeneity, high thermo stability and aging resistance helps reduce the hardening and initial aging of the binders during mixing and construction.

Worldwide, there are many additives used as reinforcing material into the asphalt mixes, among these additives used is the CRM (Mashaan, 2012; Mahrez, 2008). In this chapter, asphalt pavement design criteria displayed followed by a considerable review on the use of crumb rubber in asphalt pavement reinforcement will be presented and discussed. It also includes a review on the effects of CRM on the stiffness, rutting and fatigue resistance of road pavement. In order to understand the asphalt- rubber reinforcement technology, asphalt properties and crumb rubber characterisers will be illustrated.

2.2 Historical Experiment of Using CRM in Pavement

In the 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. In 1923, natural and synthetic rubber modifications in asphalt were further improved (Mashaan *et al.*, 2012a; Isacsson & Lu, 1999; Yildirim, 2007). According to Yildirim (2007) the development of asphalt-rubber materials being used as joint sealers, patches and membranes began in the late 1930s. The first attempt to modify asphalt binders by adding rubber was made in 1898 by Gaudenberg, who patented a process for manufacturing asphalt- rubber. France was then given credit for installing the first road with a crumb rubber modified asphalt surfacing material (Mahrez, 1999). In 1950, the use of scrap tyre in asphalt was reported (Hanson *et al.*, 1994). In the early 1960s, Charles Mc Donald was working as head Material Engineer for the city of Phoenix, Arizona, He found that after completing the mixing of crumb rubber with the virgin asphalt cement 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 asphalt contents in pavement mixes (Huffman, 1980; Mashaan et al., 2012a). 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. Asphalt-rubber using the wet process was first applied in Alaska in 1988 (Raad & Saboundjian, 1998).

Around 1983 in the Republic of South Africa, asphalt-rubber 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 asphalt rubber Stress Absorbing Membrane Interlayer's (SAMI's) and asphalt performed above expectations. The asphalt rubber overlays out-performed the virgin asphalts, under identical conditions, by a large margin. Asphalts - rubber 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, asphalt- rubber mixtures can be built successfully, quality control ought to be maintained for good performance. The 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 & 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 asphalt rubber (Huang *et al.*, 2007).

The benefits of using crumb rubber modified bitumen are lower susceptibility to varying temperature on a daily basis, more resistance to deformation at higher pavement temperature, proved age resistance properties, 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 tyre rubber can improve the bitumen performance properties (Brown *et al.*, 1997; Maupin 1996; Charania *et al.*, 1991; Adhikari *et al.*, 2000).

In Malaysia, the use of rubber as an additive for road pavement construction supposedly started in the 1940s, but there has not been any official record of such practices. The first recorded trial using rubberised bitumen technology was reported in 1988, the wet mix process was used with the mix of rubber additives in the form of latex into bitumen binder (Sufian & Mustafa, 1997). In 1993, another rubberised road trial using waste gloves and natural rubber latex was carried out in Negeri Sembilan, Malaysia (Samsuri, 1997).

2.3 Key Factors of Asphalt Rubber

2.3.1 Asphalt properties

Asphalt 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 (Mashaan et al., 2014a; Croney & Croney, 1992). It is considered as a thermoplastic visco-elastic 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, it is a complex mixture of a wide variety of molecules: paraffinic, naphthenic and aromatics including heteroatoms (Rozeveld *et al.*, 1997). Most producers use atmospheric or vacuum distillation to refine the asphalt cement. While there is some solvent refining and air blowing utilised, they are clearly of secondary importance (Youtcheff & 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, (compositions are in percentage of weight and represent the +210 °C fraction), are considered appropriate for the manufacture of bitumen (McLean & Kilpatrick, 1997; Read & Whiteoak, 2003). The most commonly used method and probably the oldest method is the atmospheric vacuum distillation of suitable crudes which produce straight-run residual asphalt. The air blowing process is done to give oxidized or semi-blown products, which inherently upgrades of low-grade asphalt. Crude heavy fractions are defined as molecules containing more than 25 carbon atoms (C₂₅), which increases with the boiling point as well as the molecular weight, the density, the viscosity, the refractive index (aromaticity) and the polarity (contents of heteroatoms and metals) (Altgelt & Boduszynski, 1994; Merdrignac & 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 (Altgelt & Boduszynski, 1994; Merdrignac & Espinat, 2007).

2.3.2 Asphalt chemical components

The chemical components of asphalt cement 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 asphalt its adhesive properties. They also act as dispersing agents for the asphaltenes. Resins provide adhesion properties and ductility for the asphalt materials. The viscous-elastic properties of asphalt and its properties as a paving binder are determined by the differing percentages between asphaltenes and maltenes fraction (Navarro *et al.*, 2002; Lewandowski, 1994; Donger *et al.*, 1996). Figure 2.1 shows the representative structures of four generic groups (SARA): saturated, aromatic, resins (which form the maltene fraction) and asphaltenes. This model is based on the colloidal (micellular) model (Masson *et al.*, 2001). The complexity, content of heteroatom, aromatic and increase of molecular weight are in the order of $S < A < R < A$ (saturates < aromatics < resins < asphaltenes) (Claudy *et al.*, 1991). A study by Loeber *et al.*, (1998) illustrated the rheological properties related to asphalt colloidal behaviour. Also, it 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 asphalt cement. Thus, asphalt 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 asphalt with high resins/asphaltenes ratio which leads to high viscous behaviour, higher softening points, and lower penetrations (Mashaan & Karim 2013a).

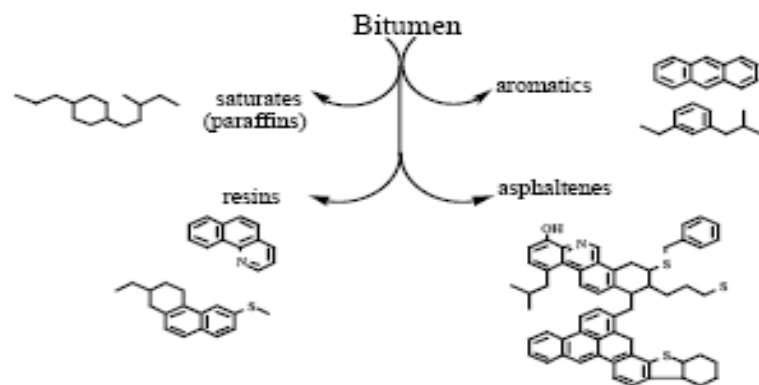


Figure 2.1: Representative structures of asphalt fractions (Masson *et al.*, 2001)

Resins are an intermediate weight, semi-solid fraction formed of aromatic rings with side chains. Also, resins are polar molecules that act as peptizing agents to prevent asphaltenes molecules from coagulating. The lightest molecular weight materials are the non-polar oils. Oils generally have a high proportion of chains as compared to the number of rings. From literature, the resins and oils are referred to collectively as maltenes. In general, asphaltenes produce the bulk of the bitumen while resins contribute to adhesion and ductility and oils influence flow and viscosity properties. According to the microstructure and the colloidal system, asphaltenes are diffused into an oily matrix of maltenes, an encased by a shell of resins whereby its thickness varies with the temperature that is being tested (Navarro *et al.*, 2009). Thus, bitumen composition and temperatures are strongly dependent on the mechanical properties, microstructure of bitumen and on the degree of aromatisation of the maltenes and the concentration of asphaltenes (Navarro *et al.*, 2009; Widyatmoko & Elliott, 2008).

2.3.3 Asphalt polarity and morphology

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

(1991), most of the naturally occurring heteroatoms, nitrogen, sulphur, 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 physic-chemical properties have an obvious significant effect on asphalt and each reflects the nature of the crude oil used to prepare it. Katman (2006) suggested that asphalt cement 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 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.

According to Robertson (1990) the most consistent description, or model, of petroleum asphalt polarity is as follows. Asphalt is a collection of polar and non-polar molecules: (i) the polar molecules are associated strongly to form organised structures and represent a more stable thermodynamic state. (ii) The non-polar model has the ability to dissociate the organised structure, but again there are possible variations from asphalt sources and its viscous behaviours are highly dependent on the temperature. Using current day technology, the morphology of asphalt has been studied in order to verify the asphalt structure.

2.4 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. 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 *et al.*, 2014a).

2.5 Waste Tyre Rubber in Road Pavement

“Crumb rubber obtained from 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, 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)” (Mashaan, 2012).

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. In 1923, natural and synthetic rubber modifications in bitumen were further improved (Mashaan *et al.*, 2012a; Yildirim, 2007). According to Yildirim, (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 Guedmberg, 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 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 (Mashaan & Karim 2013a).

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 (Mashaan, 2012).

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. 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 (Mashaan *et al.*, 2012a; Huang *et al.*, 2007). The benefits of using crumb rubber modified bitumen are as listed below:

- Lower susceptibility to varying temperature on a daily/ seasonal basis.
- More resistance to deformation at higher pavement temperature.
- Improved age resistance properties.
- 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).

2.5.1 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. 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 (Mashaan *et al.*, 2012a; Becker *et al.*, 2001). In the crumb rubber market, there are three main classes based on particle size:

- i. Type 1 or Grade A: 10 mesh coarse crumb rubber;
- ii. Type 2 or Grade B: 14 to 20 mesh crumb rubber;
- iii. 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. 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” (Mashaan & Karim 2014b). The high cost of this process is considered a disadvantage due to the added cost of liquid nitrogen (Mashaan, 2012).

2.5.2 Crumb Rubber Properties

The use of rubber crumbs instead of polymer depends on the desired properties of the modified bitumen for a particular application. However, the choice is also determined to a certain extent by the cost of modification and the availability of the modifier (Mahrez, 1999). The required properties are preferably obtained with minimal cost. The growth of vehicle productions year by year has generated wasted tyres. Due to limitation of disposal area and environmental problem the recycling of these vehicles tyres as industrial wastes has been encouraged, and the production of rubber crumbs from it, has found it suitable for use as modifier into bitumen. Also it offers other benefits such as: using with less complicated blending equipment, and minimal requirement to modify the asphalt. Comparing the use of polymer as a modifier, taking the two fundamental points cited above, the cost using polymer is much higher than using rubber crumbs and its availability is less compared to rubber crumbs. Even though the properties of using polymers may be better but they are comparable to those of rubberised asphalt (Mashaan, 2012).

2.5.3 Crumb Rubber Constitutions and Concentration

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 hot

paving grade. Asphalt rubber 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 concrete mixture, not as part of the asphalt binder), 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). There are two different methods in the use of tyre rubber in asphalt binders; first, is by dissolving crumb rubber in the asphalt as binder modifier. Second, is by substituting a portion of fine aggregates with ground rubber that does not completely react with bitumen (Mashaan *et al.*, 2014a).

According to laboratory binder tests (Mashaan *et al.*, 2011a; Mashaan *et al.*, 2011b; Mashaan & Karim, 2014b) stated that, “it is clear that rubber crumb content played a main role in influencing significantly the performance and rheological properties of rubberised bitumen binders. It could enhance the performance properties of asphalt pavement resistance against deformation during construction and road services”. The increase in rubber crumb content was from 4 - 20% thus indicating a linear increase in softening point, ductility, elastic recovery, viscosity, complex shear modulus and rutting factor. This phenomenon could be explained by the absorption of rubber particles with lighter fraction oil of bitumen, leading to increase in rubber particles during swelling during the blending process (Mashaan & Karim, 2014b). The increase in rubber content by 16% and 20% showed a corresponding increase in Brookfield viscosity value that are higher than SHRP specification limits (3 Pa). These make the two stated percentages unacceptable for field construction during asphalt pavement mixture construction.

Regarding the low temperature performance, an investigation with 18- 22% of rubber content, showed change that was little significance within this range in affecting the tensile and fracture performance of the bitumen compared to varying the binder content between 6-9 % by bitumen weight (Huang *et al.*, 2007). A study by Khalid (2005) found that higher binder content led to longer fatigue life of rubberised bitumen mixture, better resistance to rutting as well as results showing good resistance to fracture and fatigue cracking. Liu *et al.*, (2009) found that content of crumb rubber as the most significant affecting factor followed by crumb rubber type and lastly the size of the particle.

2.6 Mechanism of Bitumen -Rubber Interaction

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. 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 180°C and 8h at 140°C. On the other hand, unvulcanized rubber powder requires merely 10 minutes digestion time at 160°C 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 (Mashaan *et al.*, 2014a)

2.6.1 Bitumen –Rubber Interaction Process Variables

Interaction process variables consist of curing profile (temperature and duration) and mixing shear energy (Mashaan & Karim, 2013a; Jeong *et al.*, 2010; Shen *et al.*, 2009). Katman *et al.*, (2005b) studied the effect of the mixing types on the properties of rubberised asphalt. An ordinary propeller-type mixer and the high speed shear mixer were used. The study indicated that the resultant binder produced using the high-speed shear mixer appears to have slightly superior properties as compared to that produced using the propeller-type mixer. It showed that the viscosity and softening point of the rubberised asphalt produced using the high-speed shear mixer produced higher agitation level and shearing action that can chop the swollen rubber particles within a certain volume of binder. Thus, the absorbent of the lighter oily fraction was increased due to the large amount of the small rubber particles. A study indicated that processing procedure and tyre type play an important role in the determination of rubberised bitumen viscosity. Interaction between crumb rubber and asphalt binders is referred to as a physical interaction where the crumb rubber through diffusion, absorbs the aromatic fraction of the bitumen binders which leads to swelling of the crumb rubber particles (Thodsen *et al.*, 2009). This particle swelling compounded with reduction in the oily fraction of the binder results in increased viscosity in the rubberised bitumen binder. “In general, the bitumen binder and ground tyre rubber are mixed together and blended at elevated temperatures for differing periods of time prior to using them as a paving binder. These two factors work together to evaluate the performance properties of rubberised bitumen binder through blending process of asphalt rubber interaction. This variation in mixing time and temperature results due to the normal activities are related to bitumen paving construction “(Mashaan *et al.*, 2012a).

Nevertheless, the consistency of asphalt rubber can be affected by the time and temperature used to combine the components and thus must be cautiously used for its optimum potential to be achieved. The increase in blending time showed insignificant difference on rubberised asphalt properties in the case of 30 and 60 minutes. Whereas the increase in blending temperature corresponded to the increase in Brookfield viscosity, softening point, ductility, elastic recovery and complex shear modulus (Mashaan *et al.*, 2011a; Mashaan *et al.*, 2011b; Mashaan & Karim 2013a). Several studies (Jeong *et al.*, 2010; Lee *et al.*, 2006; Lee *et al.*, 2008; Xiao *et al.*, 2006) showed that longer reaction time for production of the asphalt rubber apparently caused increased viscosity due to the increased rubber mass through binder absorption. On the other hand, (Mashaan & Karim, 2013a; Shen and Amirkhani, 2005; Moreno *et al.*, 2011; Putman *et al.*, 2005; Paulo & Jorge, 2008) reported that the reaction time has no significant effect on the selection of the optimal binder content. In addition, there was no difference in the molecular size variation between the control binder and the asphalt rubber binders. Also, blending time had insignificant difference on asphalt rubber physical and rheological properties and a rather slight influence on the performance properties of rubberised asphalt.

2.6.2 Elasticity 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 (Beatty, 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 asphalt, 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 insensitive 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 (Mahrez, 1999; Mashaan, 2012).

2.7 Rheological and Physical Characteristics of Asphalt Rubber

2.7.1 Viscous-Elastic Behaviour (Dynamic Shear)

Asphalt binders are referred to as viscoelastic materials because they exhibit combined behaviour (properties) of elastic and viscous material as presented in Figure 2.2 (a) with the removal of the applied stress from the material, there is a complete recovery to the original position. Figure 2.2(b) explains the behaviour of a viscous material in case the strain of the material increases through time under stable stress. Figure 2.2(c) illustrates the behaviour of a viscoelastic material when stable stress increases the strain over a long period of time and when the applied stress is removed; the material loses its ability in attaining its original position resulting in permanent deformation.

According to Van der Poel (1954), generally the stiffness modulus of bitumen binders can be defined by:

$$S(t) = \sigma / \varepsilon(t) \dots\dots\dots \text{Equation (2.1)}$$

Where: $S(t)$ is dependent stiffness modulus (Pa), (t) is loading time (s), σ is the applied constant uniaxial stress (Pa) and $\varepsilon(t)$ refers to uniaxial strain at time t , (m/m). Since asphalt is a visco-elastic material, its rheological properties are very sensitive to temperature as well as to the rate of loading. With respect to temperature, the most frequent problems of road pavement are rutting, fatigue cracking and thermal cracking. Dynamic Shear Rheometer (DSR) was used to measure and determine the rheological properties of the asphalt binder at different stress/temperature sweep and various frequencies. DSR testing included parameters of complex shear modulus (G^*), storage modulus (G'), loss modulus (G'') and phase angle (δ). The formula to calculate the G^* , G' , G'' as well as δ in Equations 2.2 -2.5 respectively is demonstrated below:

$$G^* = (\tau/\gamma) \dots\dots\dots \text{Equation (2.2)}$$

$$G' = \cos(\delta) (\tau/\gamma) \dots\dots\dots \text{Equation (2.3)}$$

$$G'' = \sin(\delta) (\tau/\gamma) \dots\dots\dots \text{Equation (2.4)}$$

$$\delta = G/G'' \dots\dots\dots \text{Equation (2.5)}$$

where G^* is the complex shear modulus, τ is the shear stress, γ is the shear strain, G' is the storage modulus, G'' is the loss modulus, and δ is the phase angle. Navarro *et al.* (2002) studied the rheological characteristics of ground tire rubber-modified asphalt. The experiment was performed in a controlled-stress Haake RS150 rheometer. The study aimed at comparing the viscoelastic behaviour of five ground tyre rubber-

modified with unmodified asphalt and polymer-modified (SBS) asphalt. The study displayed that rubber-modified asphalt improved viscoelastic characteristics and therefore higher viscosity than unmodified binders. Thus, the asphalt rubber is expected to better enhanced resistance to permanent deformation or rutting and low-temperature cracking. The study also found that the viscoelastic properties of rubber-modified asphalt with 9% weight are very similar to SBS-modified bitumen having 3% weight SBS at-10°C, and 7% weight at 75°C.

Mashann and Karim (2013a) investigated the rheological properties of asphalt rubber for various combination factors of crumb rubber content and blending conditions. The DSR test was conducted to evaluate the engineering properties of asphalt binder reinforced with crumb rubber at 76 °C. Specification testing was performed at a test frequency of 10 rad/s which is equivalent to the car speed of 90 km/h. Test specimens of 1 mm thick by 25 mm in diameter were formed between parallel metal plates. The study displays increases in G^* , G' , G'' and decrease in phase angle (δ). Thus, the modified asphalt became less susceptible to deformation after stress removals. The study also presented a considerable relationship between rheological parameters (G^* , G' , G'' and δ) and softening point in terms of predicting physical-mechanical properties regardless of blending conditions (Mashann and Karim (2013a). Natu and Tayebali (1999) observed that the unmodified and crumb rubber modified binders with the same high temperature PG rating do not show similar viscoelastic behaviour over a range of frequencies. It was also concluded that the unmodified and crumb rubber modified mixtures containing binders with the same high temperature PG rating do not show similar viscoelastic behaviour over a range of frequencies. Mixtures containing the same PG rated binders performed similarly if their performance was evaluated at a frequency and temperature at which the binder high temperature PG rating was determined.

The loss tangent ($\tan \delta$) of the binder was not observed to be directly related to the loss tangent of the mixture, since the loss tangent of the mixture was much lower, perhaps due to the aggregate effects, than the loss tangent of the binder. It was also noted that the loss tangent of the mixture increased when the temperature was decreased. A similar observation was also made for the influence of frequency. With increasing frequency, the loss tangent increased to a peak value and then decreased with further increase in frequency. The loss tangent of the binder increased appreciably when the temperature was increased (Mahrez, 1999). Mixture stiffness by itself does not appear to provide a measure for assessing the propensity for rutting in mixtures containing modified binders. Higher dynamic modulus (G^*) are not necessarily associated with lower permanent deformation. With respect to binder type, the dynamic modulus is lower for the mixtures containing modified binders as compared to the mixture containing conventional binder (Mahrez, 1999).

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 $G^*/\sin\delta$). It was concluded that of the parameters considered, for this range of binders, only the SHRP $G^*/\sin\delta$ 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 when the load is removed. The frequency selected for the binder measurements has been found to have a significant impact on the quality of the correlation obtained and should be as possible to the frequency of loading applied to the mix (Mahrez, 1999). At intermediate pavement

service temperatures a reasonable correlation was found between one aspect of mix fatigue performance (ϵ), and the binder loss modulus ($G^* \sin\delta$), again measured under the same temperature and loading as the mix testing. However, above certain binder stiffness, the variation in measured fatigue life was small, due to machine compliance becoming significant at high mix stiffness. It is unlikely that binder rheology alone will be sufficient to accurately predict and explain mix fatigue life.

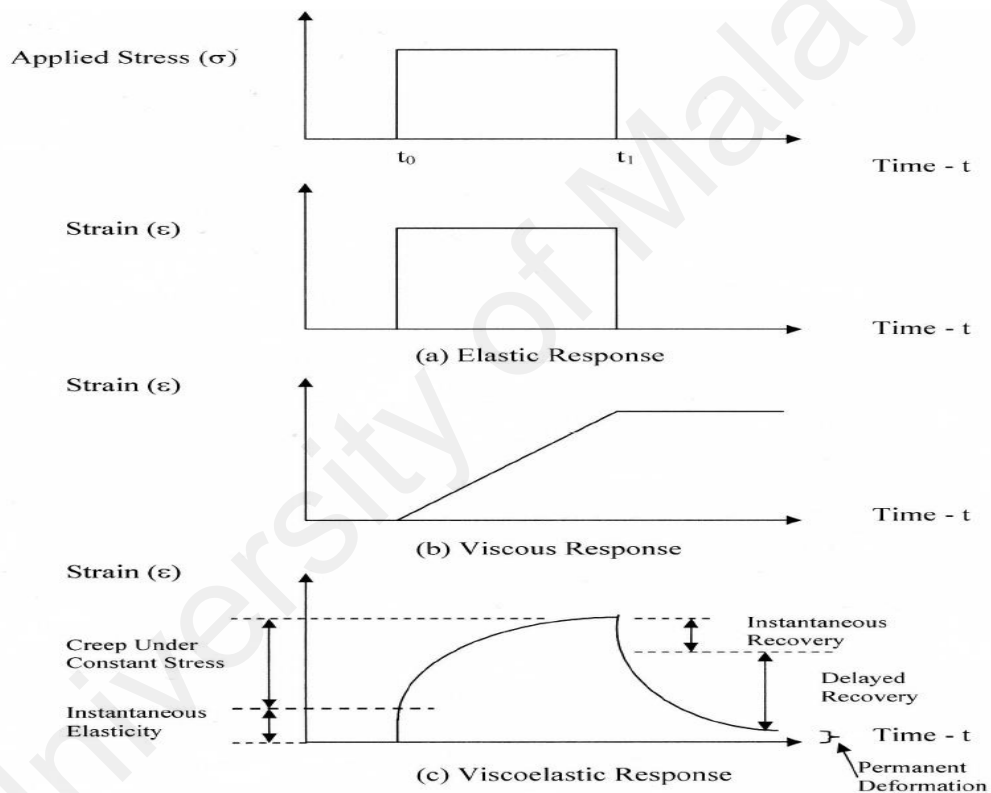


Figure 2.2: General behaviour of elastic, viscous and viscoelastic material under constant stress loading (Van der Poel,1954)

2.7.2 Viscosity Property (Flow Resistance)

“Viscosity refers to the fluid property of the asphalt cement and it is a gauge of flow-resistance. At the application temperature, viscosity greatly influences the potential of the resulting paving mixes. During compaction or mixing, low viscosity has been

observed to resulting in lower stability values and better workability of asphalt mixture” (Mashaan et al., 2012a).

Radhakrishnan *et al.* (1998) used a rotational viscometer to measure the viscosity of the soft asphalt samples while the viscosity of the blown asphalt samples are measured on a capillary rheometer. The tests were conducted to study the flow behaviour on the modification of asphalt with Liquid Natural Rubber (LNR). The findings are; for soft asphalt, the temperature dependence on viscosity is prominent up to 100°C and subsequently marginal. The addition of 20% LNR results the maximum viscosity.

Activation energy of flow of soft bitumen increased, while that of blown asphalt decreased on addition of LNR. Another study (Zaman *et al.*, 1995) found that the viscosity of asphalt cement increases with the addition of rubber, and rubber-modified asphalt-cement samples show a more uniform and higher resistance against loading as the amount of rubber increased. The degrees of shear-thickening and shear-thinning behaviour decreased by increases the amounts of rubber in asphalt cement. The liner dynamic viscosity was increased by increasing the amount of rubber in asphalt cement. A study had mentioned that the vulcanized rubber had a large effect on the viscosity of the asphalt cement. The viscosity, measured at 95°C, increased by a factor of more than 20 when 30% vulcanized rubber was added to the mixture. In contrast, the devulcanized rubber had only a very small effect. The viscosity test also showed that there is no danger of gel formation when rubber is mixed with hot asphalt cement. The use of crumb rubber in bitumen modification leads to an increase in the softening point and viscosity as rubber crumb content increases. Also, it was 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 (Katman, 2006).

2.7.3 Physical Behaviour and Stiffness Performance

A study investigated the properties of asphalt rubber binder prepared by physical blending of asphalt 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 binder (Mashaan *et al.*, 2014a). Another study (Kumar *et al.*, 2009) on penetration change was conducted using asphalt 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 asphalt binders. A researcher concluded from his study, that the elastic recovery of asphalt rubber 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°C (Katman, 2006). “Asphalt 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 asphalt attains a particular degree of softening”(Mashaan *et al.*, 2012a).

Mahrez and Rehan (2003) claimed that there is a consistent relationship between viscosity and softening point at different aging phases of asphalt rubber binder. Also, it is reported that the higher crumb rubber content leads to higher viscosity and softening point. Authors reported that the softening point value increase as crumb tuber content increase in the mix. The increase of rubber content in the mix could be co-related to the increase in the asphaltenes/resins ratio which probably enhanced the stiffening properties, making 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. According to Mashaan *et al.* (2011b), the rubberised asphalt binder was evaluated in terms of binder elasticity and rutting resistance at high temperature. The higher crumb rubber content appears to dramatically increase the elastic recovery and ductility.

According to a study (Beaty, 1992), the ductility test conducted at low temperature was found to be a useful indicator of brittle behaviour of bitumen. Latex contents in the range 3 to 5% were found to result in non-brittle behaviour in the ductility test at 5°C whereas the unmodified bitumen failed by brittle fracture in the same test. Radhakrishnan *et al.* (1998) found that the ductility decreased in the case of soft bitumen with increasing concentration of liquid natural rubber while some improvement was noticed in the case of blown bitumen at 10% loading. The ductility is measured at 27°C and pulled apart at a rate of 50 mm/min. Modified bitumen binders showed a significant enhancement on the elastic recovery, and, in contrast, the ductility decreased with respect to unmodified binders (Martinez *et al.*, 2006).

2.8 Durability and Aging of Asphalt Rubber

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 physic-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 factors might contribute to this hardening of the asphalt cement such as oxidation, volatilisation, polymerisation and thixotropy. This is because asphalt is an organic compound, capable of reacting with oxygen found in the environment. The asphalt composite changes with the reaction of oxidation developing a rather brittle structure. This reaction is referred to as age hardening or oxidative hardening. Volatilisation occurs when the lighter components of the asphalt evaporate. In general, this is related to elevated temperatures that are found firstly during the hot mix asphalt production process. Polymerisation is the means by which resins are assumed to combine into asphaltenes, resulting in an increase in the brittleness of the asphalt along with a tendency toward non-Newtonian behaviour. At the end of the reaction, thixotropy, or an increase in viscosity over time, also contributes to the aging phenomenon in asphalt. However, the most important factors in the aging process of asphalt binder seem to be oxidation and volatilisation. The occurrence of steric hardening and the time-dependent reversible molecular association have affected the binder properties but this is not considered as aging. Steric hardening is only a factor at intermediate temperatures; at high temperatures excess kinetic energy in the system prevents the association and at low temperatures the rate of association is found to be slower due to the binder's high viscosity (Mahrez, 2008).

A research stated that the mechanism by which binder properties may change at low temperature. This mechanism is called physical hardening occurs at temperatures next to or lower than the glass transition temperature and causes significant hardening of the asphalt binder. The rate and magnitude of the hardening phenomena has been observed to increase with decreasing temperatures and is reported to be similar to the phenomena called physical aging on amorphous solids. The physical hardening can be explained using the free volume theory which introduced the relationship between temperature and molecular mobility. The free volume theory includes the molecular mobility dependent on the equivalent volume of molecules present per unit of free space or free volume. Based on the free volume theory, when amorphous material is cooled from a temperature above its glass transition temperature, molecular adjustments and the collapse of free volume rapidly show a drop in temperature. At that temperature, the structural state of the material is frozen-in and deviates from thermal equilibrium due to the continuous drop in kinetic energy. Hence, it has been postulated in order for physical hardening to happen in binders, temperatures must be higher than the glass transition temperature (Ali *et al.*, 2013).

Many durability tests are based on the evaluation of resistance to asphalt hardening. Mahrez and Karim (2003) investigated ageing effects on visco-elastic properties of rubberised asphalt using the DSR. The binders were aged with the Thin Film Oven Test (TFOT), the Rolling Film Oven Test (RFOT) and the Pressure Ageing Vessel (PAV). This research found that ageing influences the rheology of rubberised asphalt. The mechanical properties of aged binder improved by increased complex modulus and decreased phase angle. The aged samples were characterized by higher stiffness and elasticity, due to an increase in the elastic (storage) modulus, G' . The high value of G' is an advantage since it improved further the rutting resistance during service. Natu and

Tayebali (1999) carried out comprehensive research study which evaluated high temperature performance characteristics of unmodified and crumb rubber modified asphalt binders and mixtures. The research showed that the effect of RFTO aging on binder rutting factor was increased at low frequencies and/or high temperatures. The improvement in the rutting factor diminished with an increased in frequency, and at very high frequencies (low temperatures) the rutting factors for unaged and RFTO aged binders were nearly the same. The increase in binder rutting factor of crumb rubber modified asphalt binders at low frequencies suggested that the binder resistance to permanent deformation has improved.

Ali *et al.*, (2013) studied the influence of the physical and rheological properties of aged rubberised asphalt. The results indicates that the use of rubberised binder reduces the aging effect on physical and rheological properties of the modified binder as illustrated through lower aging index of viscosity (AIV), lower aging index of $G^*/\sin \delta$, lower softening point increment (ΔS) less penetration aging ratio (PAR) and an increase in $\tan \delta$ with crumb rubber modifier content increasing, indicating that the crumb rubber might improve the aging resistance of rubberised binder.

2.9 Correlation Between Rheological and Stiffness Properties

As extensive research program conducted (Mahrez, 1999) 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 $G^*/\sin\delta$). It was concluded from the parameters considered, for this range of binders, only the SHRP $G^*/\sin\delta$ 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 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. At intermediate pavement service temperatures a reasonable correlation was found between one aspect of mix fatigue performance (ϵ), and the binder loss modulus ($G^* \sin \delta$), 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 $G^*=300$ MPa at 1000s provides a good indicator of the fracture temperature of the mix (Mahrez, 1999).

2.10 Fatigue Resistance

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).

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 (1995a,b) presented a description of the purpose and scope of the dynamic shear rheometer test. The 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. 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. 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.

The use of crumb rubber modified with bitumen binder seems to enhance the fatigue resistance, as illustrated in a number of studies (Mashaan, 2012; Hamed, 2010; Aflaki & Memarzadeh, 2011). The improved performance of bitumen rubber pavements compared with conventional bitumen pavements has partly resulted from improved rheological properties of the rubberised bitumen binder. 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 (Mashaan *et al.*, 2012a; 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. 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 (Mashaan et al., 2014a).

2.11 Rutting Resistance

There are various laboratory methods for studying distortion or rutting. The TRRL Wheel Tracking Test appears to be the most suitable in simulating the field conditions as closely as possible. The test was conducted for 24 hours in temperature controlled cabinet at 60°C. From the indents made on the slab, the depth of tracking was recorded at the mid-point of its length. After about 6 hours, a steady state of tracking was observed. From the deformation/time curve, the rate of increase in track depth is determined in mm per hour once the steady state is reached (Katman, 2006).

According to Shin *et al.* (1996), addition of crumb rubber and SBR increases the rutting resistance of asphalt paving mixtures. The results from laboratory study showed that the CR-modified and SBR-modified asphalt had higher stiffness at 60°C than the modified mixtures. The modified asphalt mixtures also had higher gyratory shear

strengths and lower rut depths in the Loaded Wheel tests than the unmodified mixtures. Tayfur *et al.* (2007) showed that after the initial densification, the rutting deformation of the bituminous mixture happens due to shear loads which take place occur close to the pavement surface which in fact is the contact area between the tyre and the pavement. These efforts increase without the volume variations in the bituminous mixture. They are the primary mechanisms in the development of rutting during the life span of the pavement design. Increased permanent deformation or rutting has been related to the increase in truck tyre pressures, axle loads and volume of traffic (Mashaan *et al.*, 2012a).

A study (Mahrez, 1999) claimed that the use of rubberised bitumen binder has a significant effect on improving the mixture resistance to rutting deformation. Rutting in flexible pavement can be divided into two types; consolidation rutting which happens with excessive consolidation of the pavement along the wheel path caused by decreased air voids in the asphalt concrete layer as shown in Figure 2.3, or the permanent deformation of the base or subgrade. Instability rutting is due to the asphalt mixture properties and is an occurrence in the range of the top 2 inches of the asphalt concrete layer as shown in Figure 2.4 (Sousa and Weissman, 1994).

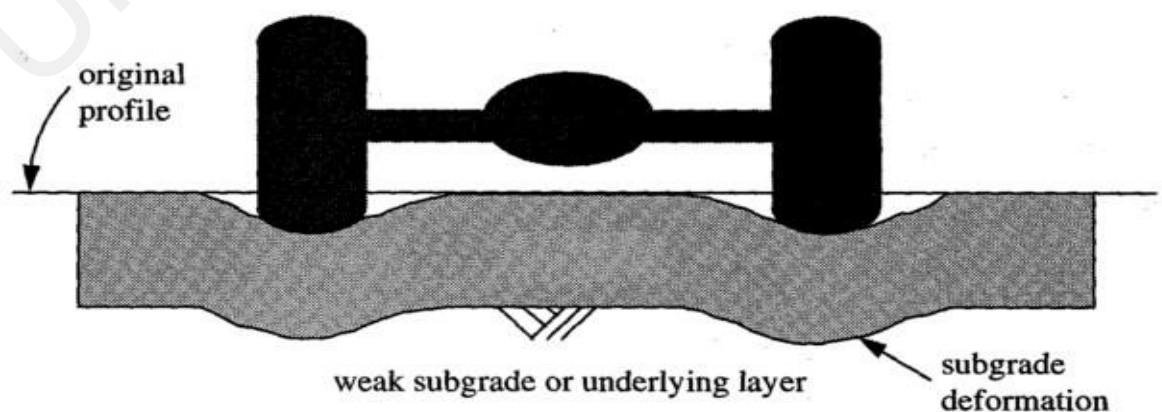


Figure2.3: Consolidations Rutting of Flexible Pavement (Sousa and Weissman, 1994).

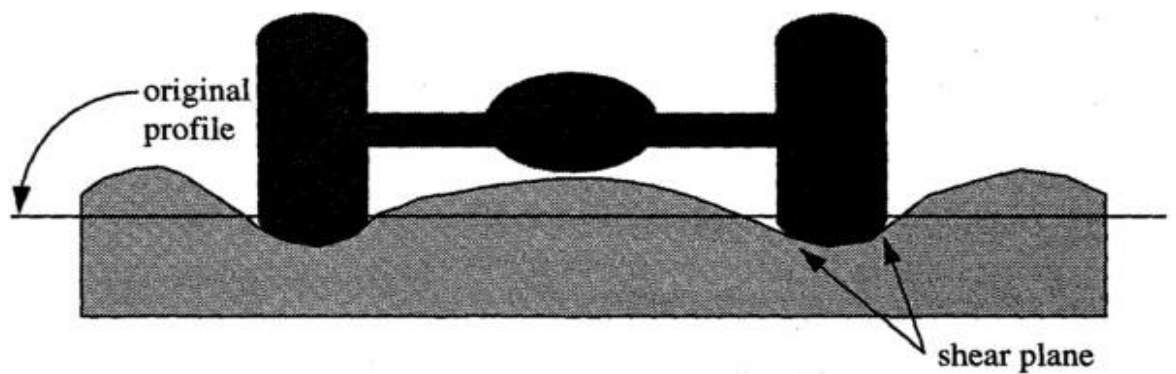


Figure 2.4: Instability Rutting of Flexible Pavement (Sousa and Weissman, 1994).

2.12 Summary

This chapter focused on certain essential factors of asphalt mix as well as waste tyre rubber modified asphalt. The application of primary polymer to asphalt mix has been promoted as good way to improve the performance of asphalt pavement. Using primary polymer contributes to increase the stiffness of the asphalt mix and improvement of its susceptibility to temperature variations and fluctuations. This, in turn, improves the resistance to rutting. However, today, asphalt mix modified with primary polymer is quite expensive for road pavement. Using waste and recycled polymer is one of the best ways to reduce the overall cost of road construction. On the other hand, in recent times a serious problem that leads to environment pollution is the abundance and increase of waste tyre disposal. Large amounts of rubbers are used as tyres for cars and trucks etc.

Despite the long run in service, these tyres are not discarded. Although, rubber as a polymer is a thermosetting material cross linked to processing and moulding, it cannot be softened or remoulded by re-heating unlike other types of thermoplastics polymer which can be softened and reshaped when heated. 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. 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 environment clean and to achieve better balance of natural resources.

University of Malaya

CHAPTER 3: METHODOLOGY

3.1 Introduction

This chapter contains discussion on several experiments and the testing methods followed to evaluate the objectives of the current study. This chapter consists of three main sections. The first section, which discusses the physical properties of materials used in the sample mixtures and related tests. The second section elaborates and deals with the Marshall mix design and also using soft computing method of adaptive neuro-fuzzy inference system (ANFIS) method for SMA modelling and design. The third section explains the methods of performance tests on the SMA samples. The model required for predicting rutting and fatigue properties is illustrated. Figure 3.1 illustrates the flow chart of the methodology used in this study.

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) and Malaysian Association for Road Work (JKR (Jabatan Kerja Raya)) to check material and mixture parameters and ensure achievement of optimal and satisfactory long- term mixture performance.

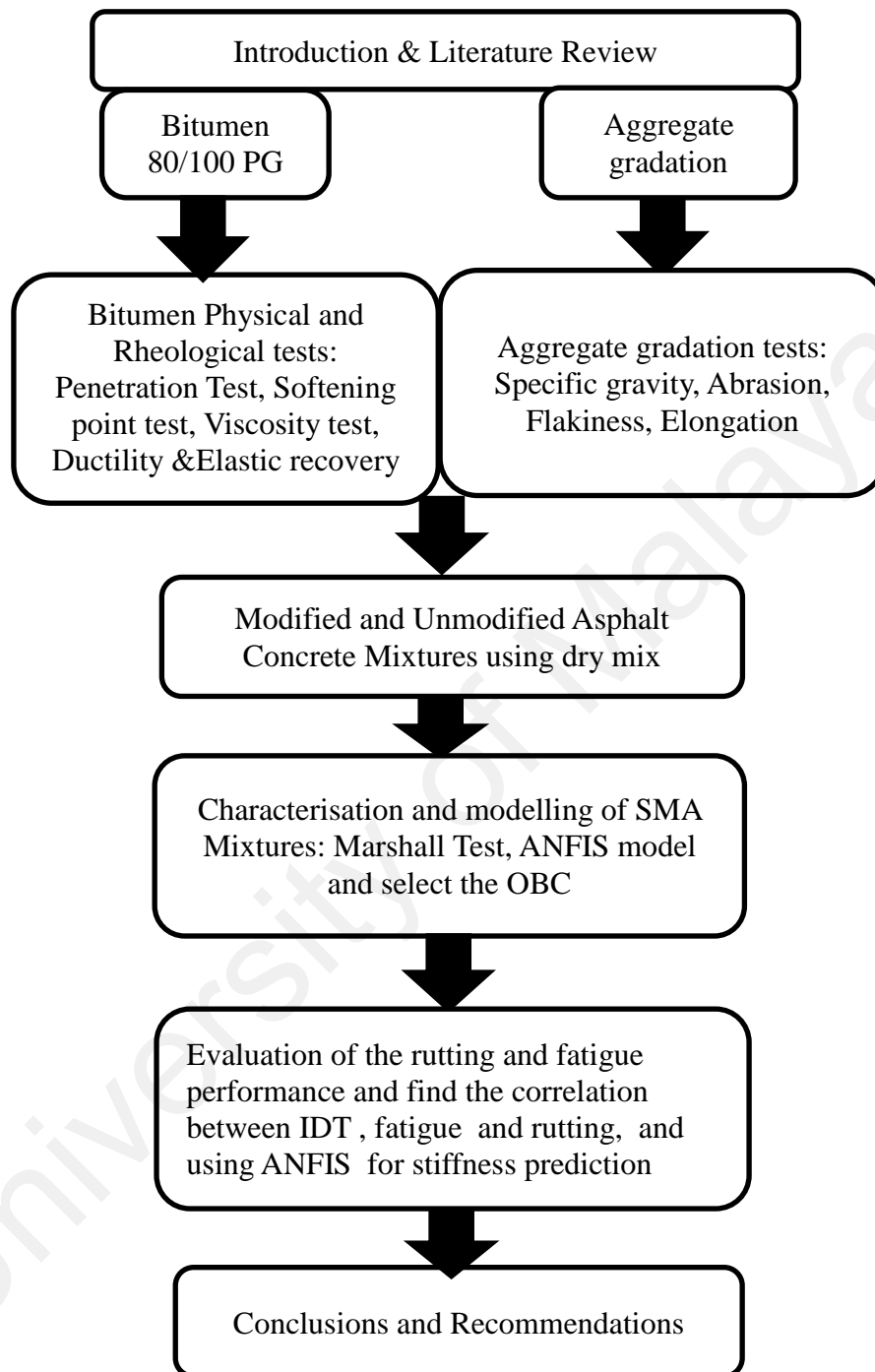


Figure 3.1: Flow chart of laboratory testing program

3.2 Materials

3.2.1 Bitumen, CRM and Aggregate

A bitumen of 80/100 penetration grade was used. The crushed granite aggregates used throughout the study were supplied by the Kajang Granite Quarry Sdn Bhd, a suburb near Kuala Lumpur, the capital city of Malaysia. The aggregate gradation of SMA 20 was selected according to Malaysian Public Works Department (PWD) specification for road works as illustrated in Figure 3.2. In this study, the gradation of crumb rubber # 40 (0.45 mm) was selected. The density of crumb rubber is about 1.15 gm/cm³. The crumb rubber modifier (CRM) produced by mechanical shredding at ambient temperature was obtained from Rubberplas Sdn. Bhd. (Malaysian supplier).

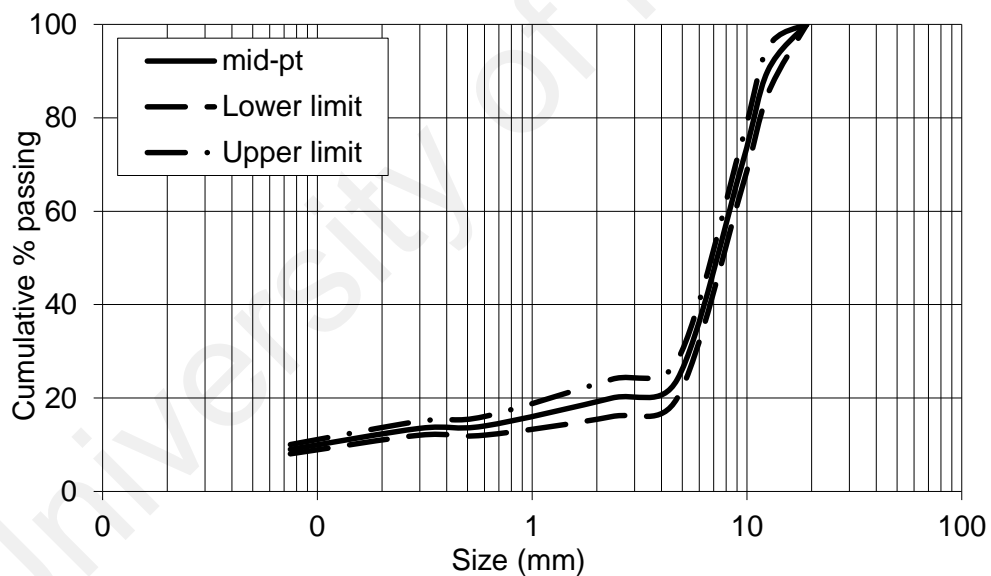


Figure 3.2: SMA 20 aggregate gradation

The elastomeric compositions for crumb rubber are natural rubber 30%, styrene-butadiene-rubber (SBR) 40% and butadiene rubber 30%. The physical properties for crumb rubber are presented in Table 3.1, and CRM chemical components are illustrated in Table 3.2.

Table 3.1: The physical properties of Crumb rubber (CRM)

Physical properties	Unite
Density	1319 kg/m ³
Young's modulus (E)	2600-2900 MPa
Tensile strength (σ_t)	40-70 MPa
Elongation @ break	25-50%
Melting point	200 °C

Table 3.2 : Chemical Components of CRM (Mahrez, 1999)

Chemical components	Test result
Acetone extract (%)	23.1
Rubber hydrocarbon (%)	46.6
Carbon black content (%)	25.08
Natural rubber content (%)	43.85
Ash content (%)	5.2
Particle size (μ)	425

3.2.2 Filler Selection

Filler can be considered to be acting in two ways; firstly it can be considered to modify the grading of the fine aggregate, thereby producing a mix with more points of contact between the aggregate particles; secondly, and probably a better way of regarding the filler is to considered that the filler and bitumen together form a binder which both lubricates and binds the fine aggregate to form the mortar. The properties of the mortar will depend on the nature of the fine aggregate and on the amount and

viscosity of the binder. Ordinary Portland Cement (OPC) was added to the combined aggregate for bituminous mixture to serve as an adhesion and anti-stripping agent. This study used 8% filler, 2% of which OPC and 6% consisted of stone powder.

3.3 Physical Properties of Aggregate

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, 5-7% binder, and air voids content between 2-4% (Mashann *et al.*, 2013b). Its strength and resistance to rutting depends mostly on the type of aggregate used in the mixture, which needs to consist of 100% crushed aggregate of cubical shape. In addition, the role the mortar binder plays is very significant in building strong binding between the various materials used in the asphalt mixture.

“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” (Mashaan *et al.*, 2012a). Aggregate interlock and particle friction are maximised and gives the structure its stability and strength. Thus, the quality and accepted limit of the physical properties of the aggregate should be taken into consideration in SMA design.

3.3.1 Aggregate Impact Value Test

The strength value of aggregate is an essential property of aggregate that provides the relative aggregate strength level making it capable of resisting the impact of traffic loadings. The laboratory strength value of aggregate was determined through the aggregate impact test. Its details and specification has been elaborated in British Standard (BS 812: Part 3, 1975). In this test, the aggregate is put into the mould and

compacted 15 blow with the heavy metal hammer that weight 14 kg falling from a height of 38cm.

3.3.2 Los Angeles Abrasion Test

The Los Angeles abrasion test apparatus was used to characteristic toughness and abrasion resistance. The aggregate used to build the lower pavement layers may be used with a high level of abrasion loss because the high load and stress imposed by the wheels and heavy traffic have less impact on them than the aggregate in the top layers, mostly near the pavement surface, which are more subjected to abrasion and required more hardness and toughness. ASTM C131 and AASHTO T 96 standard illustrate the procedure of the los angles abrasion test.

3.3.3 Polished Stone Value

The polished stone value (PSV) is a measure of the aggregate resistance against polishing resulting from friction of vehicles tyres. Where the road surface has large aggregate, the polish status of the specimen is one of the main factors affecting the skid resistance. The real relationship between skidding resistance and PSV rise with the type of surface and traffic conditions.

The PVS is conducted in two parts; first phases, the samples of stone are subjected to polishing action through an accelerated polishing machine. In second phase, the polishing status achieved with each sample is gauged through a friction test, which is then expressed as PSV test. The complete procedure of the test is available in the British Standard (BS 812: Part 3).

3.3.4 Flakiness and Elongation Index Test

The flakiness index of an aggregate test was taken in accordance British standard (BS 812, Part 3). The shape of the aggregate particles flat or elongate contents is one vital factor in the degradation of the aggregates in SMA mixture. The flakiness test for aggregate concerns the thin and flat property of the aggregate in respect of their length or width. In specific point of view, flaky aggregate is an aggregate in which the thickness is less than 0.6 of its mean size. The flaky index of aggregate is found by weight of flaky aggregate that passes a special sieve as a percentage of the aggregate tested. The strength of flaky aggregates is insufficient and it's less than the strength of cubical aggregates. Thus, a flaky aggregate does not have strong particles compared to the cubical aggregates.

On the other hand, the measurement of the elongation index was carried out in according to British standard (BS 812, Part 3). The elongation of aggregate concerns the length of aggregate and. When the length of aggregates is 1.8 times greater than the mean size, the aggregate becomes elongated. British standard (BS 812, Part 3) presents the flakiness and elongation test in full details with all the procedure, and, according to this standard, the flakiness index of the aggregates should not accessed the limit of 20 per cent for use in asphalt pavements.

3.3.5 Aggregate Crushing Value (ACV)

The ACV in the current study was conducted according to the British standard (BS 812: Part 110- 90) to determine the resistance of an aggregate to crushing under a gradually applied compressive load. A lower aggregate crushing value indicates the greater resistance ability of the particular aggregate against crushing. A test specimen was compacted in a standardised manner into a steel cylinder fitted with a freely moving

plunger. The specimen was subjected to 400KN load for 10 minutes. This crushed the aggregate specimen down to a degree, which is dependent on the crushing resistance level of the materials. Aggregate with ACV of 10 or less are considered as strong aggregate while those with ACV higher than 25 are considered as weak aggregates. In road pavement design, according to BS, aggregates with more than 30 per cent of ACV are not recommended.

3.4 Physical properties of Bitumen

The tests undertaken comprised the penetration test (ASTM D5), softening point test (ring and ball) (ASTM D36) and Brookfield viscosity (ASTM D4402), flash and fire point (ASTM D 92, 1986), Ductility test (ASTM D113) and elastic recovery test (ASTM D 6084- 97), short term aging and long term aging , respectively.

3.4.1 Softening Point Test (Ring and Ball)

According to the specification test, softening point is the temperature at which the bitumen reaches a particular degree of softening. 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).

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).

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.

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. 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.

3.4.4 Flash and Fire Point Test

Bitumen is a flammable material which can catch fire at high temperature and, hence, it is a high risk material if adequate care is not exercised during different stages of the construction procedure. Therefore, it is important to note that the flash and fire point of bitumen is required in order to control the temperature of the material during the mixing and construction procedures. The flash point is one measure of the tendency of the test specimen to form a flammable mixture with air under controlled laboratory conditions. This test method describes the determination of the flash point and fire point of petroleum products by a manual Cleveland open cup apparatus or an automated Cleveland open cup apparatus, as described in ASTM D92. Flash point is the lowest

temperature at which the vapour of substance momentarily takes fire in the form of a flash under specific conditions of test. In this test, a small cup of bitumen is subjected to a temperature increase up to specified level to start melting, and a small flame with 0.16 mm in diameter is applied to the surface of the molten bitumen at particular intervals as long as the first appears at any point on the bitumen surface.

3.4.5 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 $25\text{ }^{\circ}\text{C} \pm 0.5\text{ }^{\circ}\text{C}$ 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).

3.4.6 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 $25\text{ }^{\circ}\text{C} \pm 0.5\text{ }^{\circ}\text{C}$ and with a speed of $5\text{ cm/min} \pm 5\%$. 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).

3.4.7 Rolling Thin Film Oven Test (RTFOT) (ASTM D 2872)

The RTFOT procedure requires an electrically 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 to 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.

3.4.8 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.

3.5 Dynamic Shear Rheometer (DSR) (ASTM D-4 proposal P246)

This proposed standard contains the procedure used to measure the complex shear modulus (G^*) and phase angle (δ) of asphalt 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.

Time lag between applied stress and resulting strain, which is define as phase angle can be used to describe the viscoelastic behaviour of asphalt binder. If a substance is purely viscous then the phase angle (δ) is 90° that means $G' = 0$ and $G'' = G^*$. If a substance is purely elastic then the phase angle (δ) is zero that means $G' = (G^*)$ and $G'' = 0$. There are numerous studies for using the rheological techniques to predict pavement performance based on the main two rheological parameter complex shear modulus and phase angle.

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.

3.6 Marshall Mix Design

It is known that asphalt concrete mixture consists primarily of mineral aggregates, asphalt cement, and air. The main purpose of a mix design is to produce mixtures with high resistance to deformation and cracking. The basic concepts of the Marshall mix design method were originally developed by Bruce Marshall of the Mississippi Highway Department around 1939 and then refined by the U.S. Army. The Marshall mix design method was used to determine the optimum binder content (OBC) in accordance to ASTM D 1559. 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.

Marshall mix design method pays proper attention to voids, strength, and durability. The Marshall design method has been used for dense graded mix containing aggregate size of one inch or less. This method has also been significantly used for SMA showing an acceptable performance results. Therefore, due to its simplicity, this method is still considered as the most common method for designing asphalt mixes.

3.6.1 Marshall Samples Preparation

The two methods that are usually used to add the selected additive to the asphalt mixture are the wet and dry processes. In the first method, the wet process, before the addition of the binder to the asphalt mixture, the binder is first mixed with the additive. While in the dry process method, the additive is blended with aggregate prior to adding bitumen.

The emphasis during mixing is to achieve an even coating of the binder over all the aggregates and the additive (CRM). The asphalt was not added to the mixer until all the additive was homogeneously dispersed throughout the aggregate. It is quite important to ensure that the additive (CRM) is well distributed and that cases of clumping do not occur during mixing. In case this occurs, it often results in balling, inadequate aggregate coating and, hence, will lead to either an increase or decrease in the ability of the composite to absorb the applied stress. In order to ensure sufficient and uniform distribution of the additive throughout the mix, the CRM was introduced into the mixer concurrently with the heated aggregate at the beginning of the dry mix cycle. In the current study the dry process was used with some modification in the terms of

manufacturing. The manufacturing process first involved a 20 seconds agitation of the natural aggregates in the mineral skeleton in order to better homogenize them. The CRM was then added to the aggregates, and mixed together for a period of 60 second in order to ensure a homogeneous dispersion of the particles throughout the mix. Then the bitumen was added and blended with the aggregates and CRM for a 10 minutes agitation period until it was thoroughly and completely the aggregate coated with bitumen and CRM. The last ingredient added was the filler. This was followed by a final agitation lasting 5 minutes, which allowed the formation of the mastic to provide cohesion.

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 preparation was illustrated below:

- Sieving the aggregate and classified them into different sizes according to the SMA 20 gradation. Each aggregate size is separately deposited in a different tray. Before proportioning, however, all the aggregates were first desiccated in an oven.
- Weighting each aggregate size based on its proportion in the mix according to the SMA 20 gradation and keeps them in a plastic bag. All the aggregates were initially dried in an oven at 100 °C prior to proportioning.
- Sieving, weighting and classifying each of the CRM into different contents and keeps them in different plastic bag.

- The appropriate proportion of aggregate was weighed, placed into the oven and heated up to 150 °C for 3 hours.
- Bitumen required for the specimen was simultaneously heated up to temperature of 120 °C for one hour.
- Once the aggregate reached the required temperature, the needed quantity of CRM and the filler were added into the aggregates. Later, they were mixed together (blender machine).
- The required amount of heated bitumen was introduced into the mixer of (aggregate and CRM) while the mixing continued until all the aggregate and CRM have been coated totally by the bitumen.
- For mix homogeneity there should be no visual sign of any uncoated filler particles which show as white specks against the backgrounds of the bituminous mix.
- 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 Marshall mould.
- The mixture was tampered with a spatula ten times around the perimeter and another ten times over the interior of the mould.
- The stainless steel thermometer was put in the centre of the mould and mixture was then ready for compaction at temperature of 150 ± 5 °C.
- Another filter paper was used on the top of the mixture sample and later, the mould assembly was shifted to the Marshal Compactor device.
- All samples were subjected to 50 compacted blows at each side.
- After finishing compaction, each sample was kept to cool at room temperature overnight before being extruded from the mould.

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

3.6.2 Theoretical Maximum Density

The theoretical maximum density (TMD) was used to determine the volumetric properties of the samples. The asphalt mix TMD was measured through the Rice method in accordance with ASTM D2041.

3.6.3 Void in Mix

Voids in mixture (VIM) for hot mix asphalt (HMA) are defined as “the volume of the void among the aggregate coated with bitumen”. The void analysis was carried out in accordance with ASTM D3202.

3.6.4 Marshall Stability and Flow

This test method covers the measurement of the resistance to plastic flow of cylindrical specimen of bituminous paving mixture loaded on the lateral surface by means of the Marshall apparatus. The Marshall stability refers to the maximum load resistance escalated during the test procedure at 60 °C at a loading rate of 50.8 mm/min is known as the Marshall stability, which significantly depends on the aggregate quality (angularity) and asphalt binder viscosity, before the compacted specimen failure. The Marshall stability is defined “as a measurement of the susceptibility of a bituminous mixture to deformation ensuring from frequent and heavy traffic load.” Flow can be understood to mean a measurement of the permanent strain which takes place in a Marshall test at failure. It had indicated that the flow parameter as obtained from the Marshall test is rather unfortunate as a higher flow value does not necessary imply a higher tendency to flow or deform under load (Mashaan *et al.*, 2013a).

According to ASTM D1559, the standard Marshall specimen dimension is diameter of 101.6 mm and thickness of 63.5 mm. Measurement of the specimen stability and flow was carried out by immersing each sample in a water bath for half an hour at 60°C. After removing the specimen from the water bath, it was placed within the grips of Marshall apparatus. The loading was carried out at the standard rate of 50.8 mm/min and the resulting stability and flow values were recorded.

3.6.5 Optimum Binder Content

The optimum binder content (OBC) 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 OBC is the numerical average of the binder content percentage determined with the highest Marshall stability, maximum bulk density, and medium range of voids in total mix. 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 at midpoint of selecting average.
- (ii) Obtain from the test plots the value of stability, flow and VIM corresponding to the average binder content calculated in (i).
- (iii) Verify that values determined in (ii) satisfy the limiting criteria (Asphalt Institute, 1990).

For the current study, the OBC was selected to provide 4% of air voids in the mixture and satisfy the maximum Marshall stability. The National Center for Asphalt Technology (NCAT) suggest that SMA mixture produced with 4% air voids is expected to provide better rutting resistance, especially at hot climates.

3.7 SMA Mix Performance Test

3.7.1 Indirect Tensile Test (IDT)

This test covered the procedure for testing laboratory or field 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 (ASTM D4123, 82). 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.

The test was conducted at temperature of 25°C with Poisson's ratio of 0.35, pulse period equal to 1 second and the rise time equal 70 ms. The indirect tensile test for resilient modulus of bituminous was carried out according to ASTM D1234 (1987), using the UMATTA (Universal Materials Testing Apparatus) as shown in Figure 3.24. Initially, the diameter and thickness of the samples were measured using a vernier caliper 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, 1982).

3.7.2 Dynamic Creep Test (AAMAS 338)

This test simulates the passage of moving traffic loads on the pavement to study the permanent deformation characteristics of bituminous materials and its ability to resist the creep distress under repeated load. The test was conducted at temperature of 40 °C for a period of 1 hour with loading stress of 100 kPa, the pulse period; pulses width, terminal pulse and conditioning stress count were at 2000ms, 200ms, 1800 counts and 1Kpa, respectively by using the UMATTA apparatus. The cylindrical Marshall specimen was used in which its preparation is described before. Using cutting machine, both the diametral surfaces of the specimen were cut smoothly to a thickness of 50 +/- 1 mm. Then capping by using grease and powder applied at the surface, this will reduce the friction with loading plates and thus ensure a uniaxial stress condition. The specimen then would be cured in the control temperature cabinet for two hours prior testing at testing temperature of 40 °C. The deformation is measured by the change in distance between the loading plates throughout the test. The uniaxial unconfined jig comprise of an upper and lower loading platen which distribute the load evenly to the ends of the specimen.

3.7.3 Wheel Tracking Test

This test determines the susceptibility of bituminous mixtures to deform plastically at high road temperatures under pressures similar to those experienced on the road. The susceptibility of bituminous materials to deform is assessed by the rut depth formed by repeated passes of loaded wheel at constant temperature. The purpose of this test is to determine the ability of asphalt mixture to resist rutting deformation. The Transport and

Road Research Laboratory (TRRL) have developed the wheel tracking test machine for examining the rutting of bituminous mixtures under different conditions of speed and temperature. The test was conducted according to the British standard (BS 598 : 110, 1998). The testing of each specimen was conducted at 45°C with 21 cycles/minute. In accordance with the standard, the test lasted for 45 min or longer got the accuracy of 15 mm of deformation in the tested specimen. A 520 kN loading was imposed on the specimen surface by a wheel with 50mm width, and every 5 min, every 105 cycles, the rut depth was read from the gauge and recorded accordingly. For specimen preparation in this study, the test slabs were prepared with an equivalent density to the corresponding Marshall specimens.

3.7.4 Moisture Induced Damage Test

Moisture susceptibility is a primary cause of distress in bituminous mixtures. It is an assessment of how susceptible a bituminous mixture's internal binder- aggregate bond is to weakening in the presence of water. Water will cause the binder to not adhere to the aggregate. Since the binder is the "glue" that holds the pavement together, rapid failure of the pavement can be expected if the binder cannot adhere to the aggregate. This is often referred to as stripping. The indirect tensile strength test was employed in full compliance with AASHTO T 283 standard to determine the susceptibility level of the SMA samples against moisture in wet environments.

3.7.5 Binder Drain Down Test

The binder drain down test is more essential for SMA than conventional dense-graded mixture. This test was developed and introduced by AASHTO T 305 (2000), it is usually expected to simulate the likely conditions that the mixture will encounter during its production, storage, transportation and placement stages. This test was carried out on three loose mixture representing each selected mixture type to ensure that the

engineering properties of the binder drain down of the SMA fell within the acceptable range of the preferred levels following the mentioned standard.

3.7.6 Indirect Tensile Fatigue Test

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 in to two pieces.

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.

Universal Materials Testing Apparatus (UMATTA) was used to determine the repeated load indirect tensile test as a method of assessing the fatigue resistance of bituminous materials. A stiffness reduction of 50% was used to present the sample failure due to fatigue deformation. 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 allowed 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.

3.8 Adaptive Neuro-Fuzzy Application

The adaptive neuro-fuzzy inference system (ANFIS) can serve as a basis for constructing a set of fuzzy ‘If-Then’ rules with an appropriate membership function to generate the stipulated input-output pairs (Shahaboddin *et al.*, 2015). In this study, the ANFIS system that is functionally equivalent to the first-order Sugeno fuzzy model was used. A typical rule set with a fuzzy ‘If-Then’ rule can be expressed as

$$\text{if } x \text{ is } A \text{ then } f_1 = p_1x + t \quad \dots\dots\dots\text{Equation (3.1)}$$

The ANFIS architecture for three inputs x , y and z is shown in Figure 3.28. Nodes at the same layer have similar functions. The output of the i th node in layer l is denoted as $O_{l,i}$.

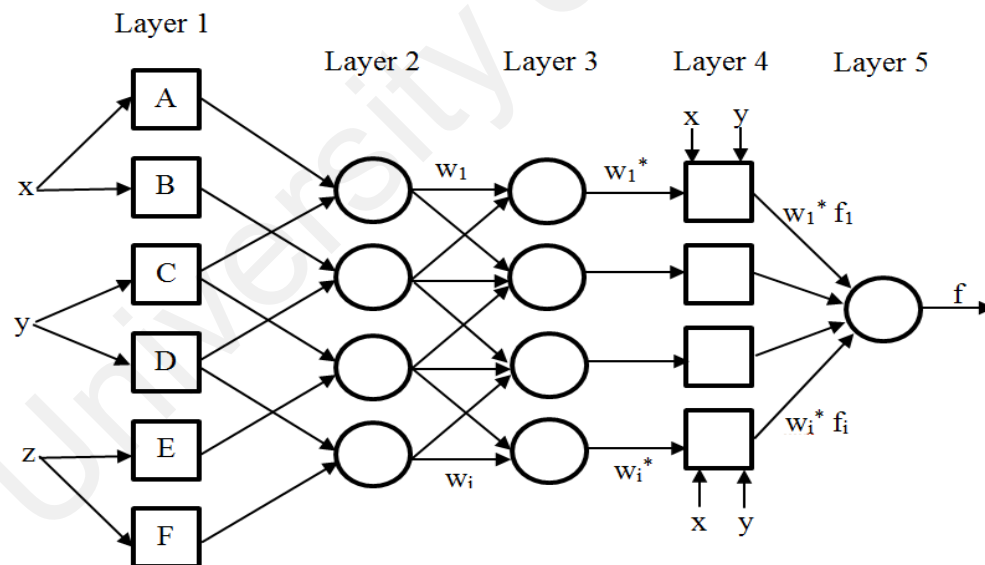


Figure 3.3: ANFIS structure with two inputs

The first layer consists of input variables membership functions (MFs) and supplies the input values to the next layer. Every node i is an adaptive node with a node function

$$O_{1,i} = \mu(x, y, z)_i \text{ for } i = 1, 2 \dots \dots \dots \text{Equation (3.2)}$$

where $x, y =$ input to the i th node; $\mu(x, y, z)_i =$ membership functions.

The MFs can be described by bell-shaped function

$$f(x; a, b, c) = \frac{1}{1 + \left(\frac{x-c}{a}\right)^{2b}} \dots \dots \dots \text{Equation (3.3)}$$

where $\{a, b, c\}$ is the parameter set.

The second layer (membership layer) multiplies incoming signals from the first layer and sends the product out. Each node output represents the firing strength of a rule or weight like

$$O_{2,i} = w_i = \mu(x)_i \cdot \mu(x)_{i+1}, i = 1, 2. \dots \dots \dots \text{Equation (3.4)}$$

The third layer (i.e. the rule layer) is non-adaptive where every node i calculates the ratio of the rule's firing strength to the sum of all rules' firing strengths like

$$O_{3,i} = w_i^* = \frac{w_i}{w_1 + w_2}, i = 1, 2. \dots \dots \dots \text{Equation (3.5)}$$

The outputs of this layer are called normalized firing strengths or normalized weights. The fourth layer (i.e. the defuzzification layer) provides the output values resulting from the inference of rules, where every node i is an adaptive node with node function

$$O_{4,i} = w_i^* \cdot f_i = w_i^*(p_i x + q_i y + r_i) \dots\dots\dots \text{Equation (3.6)}$$

where $\{p_i, q_i, r_i\}$ is consequent parameters.

The fifth layer sums up all the inputs from the fourth layer and converts the fuzzy classification results into a crisp output. The node in the fifth layer is not adaptive and this node computes the overall output of all incoming signals

$$O_{5,i} = \sum_i w_i^* \cdot f_i = \frac{\sum_i w_i \cdot f_i}{\sum_i w_i} \dots\dots\dots \text{Equation (3.7)}$$

3.8.1 Evaluation of Model Performances

To analyse the performance of the ANFIS model and measurement values, the following statistical indicators were selected:

- 1) root-mean-square error (RMSE)

$$RMSE = \sqrt{\frac{\sum_{i=1}^n (O_i - P_i)^2}{n}}, \dots\dots\dots \text{Equation (3.8)}$$

- 2) coefficient of determination (R^2)

$$R^2 = \frac{\left[\sum_{i=1}^n (O_i - \bar{O}_i) \cdot (P_i - \bar{P}_i) \right]^2}{\sum_{i=1}^n (O_i - \bar{O}_i) \cdot \sum_{i=1}^n (P_i - \bar{P}_i)} \dots\dots\dots \text{Equation (3.9)}$$

where O_i = ANFIS value, P_i = measurement values, and n = the total number of test data.

3.9 Summary

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 SMA mixes. 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.

There are different test methods used throughout the world to measure fatigue resistance for asphalt concrete mixtures. This study 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

CHAPTER 4: RESULTS AND DISCUSSION

4.1 Introduction

The current chapter focuses on the results of the experiments and tests elaborated upon in the previous chapter. High quality materials are essential for obtaining high quality SMA mix. Acceptability of aggregate durability range and other tests required for the aggregates are among the factors that contribute to the desired quality of SMA. In addition, the significant role that asphalt plays in SMA mix is not ignorable. Therefore, the process began with tests of the materials. The achieved results were compared with the minimum standard requirements to determine the validity of the selected materials. The determination of optimum binder content was carried out based on the Marshall mixture procedure. The following parameters were considered- Marshall stability, Marshall flow and bulk density.

This chapter also presents the SMA performance tests, which included various percentage of waste CRM. The performance tests including, resilient modulus, wheel tracking test, dynamic creep test, drain down, moisture susceptibility test and fatigue test were carried out on the mixture that included various contents of CRM of 0, 6, 12, 16 and 20% by weight of asphalt content. In addition, in this chapter, the accuracy of soft computing techniques was employed for the prediction of Marshall stability, Marshall flow, resilient modulus (stiffness), rutting and fatigue life of CRM modified asphalt mixture. The process, which simulates the mixture's deformation, was constructed with adaptive neuro-fuzzy inference system (ANFIS).

4.2 Materials Test Results

4.2.1 Bitumen Test Results

The bitumen used for this study was 80/100 penetration grade and the quality control of the bitumen was gauged through bitumen test of penetration, softening point, Brookfield viscosity, ductility, flash point and fire point. Short term aging RTFOT and long term aging PVA were also determined. The results of these tests are presented in Table 4.1.

Table 4.1: Properties of Base Binder Grade 80/100 Penetration

Test properties	Standard test	Value	Standard requirement
Viscosity @135 °C (Pas)	ASTM D4402	0.65	-
$G^*/\sin \delta$ @ 64°C (kpa)	ASTM D-4	1.35	1000
Ductility @ 25 °C	ASTM D-113	> 100	+ 100
Elastic recovery @ 25 °C	ASTM D6084	19.8	-
Softening point @ 25 °C	ASTM D-36	48	47-49
Penetration @ 25 °C	ASTM D-5	88	84-95
Flash point (°C)	ASTM D-92	300	275-302
Fire point °C	ASTM D92	317	>302
RTFOT aged $G^*/\sin \delta$ at 64 °C (kPa)	ASTM D 2872	6.022	-
PAV aged $G^*\sin \delta$ at 25 °C (kPa)	ASTM D 6521	3122.5	+ 5000

The obtained values displayed in the above table confirm that this type of bitumen meets the standard requirements and it is a suitable choice for use in asphalt mixture construction.

4.2.2 Aggregate Test Results

The quality of aggregate has a central role in the quality of asphalt mixture and the performance of the mixture is significantly affected by the aggregate blend properties. The properties of the aggregate that significantly affect the asphalt mixture include

shape, size, gradation and strength, which need to be strictly controlled and monitored. Crushed granite with SMA 20 that was obtained from Kajang Granite Quarry Sdn Bhd (a suburb near Kuala Lumpur, Malaysia capital) was used as aggregate materials. The results of aggregate tests are tabulated in Table 4.2.

Table 4.2 : Physical properties of the crushed aggregate

Test	standard test	SMA 20 value	standard value
L.A. abrasion (%)	ASTM C-131	21%	below 30%
Soundness (%)	BS812: Part 3	4.10%	Below 12%
Flakiness index (%)	BS 182 : Part 3	4%	below 20%
Elongation index (%)	BS 182 : Part 3	14.30%	below 20%
Impact value (%)	BS812: Part 3	12.40%	below 15%
Polished stone value (%)	BS812: Part 3	48.50%	above 40%
Aggregate crushing value (%)	BS812: Part 3	20%	below 30%

Based on the achieved test results, this aggregate could satisfy the standard requirements, which makes it suitable for used in the asphalt mixture construction.

4.3 Marshall Test Results

4.3.1 Marshaall Sability

The results obtained for various CRM content for each binder contents are shown in Figure 4.1. Marshall stability refers to the maximum load resistance escalated during the test procedure at 60°C at a loading rate of 50.8 mm/min is known as the Marshall stability, which significantly depends on the aggregate quality (angularity) and aspahtl binder viscosity, before the compacted specimen failure. The Marshall stability is defined “as a measurement of the susceptibility of a bituminous mixture to deformation ensuring from frequent and heavy traffic load” (Mashaan *et al.*, 2013b).

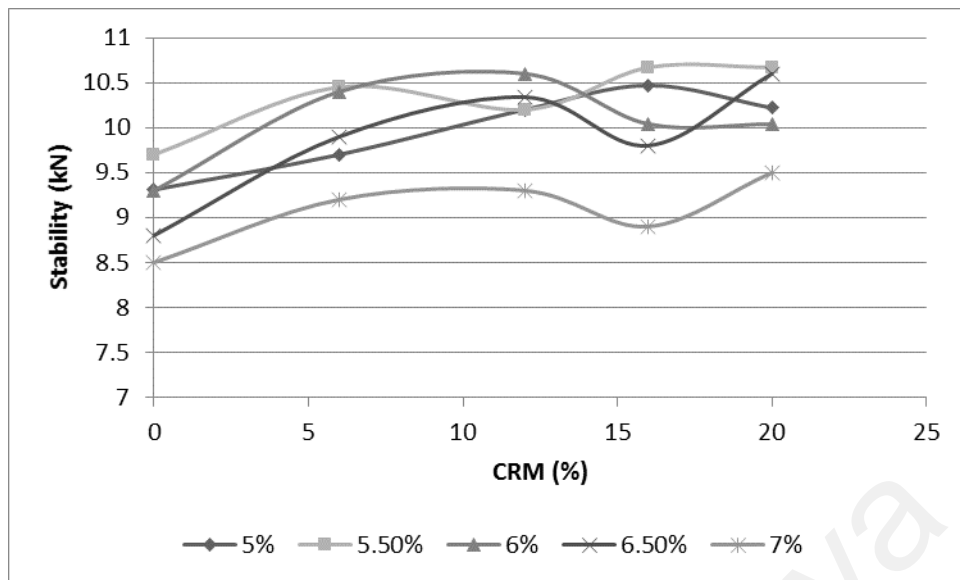


Figure 4.1 : CRM content and Marshall Stability relationships

Figure 4.1 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 12-16% 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. 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.

In Malaysia, 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. In another investigation models were developed to estimate Marshall Property of polypropylene modified asphalt mixture using neural network. Different variables were designated to predict Marshall Property of mixtures including: polypropylene type, polypropylene percentage, bitumen percentage, unit weight, specimen height, air voids, voids in mineral aggregate and voids filled with asphalt. Final results of this study showed that there was a good agreement between the predicted responses obtained from the neural network models and the experimental results (Tapkın & Cevik, 2010).

4.3.2 Marshall Flow

The results obtained for various CRM content for each binder contents are shown in Figure 4.2. Flow value is known “as the total movement of strain in units of 0.25 mm which occurred in the specimen between no load and maximum load during the stability test” (Mashaan *et al.*, 2013b). Flow can be understood to mean a measurement of the permanent strain which takes place in a Marshall test at failure. It had indicated that the flow parameter as obtained from the Marshall test is rather unfortunate as a higher flow value does not necessarily imply a higher tendency to flow or deform under load (Mashaan *et al.*, 2013b).

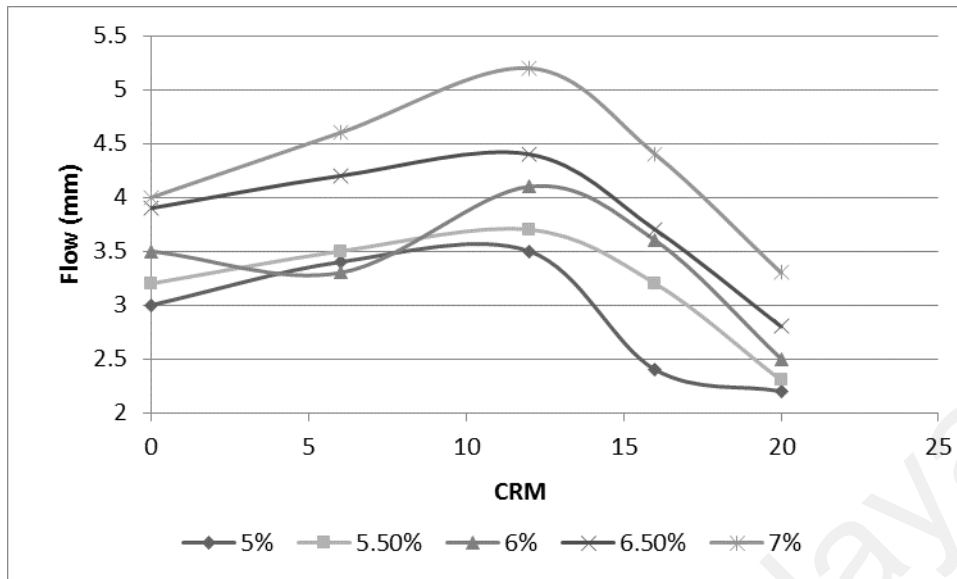


Figure 4.2 : CRM content and Marshall Flow results

Figure 4.2 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 flow. Additionally, Figure 4.2 shows that the increase in CRM content in the SMA mixture does not necessarily increase the flow values. Increased CRM content in the mix increased the stability value as shown in Figure 4.1. 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). The high flow value indicates that the mixture has high plasticity, which can lead to pavement rutting failure in the life service. Also, the low flow value could be as results of high volume of voids and insufficient bitumen content, which may lead to pavement cracking deformation (Mashaan *et al.*, 2013b).

4.3.3 Density of 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 4.3 shows that for any specific CRM 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.

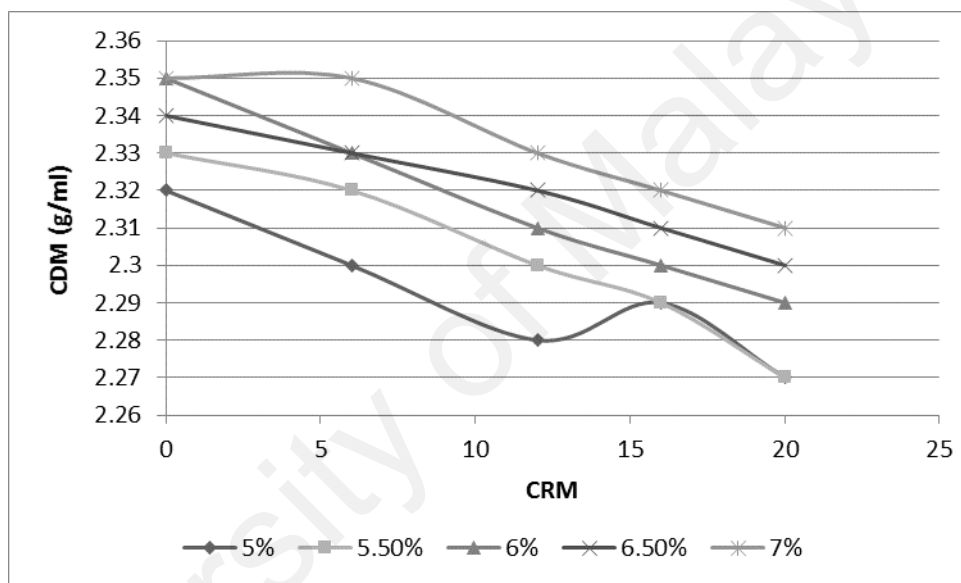


Figure 4.3 : CDM results using different CRM content

The results indicated a lower density for the mixtures with incorporation of crumb rubber. Figure 4.3 shows 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. In Malaysia, other researcher's results (Mahrez, 2008) that had used dry-SMA mix with glass fibre, it showed significant use of improving the performance properties of fiber glass-SMA dry mix. Current study of using CRM-SMA dry mix showed significant and promising method for pavement engineering, which revealed that for ideal paving mixture a good correlation between binder viscosity and the compaction effort is required.

4.3.4 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 4.4.

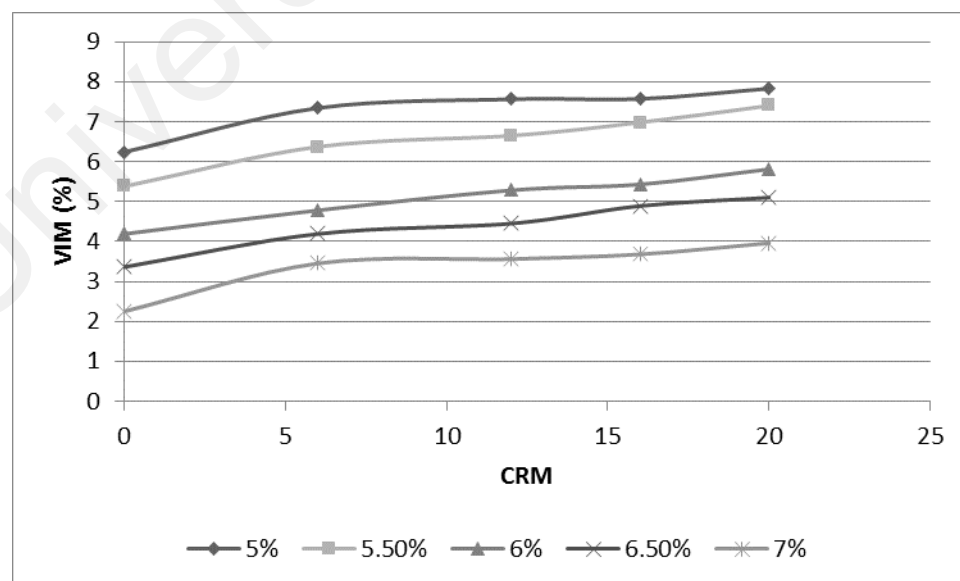


Figure 4.4 : VIM results using different CRM content

Figure 4.4 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 accounted whereby bitumen is squeezed out of the mix to the surface.

4.3.5 Analysis of Variances (ANOVA) Test of Marshall Test

Analysis of Variances (ANOVA) is a statistical method that is used for assessing relationships and differences among the means of two or several data sets. It is a guide for determining whether or not differences in a set of counts or measurements were most likely due to the modern chance variation. In this study, ANOVA was performed using Two Factor without replication: this performs an analysis of variance between two or more data sets (Mashaan & Karim 2013a). Normally, it is used when there is only one sample from each data set. Results of ANOVA test for Marshall parameters are found in Appendix A.

“The significant level of the data sets is determined by evaluating the F- ratio and comparing it to the F- critical value for the samples. If the F- ratio (F- Statistic) is larger than F- critical, then the variation between the groups is statistically significant. The observed P- value is the probability of observing the F- ratio or larger when the mean test results are equal” (Mashaan & Karim 2013a). If the P- value is less than the desired level of significance (α), then the corresponding variant becomes significant. The level

of significance (α) used in this research was 0.05 that represented a probability of 5% of the hypothesis represented, thus, the model may not be true (Mashaan, 2012).

4.3.6 Marshall Results and ANFIS

The adaptive neuro-fuzzy inference system (ANFIS) can be used as a foundation for constructing a set of fuzzy 'If-Then' rules with an appropriate membership function to generate the stipulated input-output pairs.

In this study, analyses of the influence of CRM content and asphalt binder content on Marshall properties including of stability, flow, void in mix and density of compacted mix. Since it is a very uncertain and nonlinear process analysing could be very challenging and time consuming, soft computing techniques are preferred. ANFIS is one of the most powerful types of neural network system. ANFIS shows very good learning and prediction capabilities, which makes it an efficient tool to deal with encountered uncertainties in any system. ANFIS, as a hybrid intelligent system that enhances the ability to automatically learn and adapt, was used by researchers in various engineering systems (Shahaboddin *et al.*, 2015).

The process of estimating the accuracy and prediction model, which simulates the Marshall property with adaptive neuro-fuzzy inference system (ANFIS) was constructed. The developed ANFIS network was with two neurons in the input layer, and one neuron in the output layer (Shahaboddin *et al.*, 2015). The inputs are crumb rubber content and asphalt binder content using aggregate gradation of stone mastic asphalt SMA20. The performance of proposed system is confirmed by simulation results. The ANFIS results are compared with the experimental results using root-mean-square error (RMSE) and coefficient of determination (R^2). The experimental results

show that an improvement in predictive accuracy and capability of generalization can be achieved by the ANFIS approach. The following characteristic are obtained: ANFIS model can be used for determine Marshall Stability, Marshall flow, void in mix, and density of mix with high reliability. The experimental data of Marshall test and predicted values using ANFIS model are shown in Figure 4.5. As can be seen in Figure 4.5, R^2 correlation coefficient is very high. Therefore ANFIS has good correlation with the training data.

The performance of ANFIS model that estimated Marshall properties was evaluated according to statistical criteria such as RMSE and coefficient of determination R^2 . This confirms the RMSE statistics evaluated in Table 4.3. It should be concluded, that the proposed ANFIS model can be used for forecasting with high reliability.

Table 4.3: Performance statistics of the ANFIS model in Marshall test estimation

	RMSE	R^2
Marshall stability	0.153742	0.9661
Marshall flow	0.032275	0.9941
Void in mix (VIM)-	0.043243	0.999
Density of compacted mix	0.004262	0.987

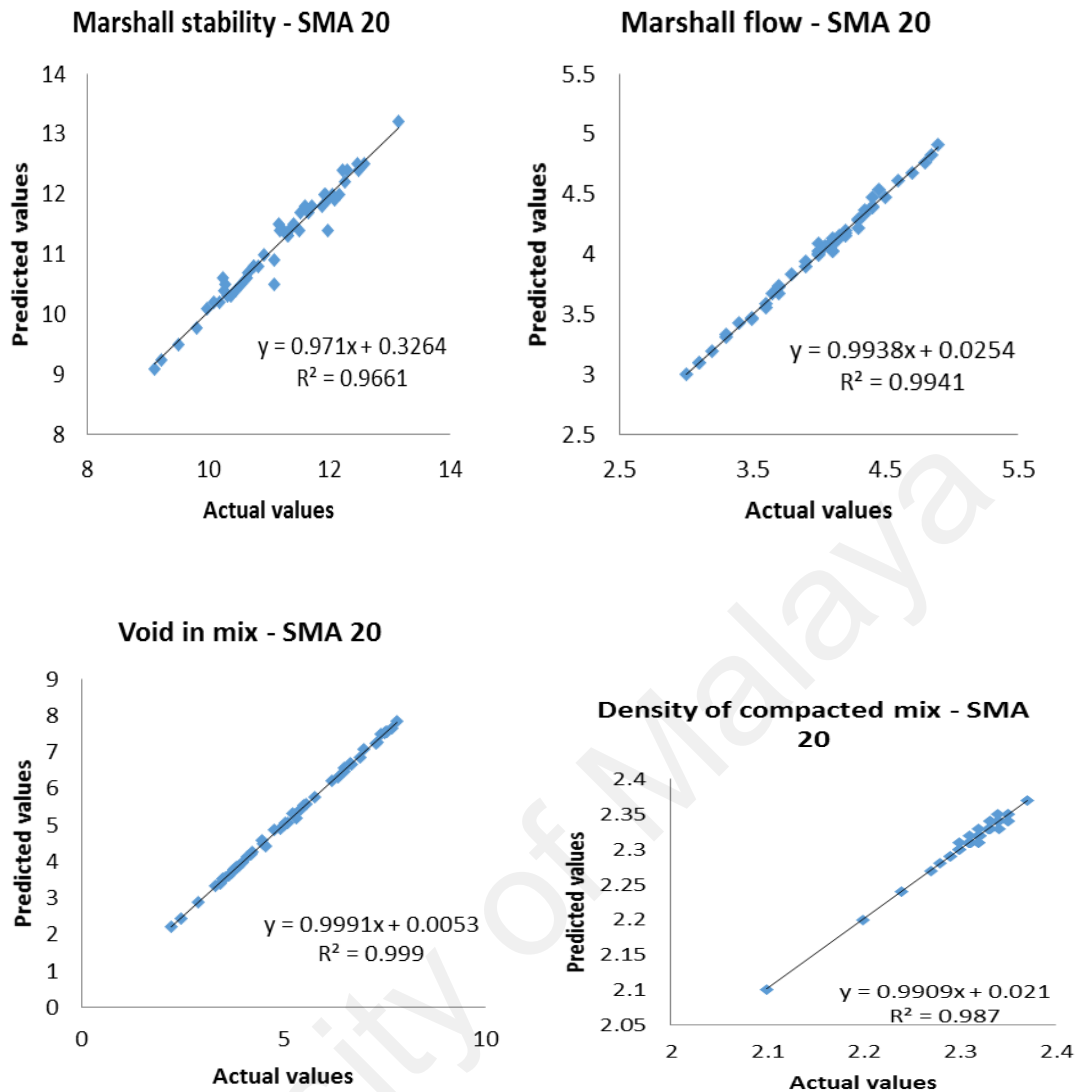


Figure 4.5: ANFIS model used to predict values of Marshall Test

4.4 Optimum Binder Content (OBC) Results

The method of the Marshall mix design was employed to determine the OBC for different CRM content rating between 0- 20%. For the determination of OBC, three graphs, namely, stability, bulk density and air void were plotted versus the percentage of bitumen binder for all the content of CRM that have been used in this study. Based on the results of Marshall test, OBCs were calculated and registered. According to Asphalt Institute, the OBCs were selected in a way to satisfy the following requirements (Mashaan *et al.*, 2013b):

- Maximum Marshall stability
- Maximum density
- Median range of air void (between 3-5% for SMA)

The OBCs obtained from the test parameters for the mixes with various CRM content are tabulated in Table 4. 4

Table 4.4: The OBCs for the mixes with different CRM content

CRM %	0	6	12	16	20
OBC	5.9	6.5	6.40	6.20	6.31

4.5 Performance Test Results

4.5.1 Indirect Tensile Test Results

In order to determine the stiffness properties, the resilient modulus of specimens was tested following the standard method (ASTM D 4123). In asphalt samples, as a result of the surplus strain, cracks appeared in relation to the tensile strength which was primarily micro-cracks. These cracks were vertical to the maximum tensile stress direction; consolidate 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.

As shown in Figure 4.6, the value of the resilient modulus with different CRM-reinforced SMA at OBC is greater than the non-reinforced SMA (without CRM). This means that at optimum conditions the control samples (non-reinforced) has a bigger elastic deformation than the rubberised samples (CRM-reinforced-SMAs) under dynamic traffic loading conditions. The results displayed an increase in resilient modulus as the crumb rubber contents increases up to optimum value (16 % CRM) than it decreases back with further increase of crumb rubber content. The reduction of the MR after adding more than 16% CRM can be related to excessive OBC content and mainly on the constant air void required. As revealed in Figure 4.8, there is a marked variation between the modified and non-modified mixes in the stiffness modulus. In modified asphalt mix with CRM, the crumb rubber content absorbs a portion of bitumen resulting in the optimum asphalt content to increase. As the crumb rubber content is increased, more bitumen is absorbed, which in turn increases the optimum asphalt content of the mix. It is evident that the stiffness of modified asphalt samples is higher compared to the non-modified samples.

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 (Mashaan *et al.*, 2013b). 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).

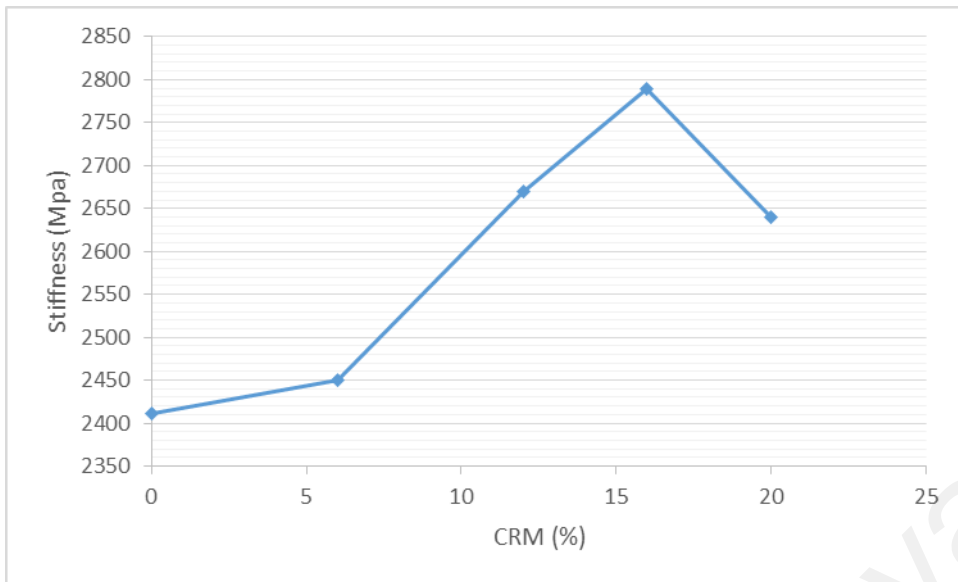


Figure 4.6: Resilient modulus results using various CRM content

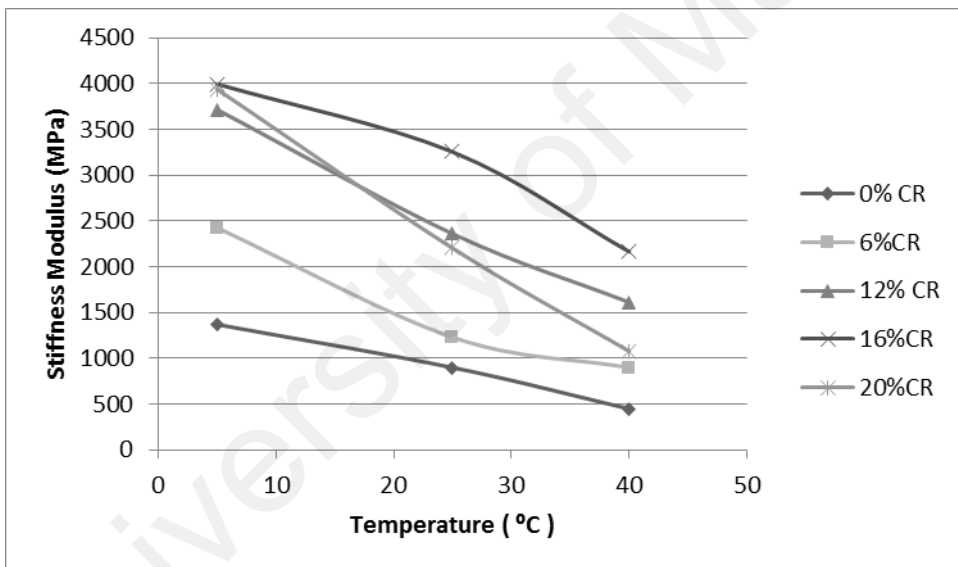


Figure 4.7: Effect of different temperature on Stiffness modulus

Figure 4.7 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. 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).

The resilient modulus results 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).

4.5.2 Indirect Tensile Fatigue Test

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 4.5 for various stresses.

Table 4.5 and Figures 4.8-4.10, 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. As shown in the figures, 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 4.5. 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 4.5 indicates the variation of cyclic loading on the specimens containing varying percentage 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. 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 (Mashaan et al., 2013c; Arabani *et al*, 2010). However, the number of cycles to failure different for asphalt samples which contain various percentages of crumb rubber. Reinforced samples tend to have longer fatigue life compared with non-reinforced samples. From Table 4.5 and Figures 4.8-4.10, 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. Guddati *et al.* (2002) have also stated that there is good potential in predicting fatigue cracking using indirect tensile strength results. A study was conducted 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.

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_i) 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.

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. 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; that is, such mixes are more likely to fail by cracking (Mashaan *et al.*, 2013b). In addition, the higher crumb rubber content, the lower $G \cdot \sin \delta$ led to higher resistance to fatigue cracking bitumen (Ali *et al.*, 2013).

Table 4.5: Fatigue models and fatigue prediction equations

CRM (%)	σ (N)	Fatigue module	K1	K2	R ²
0	2000	$Nf = 2.45 \times 10^{13} (1/\epsilon)^{0.322}$	2.23×10^3	1.45	0.89
	2500	$Nf = 2.361 \times 10^{12} (1/\epsilon)^{0.22}$	2.45×10^3	1.65	0.85
	3000	$Nf = 2.70 \times 10^{10} (1/\epsilon)^{1.1}$	1.99×10^{10}	1.87	0.88
6	2000	$Nf = 2.87 \times 10^{11} (1/\epsilon)^{1.111}$	2.44×10^{13}	1.34	0.94
	2500	$Nf = 3.56 \times 10^{12} (1/\epsilon)^{0.322}$	2.88×10^{11}	1.66	0.922
	3000	$Nf = 3.87 \times 10^{12} (1/\epsilon)^{2.112}$	2.09×10^{13}	1.87	0.933
12	2000	$Nf = 2.89 \times 10^{12} (1/\epsilon)^{1.312}$	1.989×10^{11}	1.97	0.94
	2500	$Nf = 3.55 \times 10^{12} (1/\epsilon)^{2.22}$	2.344×10^{11}	1.88	0.951
	3000	$Nf = 2.99 \times 10^{12} (1/\epsilon)^{0.11}$	2.11×10^{10}	2.11	0.911
16	2000	$Nf = 3.10 \times 10^{10} (1/\epsilon)^{0.23}$	1.98×10^{10}	2.88	0.944
	2500	$Nf = 2.78 \times 10^{10} (1/\epsilon)^{1.03}$	2.93×10^{11}	2.99	0.935
	3000	$Nf = 3.11 \times 10^{11} (1/\epsilon)^{0.03}$	2.78×10^{10}	3.11	0.916
20	2000	$Nf = 2.56 \times 10^{13} (1/\epsilon)^{1.3}$	2.90×10^{10}	2.88	0.899
	2500	$Nf = 2.88 \times 10^{13} (1/\epsilon)^{1.10}$	2.95×10^{11}	2.90	0.970
	3000	$Nf = 4.23 \times 10^{13} (1/\epsilon)^{2.1}$	3.11×10^{12}	2.89	0.99

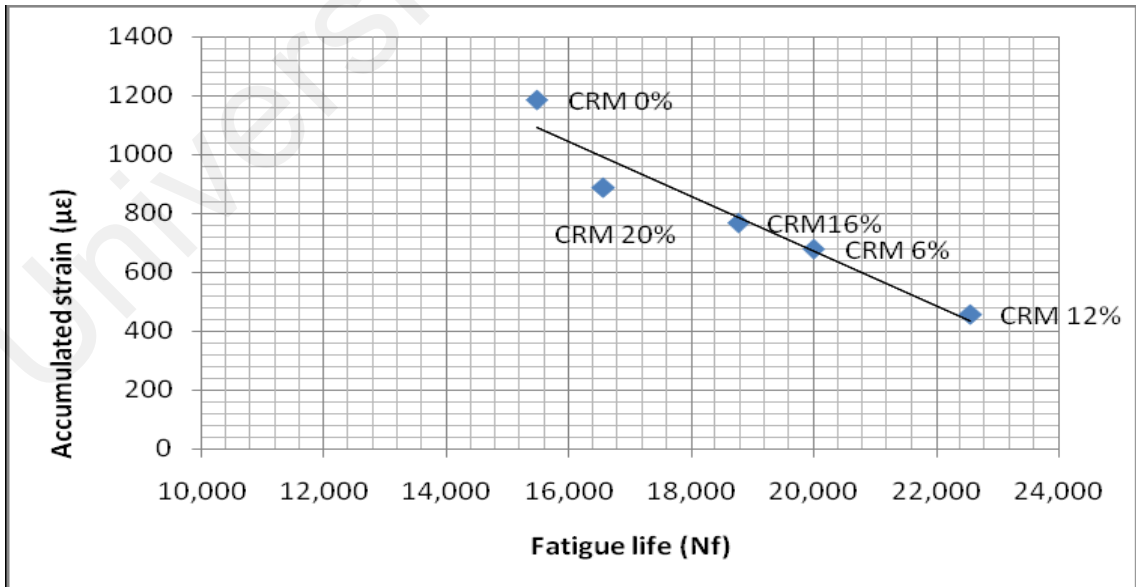


Figure 4.8: Fatigue life and related Strain at 2000 N stress

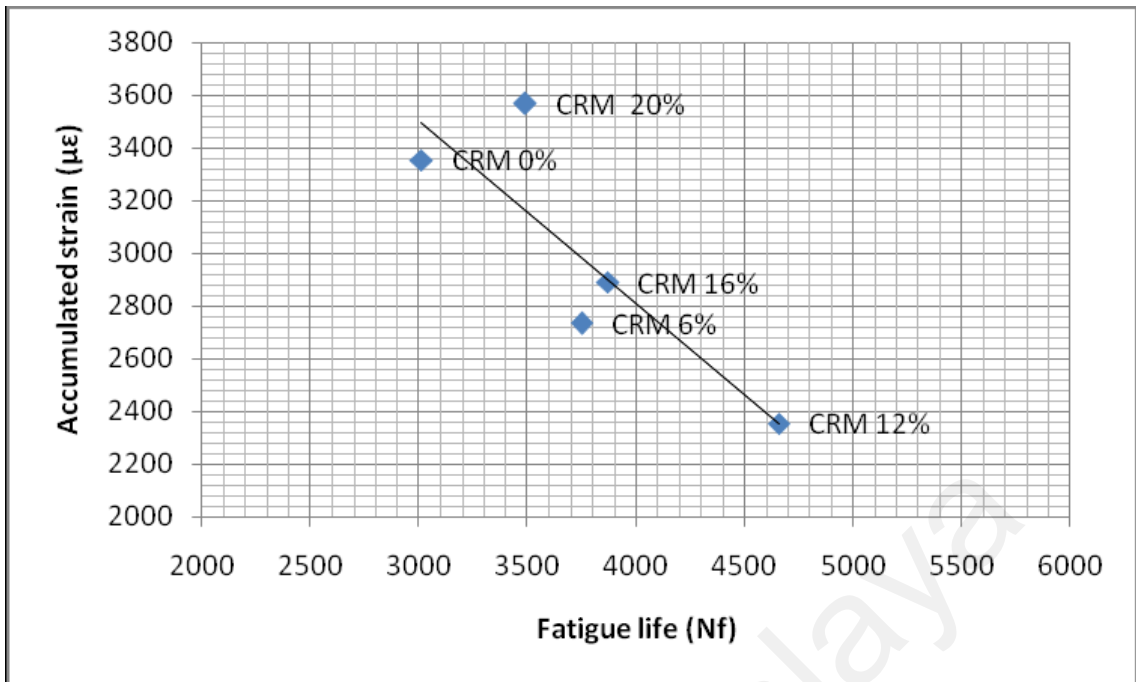


Figure 4.9: Fatigue life and related Strain at 2500 N stress

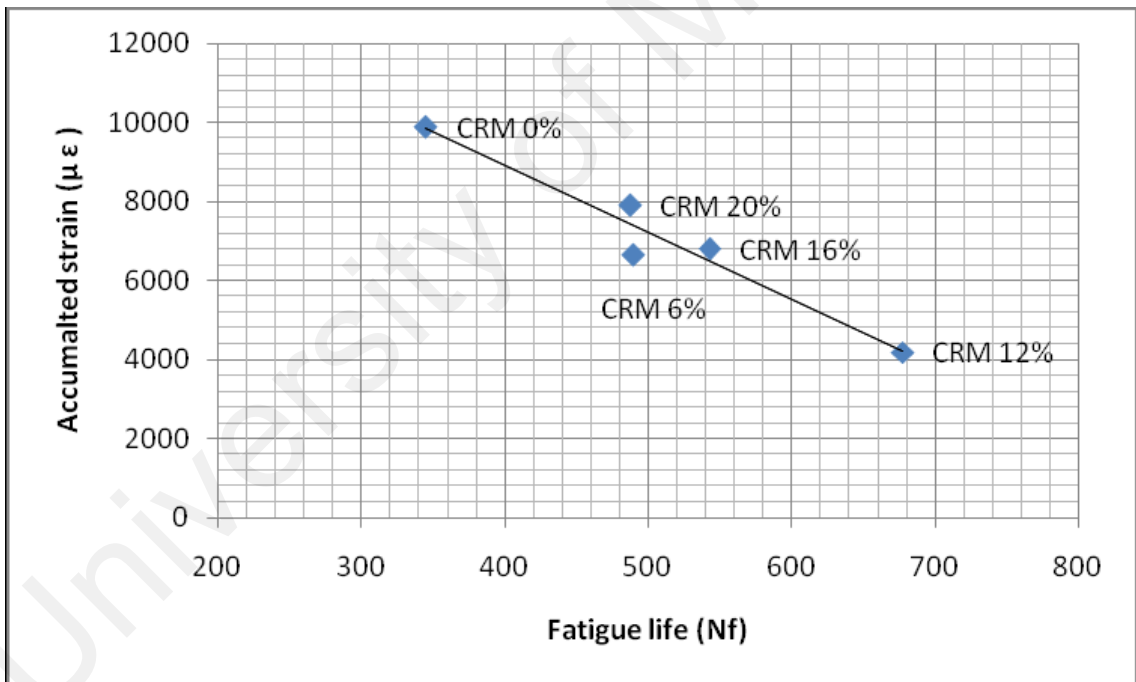


Figure 4.10: Fatigue life and related Strain at 3000 N stress

4.5.3 Dynamic Creep Test Results

The dynamic creep test is used because it is ranked between the wheel tracking test and static creep and its ability to quantify the permanent deformation of the asphalt mix. The results obtained for dynamic creep test represented by the permanent strain for mixes at OBC with different CRM content at testing temperature of 40°C is given in Figure 4.11. The results obtained displayed the same trend as before, a decrease in strain values as the crumb rubber content increases and with further inclusion of rubber the strain value increases back. Crumb rubber content of 16% provided the lowest strain value. The addition of 12 and 16% CRM to the mix reduce the strain value significantly as compared to the control mix. The dynamic creep data suggests that at the optimum condition the addition of more than 16% CRM will cause minimal effect on resistance to permanent deformation. Also, following the trend of the plotted data as shown in Figure 4.11, it is expected that with further inclusion of CRM and beyond certain CRM content the strain value of the mixture will increase higher as compared with that of the non-reinforced mix causing a detrimental effect to the reinforced mix by reducing its resistance to permanent deformation.

According to a study attributed to the effect of rubber content, enhancing the rubber content led to the increase in carbon black reacting with the natural rubber, which corresponded to the elastic part of the crumb rubber chemistry. It seems that higher CRM content has significant effect on the elastic recovery of the modified bitumen by increasing the rubber mass due to the absorption of maltene from the bitumen binder. Thus, the modified binder become more elastic and thus improved its resistance to elastic deformation under high tensile stress (Mashaan *et al.*, 2013b; Ali *et al.*, 2013; Mashaan & Karim, 2013a).

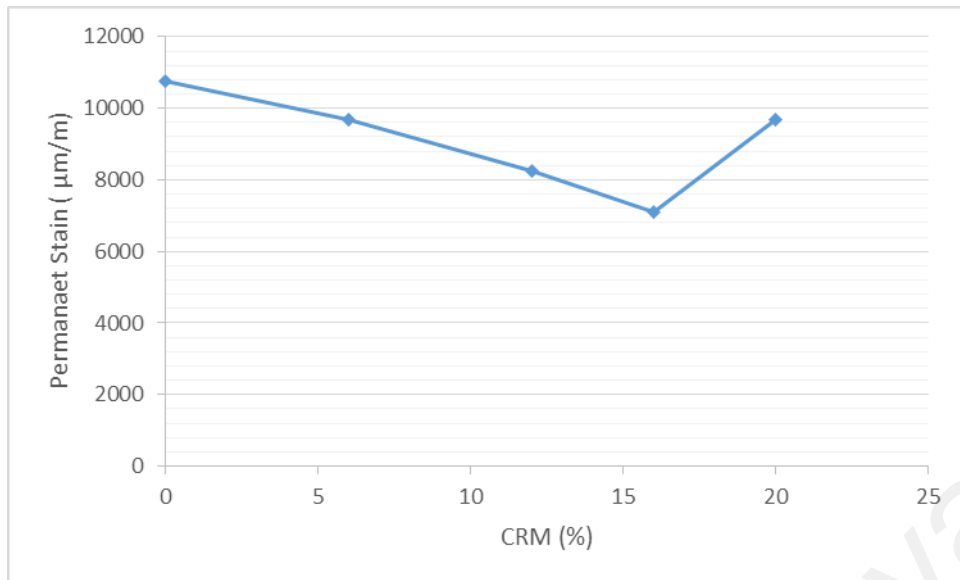


Figure 4.11: dynamic creep for various CRM content

4.5.4 Wheel Tracking Test Results

Rutting is known as longitudinal depressions which follow the line of the wheel path. The wheel tracking test determines the susceptibility of bituminous mixtures to deform plastically at high temperatures under pressures similar to those experienced on the road. The susceptibility of bituminous to rutting is based on pass/fail criteria formed by repeated passes of a loaded wheel. This method measures the rut depth and number of pass to failure. The test measures the rutting of asphalt mixture by rolling a steel wheel across the surface of an asphalt mix slab that is held at temperature of 60° C (Katman, 2006). The result obtained from wheel tracking test represented by rut depth for mixes with different CRM content at optimum binder content (OBC) is plotted in Figure 4.12. The figure show that rut depth measurement provided a similar trend as compared to the creep test results, a reduction in rut depth as the crumb rubber content increases up to an optimum value which corresponds in the case to crumb rubber of 16% and then it increases back with further inclusion of 20% CRM. The results could be explained by the physical and chemical interactions during the mixing process of the rubberised bitumen. Also, it 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, leading to the production of viscous gels that enables the bitumen binder to coat the aggregate effectively. Accordingly, an increase in binder mass could make the binder more elastic, stiff and highly resistant to pavement rutting (Mashaan *et al.*, 2013b; Ali *et al.*, 2013; Mashaan & Karim, 2013a). From Figure 4.12, the results show that the use of CRM in reinforcing SMA samples significantly improved the rutting performance as compared to the non-reinforced SMA (without CRM). The increase in rubber content leads to increase in elastic recovery results and it is similar to ductility results of rubberised bitumen, which presented consistency with binder elasticity, rheology, rutting resistance and elastic recovery after deformation. Hence, lead to improving modified binder's resistance to rutting (Mashaan *et al.*, 2013b; Mashaan *et al.*, 2012a). According to Hamed (2010), addition of crumb rubber and SBR increases the rutting resistance of asphalt paving mixtures. The results from laboratory study showed that the CR-modified and SBR-modified asphalt had higher stiffness at 60°C than the modified mixtures. The modified asphalt mixtures also had higher gyratory shear strengths and lower rut depths in the Loaded Wheel tests than the unmodified mixtures. Tayfur *et al.* (2007) mentioned that after the initial densification, the permanent deformation of the bituminous mixture happens due to shear loads which take place occur close to the pavement surface which in fact is the contact area between the tyre and the pavement. These efforts increase without the volume variations in the bituminous mixture. They are the primary mechanisms in the development of rutting during the life span of the pavement design.

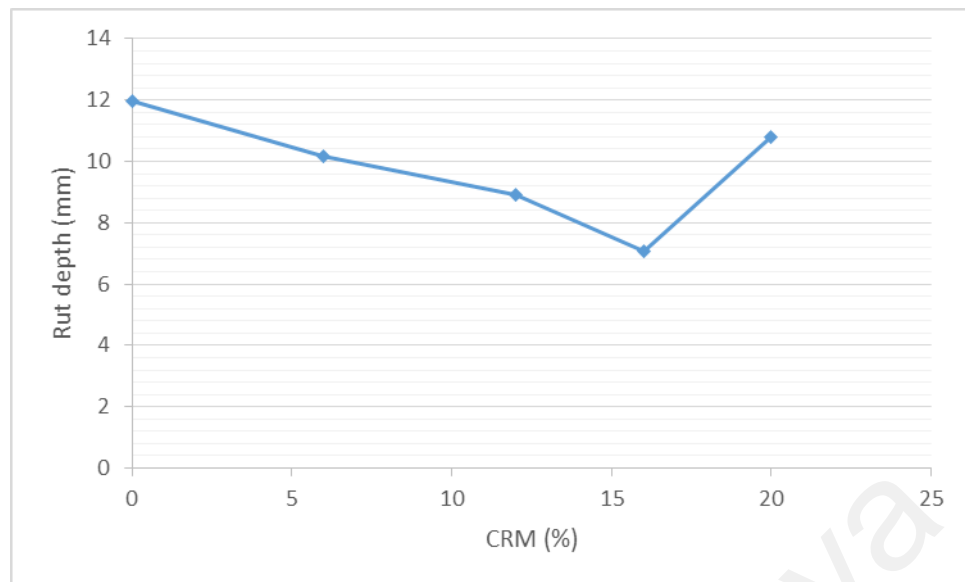


Figure 4.12: Rut depth plot for various CRM content

4.5.6 Drain Down Results

Because the interaction of CRM and virgin binder in the dry process was unknown, drain down of the mixtures was a concern for the gap graded mixtures, SMA, in this study. As illustrated in Figure 4.13, the drain down of CRM-reinforced-SMA show lower results as compared to the control mix. Moreover, results displays that the CRM 20 % has the best and optimum results of drain down of 0.21% which was much lower than the control mix of 0.3% drain down. This might indicate that the dry process using the CRM led to the formation of a viscous gel in the mixture. However, the results of drain down show that CRM-reinforced- SMA was lower than the drain down value of the non-reinforced mixture, and, thus, any additional CRM content might added into the mixture would reduce the value of the drain down. The lower value of drain down value might be attributed to the CRM used in the mixture, which remains in powder form, leading to an increase in the surface area. The increased surface area, however, needs to be wetted with binder, which would finally lead to stabilizing and holding the binder on its surface and decrease the binder drain down.

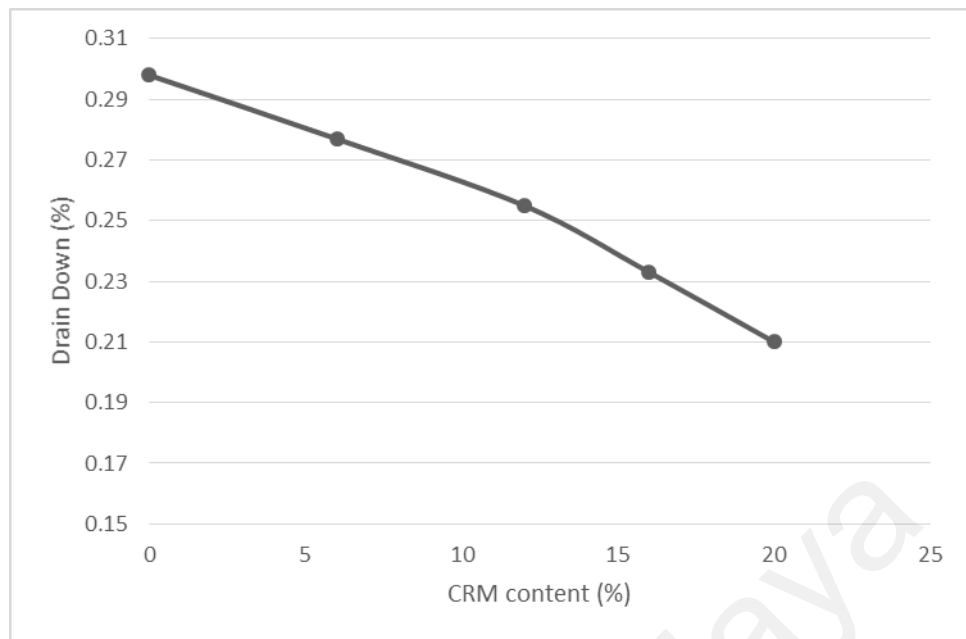


Figure 4.13 : Drain down test results

4.5.7 Moisture Susceptibility Test

“The moisture susceptibility of asphalt mixture is known as the vulnerability of the asphalt mixture to be damaged by water. When moisture collects within the mixture, it can cause damage to the bond between the aggregates and asphalt binder, which, in turn, accelerates the development of other kinds of distress and deformation, for instance, cracking and potholing”(Esmail *et al.*, 2012).

The results obtained from the tensile strength test of conventional mixture and the CRM-mixture is in Figures 4.14 and 4.15. Results illustrate the tensile strength and (TSR) ratio values of the mixtures decrease with the addition of CRM. Furthermore, TSR values between 70-80 % have been set as the minimum requirements by AASHTO T 283 and ASTM D 4867 standards. As Figure 4.15 shows, all values of TSR are above 70% showing that all mixes may have sufficient resistance against damage encouraged by moisture (Katman, 2006; Hamed, 2010).

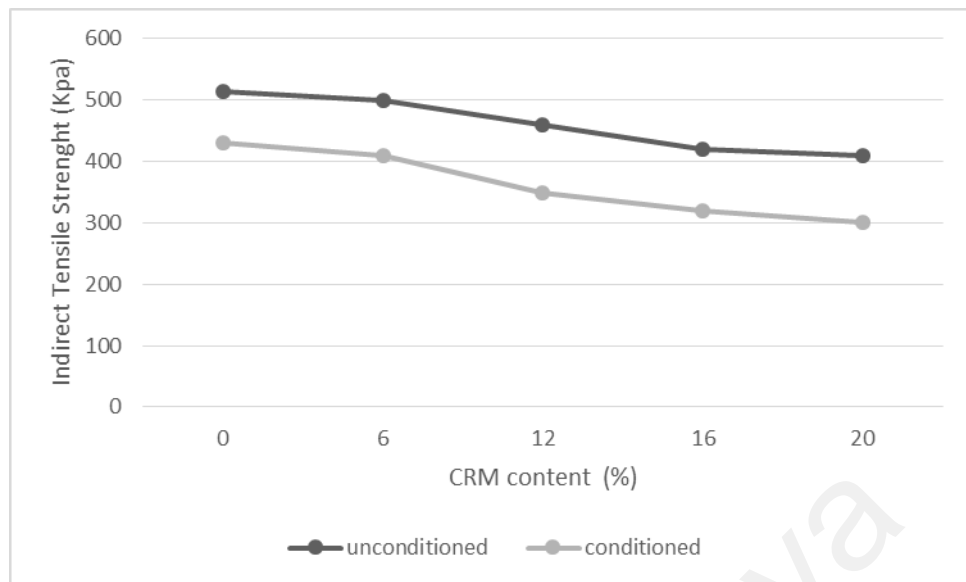


Figure 4.14: moisture susceptibility results

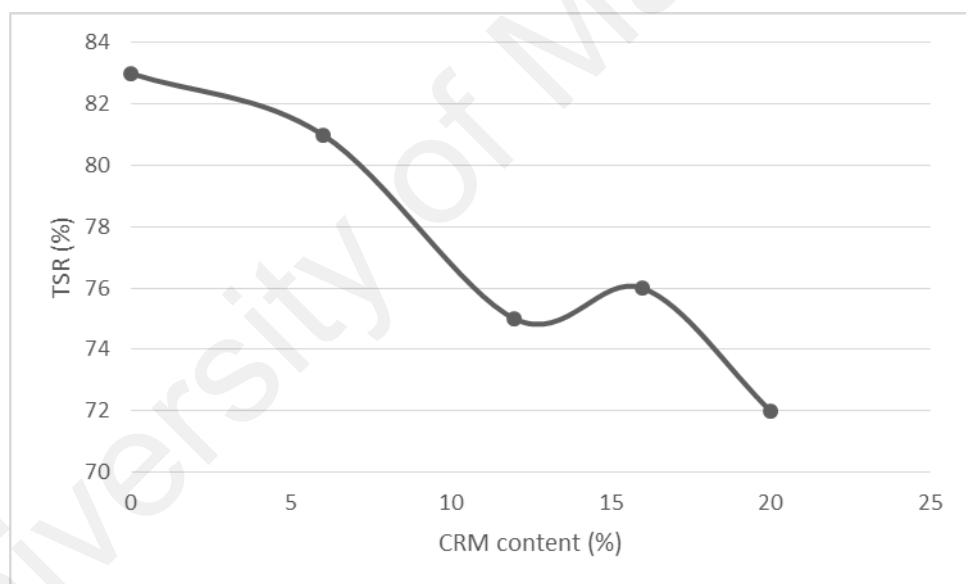


Figure 4.15: TSR results for various CRM content

4.6 Relationship Between Rutting, IDT and Creep

Figures 4.16 and 4.17 display the correlation between rut depth and resilient modulus and the permanent strain, respectively. The figures showed an acceptable linear relationship between the rut resistance and the tested data (resilient modulus and permanent strain), due to the addition of CRM to ordinary SMA mixes. The

correlation coefficient between the rutting resistance and resilient modulus (stiffness) is about 0.71, while a slightly higher correlation coefficient is about 0.98 between the rutting resistance and permanent strain (creep). The results indicated that the increase in stiffness properties of the samples due to the addition of CRM is followed by a decrease in rut depth as shown in Figure 4.16. Similarly an increase in permanent strain due to the addition of CRM of the samples is followed by an increase in rut depth as shown in Figure 4.17 from science point of view this is understandable. According to the previous studies (Mahrez, 2008; Hamed, 2010), indirect tensile test (IDT) was used to identify and evaluate the performance of asphalt mixture. Dynamic creep test was deemed a better alternative to be able to determine the creep deformation and quantify the permanent deformation of bituminous mixes. Further, the interpretation of the strain/time response of a material undergoing a creep test provides significant parameters, which describe the instantaneous elastic/plastic and viscoelastic/plastic components of the material response (Mashaan *et al.*, 2014c).

Permanent deformation or rutting takes place due to the complex stress-strain behaviour of the bituminous mix. Also, rut resistance is due to both cohesion and internal friction; where the significant of internal friction is contributing to better rut resistance; and mixes cohesion is the dominate factor in determining the rut resistance. By using the same concept resilient modulus and creep tests data can be related to cohesion and internal friction of asphalt thus to rutting resistance. The resilient modulus test being non-destructive test is in fact is a good indicator of mixture cohesion. On the other hand, creep test being a destructive test it better accounts for internal friction. In this study higher correlation coefficient was obtained between the rut depth and creep test as compared to resilient modulus, thus

dynamic creep test might be more reliable test in evaluating the rut resistance of asphalt mixture (Mashaan & Karim 2013c).

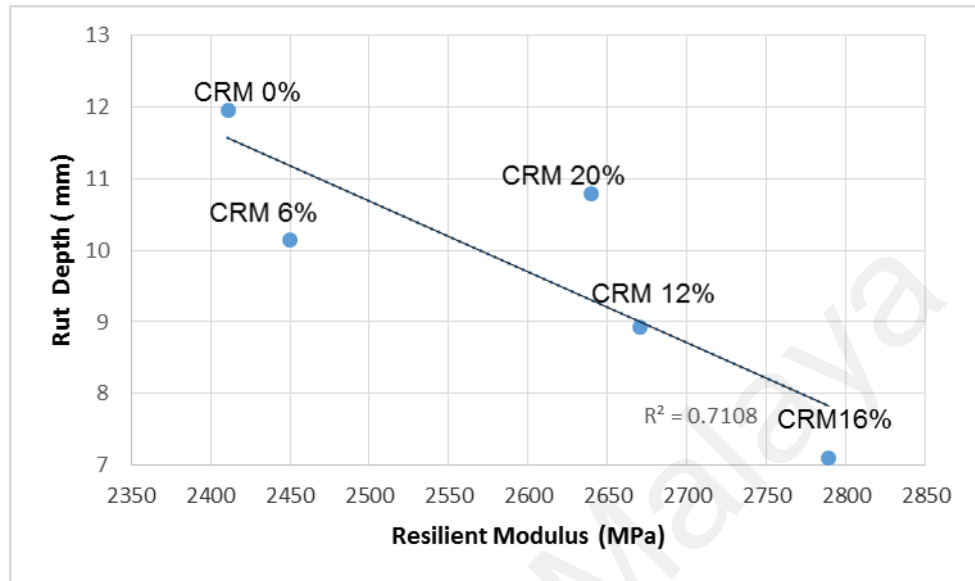


Figure 4.16: Relationship between rut depth and dynamic stiffness

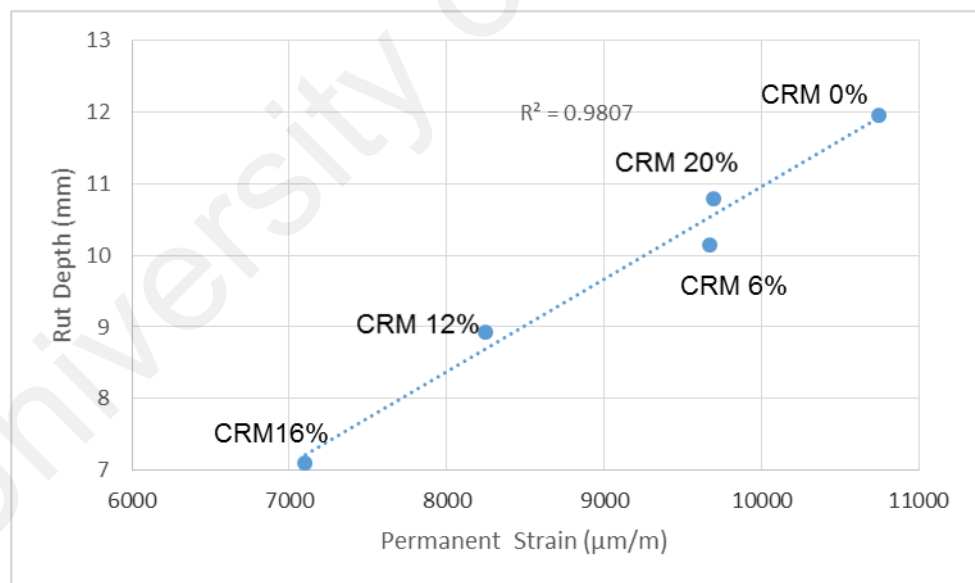


Figure 4.17: Relationship between rut depth and creep permanent strain

4.7 Relationship between Fatigue, Stiffness and Creep

Considerable effort and cost are required to measure the fatigue response of asphalt mixture using conventional laboratory procedure and to use the information so obtained to design fatigue resistance mixtures. The aim of these alternative procedure is to simplify both testing and analysis procedure to estimate the fatigue performance. Considered here in this section is an attempt to relate some of the conventional mix tests results (Dynamic creep and resilient modulus) to fatigue response.

Figures 4.18- 4.20 show fatigue accumulated strain versus creep permanent strain and stiffness modulus, respectively for the stress of 2000, 2500 and 3000 N used in this study. The figures showed that the fatigue accumulated strain of the specimen resulted in a proportional correlation with the creep permanent deformation of the mixes, whereas an inverse correlation was obtained between the fatigue and stiffness properties. In simple words the results indicate that the decrease in fatigue accumulated strain due to the addition of CRM resulted in a decrease of creep strain of the specimen and an increase in the stiffness properties. Table 4.6 shows a strong correlation coefficient between the fatigue strain and stiffness about $R^2 = 0.95$, 0.89 and 0.93 respectively.

Even though, fatigue test is considered a destructive test, it seems to correlate better with resilient modulus test which is non-destructive test as compared to the creep test which is destructive test. Resilient modulus test being non-destructive test is in fact a good indicator of mixture cohesion, on the other hand creep test being a destructive test seem to account better for differences in internal friction among asphalt mixture than mixture cohesion. Therefore, resilient modulus test might be

more reliable test in evaluating the fatigue performance of asphalt mixtures as compared to dynamic creep test, or simply the significance of mixture cohesion in contributing to the fatigue resistance of asphalt is more important than mixture internal friction.

Table 4.6: Regression coefficient between fatigue strain and creep strain and stiffness using SPSS analysis

Stresses (N)	Regression Coefficient (R ²)	
	Stiffness	Creep
2000	0.95	0.71
2500	0.89	0.67
3000	0.93	0.68

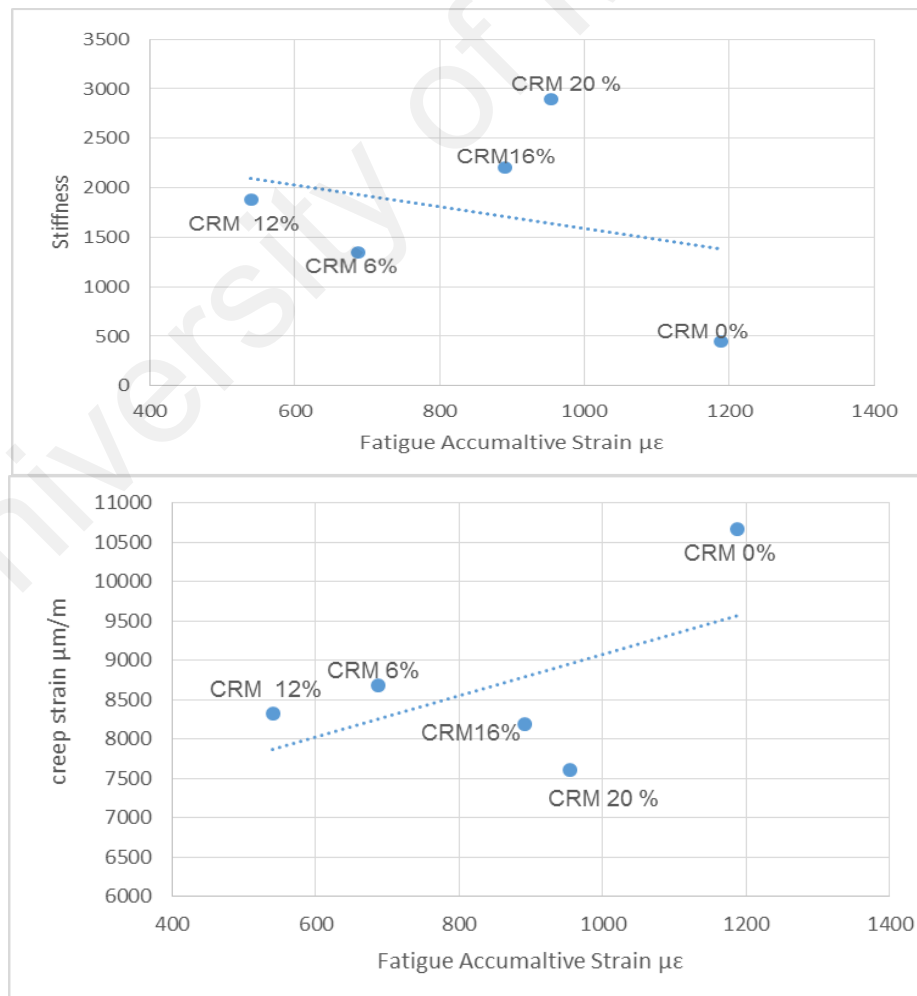


Figure 4.18: Fatigue & stiffness and creep at 2000 N stress

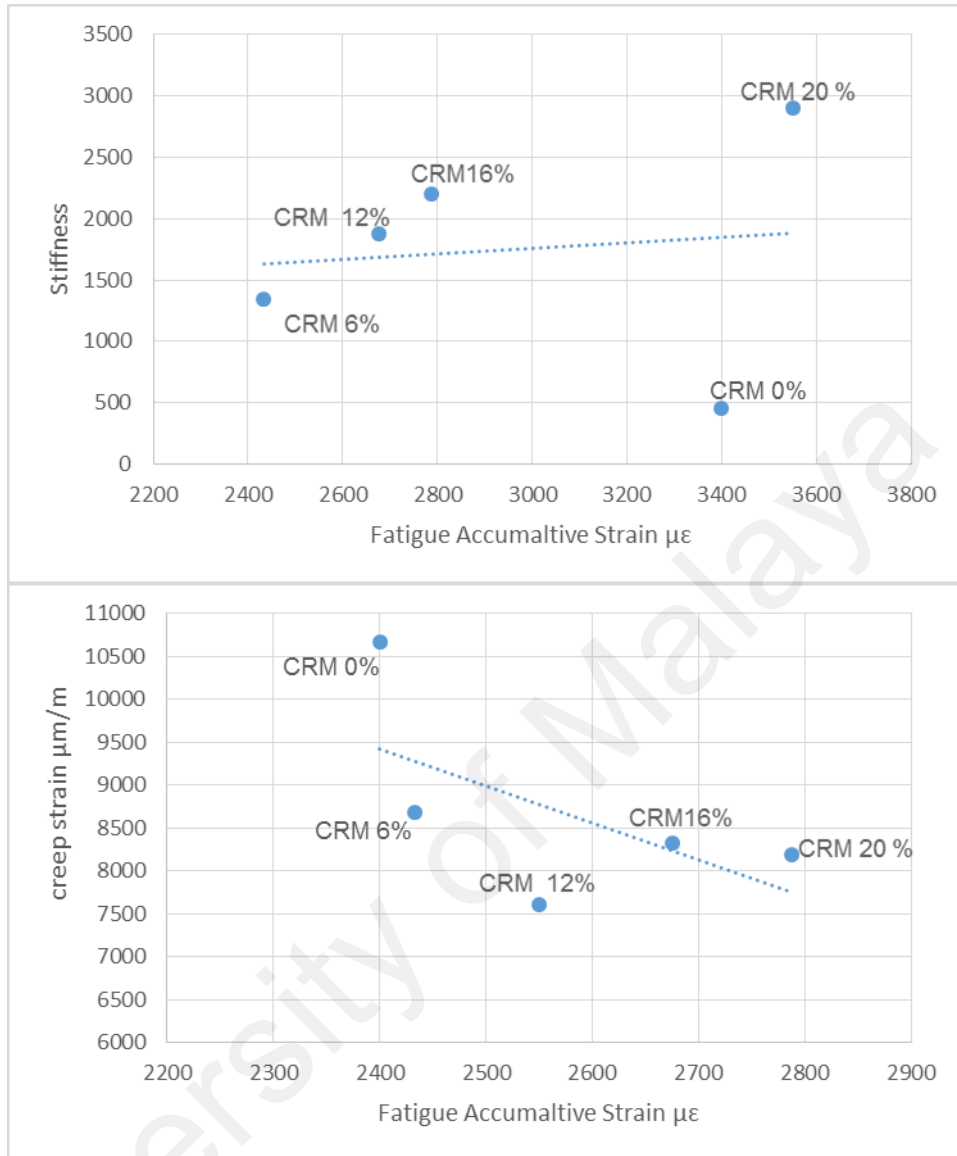


Figure 4.19: Fatigue & stiffness and creep at 2500 N stresses

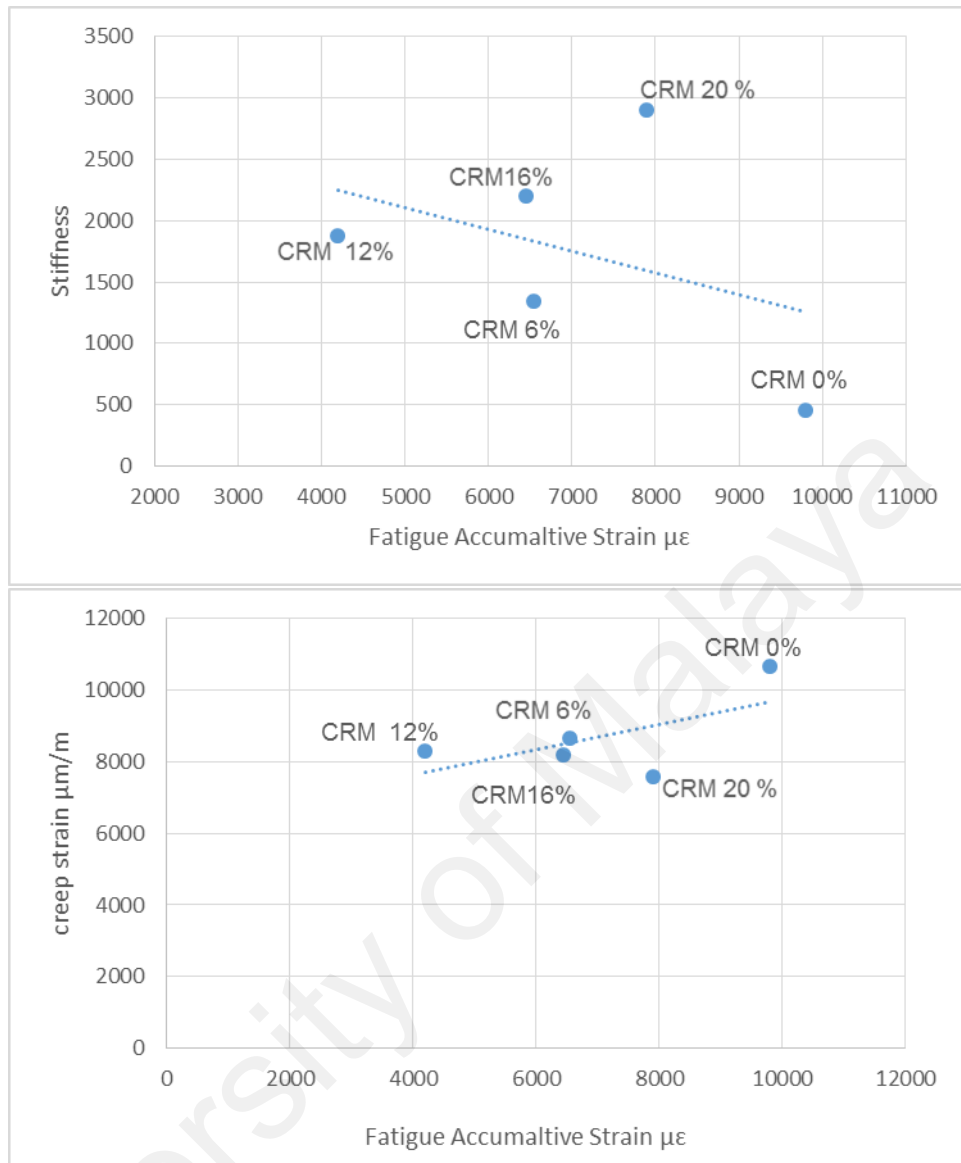


Figure 4.20: Fatigue & stiffness and creep at 3000 N stresses

4.8 ANFIS Model Analysis

4.8.1 Resilient Modulus

In this study, the ANFIS network was trained with data measured in the presented experimental procedure. 70% data was used for training and 30% data was used for testing. Three bell-shaped membership functions were used to fuzzify the ANFIS inputs. After the training process, the ANFIS network was tested to determine the resilient modulus. In this study, ANFIS was used for resilient modulus prediction based

on a series of measurements of CRM content in Stone Mastic Asphalt (SMA) mixtures. The statistical input parameters (CRM content and temperatures) for the data sets are calculated and presented in Table 4.7. °C

Table 4.7: Statistical parameters for data sets

Variable	Statistical parameters			
	Min	Max	Mean	Standard deviation
Temperature	25	40	32.50	7.628214416
CRM %	0	20	10	14.1421356237

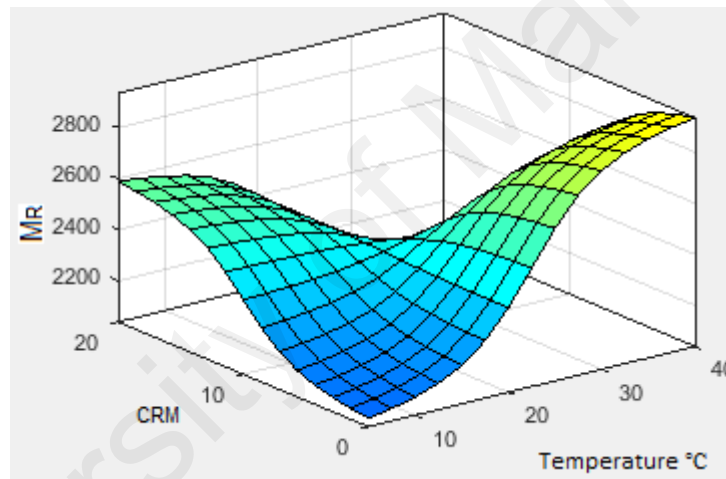


Figure 4.21: ANFIS decision service of resilient modulus results

Figure 4.21 shows ANFIS decision surfaces for input 1: environment temperature and input 2: CRM content. According to the experiments, the input parameters (temperature and CRM content) and output (resilient modulus) were collected and defined for the learning technique.

The experimental data of stiffness modulus and predicted values using ANFIS model are shown in Figure 4.22. As can be seen in Figure 4.22, R^2 correlation coefficient is very high. Therefore ANFIS has good correlation with the training data.

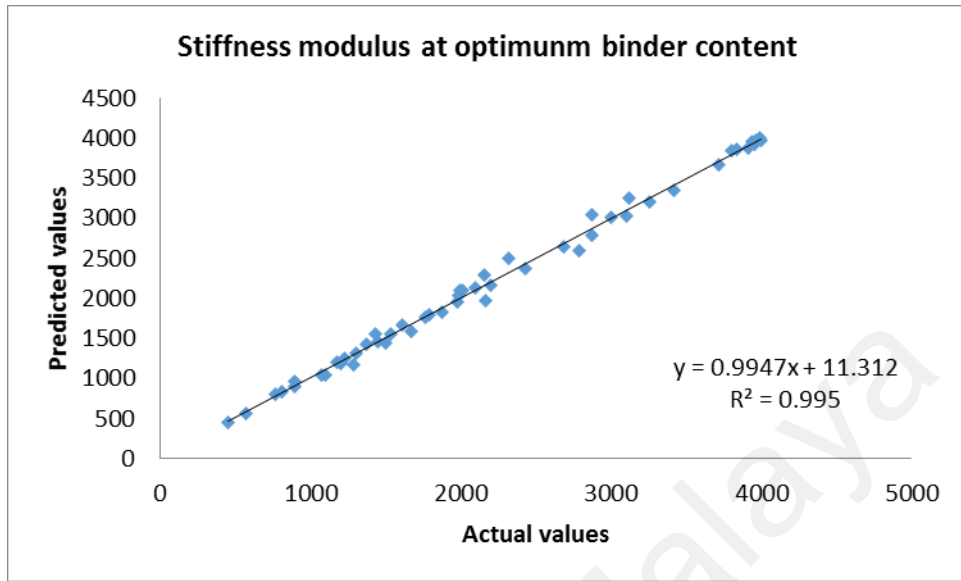


Figure 4.22: ANFIS model used to predict values of stiffness modulus

The performance of ANFIS model that estimated stiffness modulus was evaluated according to statistical criteria such as RMSE, Pearson correlation coefficient (r) and coefficient of determination R^2 . This confirms the RMSE statistics evaluated in Table 4.8. It should be concluded, that the proposed ANFIS model can be used for forecasting with high reliability.

Table 4.8: Performance statistics of the ANFIS model in stiffness modulus estimation

	RMSE	R^2
Stiffness modulus (IDT)	68.83785	0.995

4.8.2 Creep Test

The analysed of the ANFIS model for the rutting performance estimation of CRM reinforced- SMA mixtures based on the three inputs. ANFIS RMS error after training process for strain estimation was 0.056. The ANFIS decision surface for strain

estimation is shown in Figure 4.23 for combination of the temperature (input 1) and stress level (input 2) (Figure 4.23 (a)), temperature (input 1) and CRM (input 3) (Figure 4.23(b)) It is apparent from Figure 4.23 the amount of cumulative permanent strain (CPS) decreases by application of CRM which represents lower mixture susceptibility against permanent deformation, and it is more significant at higher CRM contents. It can also be realized that increment in stress level has a negative influence on the creep tests results and, hence, improve the rutting performance of asphalt mixture. A Cumulative permanent strain (CPS) values rise considerably when the amount of stress increases from 500 kPa to 700 kPa. It can be noticed from the results that different temperature has notable influence on the rutting performance of asphalt mixture. This indicates that the asphalt mixture is very susceptible against rutting damage at higher temperatures. The same thing can be found from Figure 4.24 and, moreover, it shows the effect of temperature is more drastic at higher stress levels. According to the experiments, the input parameters of temperature, stress and CRM content, and output (permanent defamtion) were collected and defined for the learning technique. The experimental data of permanent defamtion and predicted values using ANFIS model are shown in Figure 4.24. As can be seen in Figure 4.24, R^2 correlation coefficient is very high. Therefore ANFIS has good correlation with the training data. This confirms the RMSE statistics evaluated in Table 4.9. It should be concluded, that the proposed ANFIS model can be used for forecasting with high reliability.

Table 4.9: Performance statistics of the ANFIS model estimation

	RMSE	R^2
Cumulative permanent strain	48.57674341	0.9988

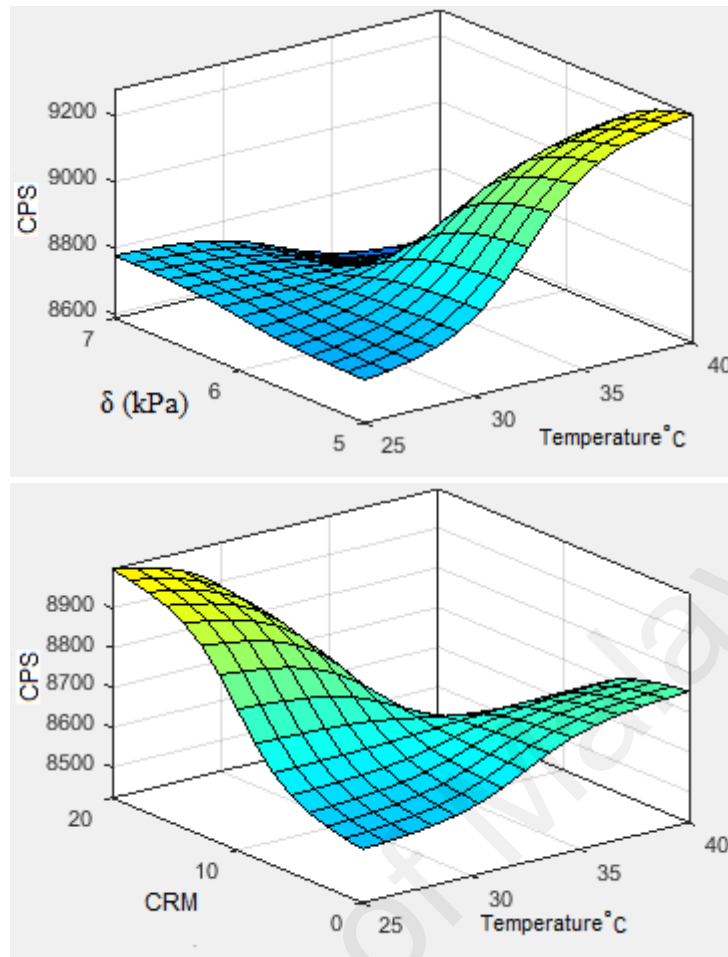


Figure 4.23: ANFIS decision surface for creep estimation

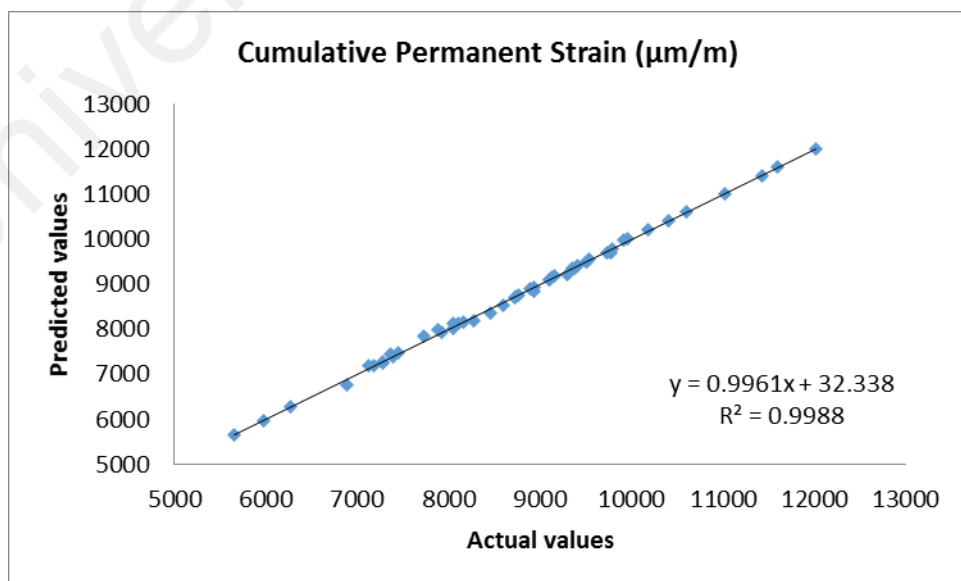


Figure 4.24 : ANFIS model used to predict values of creep test

4.8.3 Fatigue Test

Generally, the ANFIS network was trained with data measured in the presented experimental procedure. 70% data was used for training and 30% data was used for testing. Three bell-shaped membership functions were used to fuzzy the ANFIS inputs. After the training process, the ANFIS network was tested to determine the resilient modulus. In this study, ANFIS was used for fatigue life prediction based on a series of measurements of CRM content in Stone Mastic Asphalt (SMA) mixtures.

Figure 4.25 shows ANFIS decision surfaces for input 1: environment stress and input 2: CRM content. According to the experiments, the input parameters (stress and CRM content) and output (fatigue life) were collected and defined for the learning technique. The experimental data of stiffness modulus and predicted values using ANFIS model are shown in Figure 4.26. As can be seen in Figure 4.26, R^2 correlation coefficient is very high. Therefore ANFIS has good correlation with the training data.

The performance of ANFIS model that estimated stiffness modulus was evaluated according to statistical criteria such as RMSE, and coefficient of determination R^2 . This confirms the RMSE statistics evaluated in Table 4.10. It should be concluded, that the proposed ANFIS model can be used for forecasting with high reliability.

Table 4.10: Performance statistics of the ANFIS model in fatigue life estimation

	RMSE	R^2
Fatigue life	49.43170373	0.9878

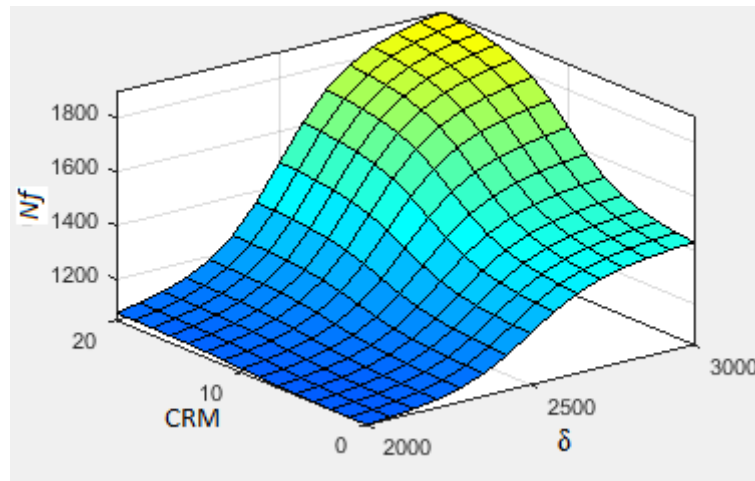


Figure 4.25: ANFS decision services for fatigue life

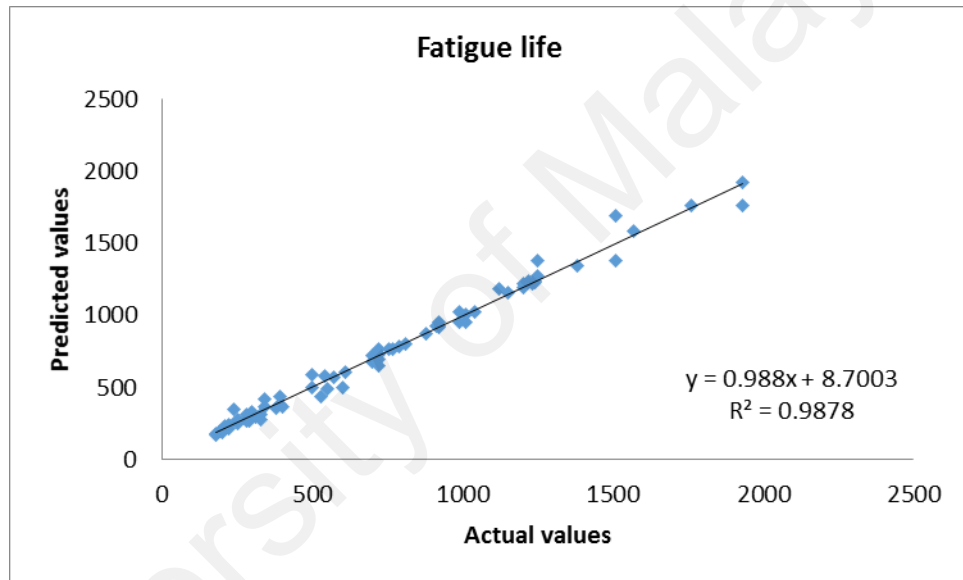


Figure 4.26 : ANFIS model using predicted values of fatigue life

4.9 Summary

This chapter focused on the laboratory test results on the SMA properties enriched with CRM as an additive. First, the discussion on the properties of materials used, and then the Marshall test results, and performance test were evaluated. According to the results obtained, the addition of CRM to the mixture has significant results in stability and stiffness performance. The estimation of stiffness, rutting and fatigue deformation showed that the predicting model of ANFIS can be used for forecasting with high reliability.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

In Malaysia, 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 cost of maintenance and rehabilitation 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 narrow range 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 CRM in reinforcing asphalt pavement has been introduced and evaluated thoroughly.

This current study was divided into several parts to determine the impact of incorporating waste tyre rubber, in the form of CRM, on the engineering properties of SMA. Based on the study conducted, the following conclusion can be derived:

- 1- Marshall stability and resilient modulus (stiffness) are improved by adding CRM to the stone mix asphalt as better adhesion is developed. This increased stiffness modulus however is not related to increased brittleness of reinforced asphalt samples. The stiffness modulus of reinforced samples is in fact less severely affected by the increased temperature compared to the non-reinforced samples. The optimum amount of the added CRM was found to be 12 by weight of bitumen.

- 2- With the presence of CRM, the fatigue life of reinforced samples is significantly improved. The resistance of CRM 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.
- 3- Based on test results the mixtures containing CRM resulted in higher resistance to permanent strain and higher resistance to rutting. It was obvious that the CRM content of 12 - 16% by weight of the total mix resulted in highest performance in terms of resistance to rutting deformation as compared to the ordinary mix.
- 4- As correlation between resilient modulus and permanent strain and rutting performance of CRM-reinforced-SMA mixture was obtained. The factor that probably influences the rutting performance could be attributed to the internal friction between the aggregate and mix cohesion. Dynamic creep test was found to be more reliable in evaluating the rutting resistance of asphalt. As for fatigue test, higher correlation coefficient was obtained between the fatigue life and resilient modulus as compared to permanent strain; therefore, dynamic stiffness seems to be more adequate in evaluating the fatigue life.
- 5- Generally, ANFIS method can estimate Marshall stability, Marshall flow, resilient modulus, creep and fatigue life of CRM modified mixture with high estimation accuracy. The ANFIS results were compared using root-mean-square error (RMSE) and coefficient of determination R^2 . The computing methodology

showed very good learning and prediction capabilities, and it is adaptable to optimization and adaptive techniques.

5.2 Recommendations

The aim of this study is to investigate the way in which waste tyre rubber improve the mechanical properties of the SMA mixture following the dry process. For contractors, industries and agencies looking for sustainable bituminous mix, results from this study further recommends the innovative use of CRM in SMA mix. For such mix design, the following might be offered and considered:

- 1- This study encouraged and recommended the use of CRM-SMA-dry process as an environmentally-friendly method for producing asphalt mixture wearing course with high resistance to structural failure in Malaysia. The design approach recommended in this study combines improve design development of mechanical properties in accordance with Malaysian Association for Road Work (JKR). The study recommended the optimum condition of CRM- SMA mix for field trial and application.
- 2- Based on Marshall test and stiffness results of the reinforced mix, its recommended to use binder content of 16 % for CRM-SMA dry mix design. Stiffness modulus of reinforced samples is in fact less severely affected by the increased temperature compared to the non - reinforced samples and, thus, it is recommended that the asphalt was not added to the mixer until all the CRM was homogenously dispersed throughout the aggregate.

- 3- The dynamic creep data recommended that at the optimum condition the addition of more than 16% CRM will cause minimal effect on resistance to rutting deformation. With further inclusion of CRM the strain value of the reinforced -SMA mix will increase higher as compared with that of the non-reinforced mix causing a detrimental effect to the reinforced mix by reducing its resistance to permanent deformation. Thus higher percentage of CRM might be not recommended as it required full understanding of the interaction mechanism of rubber- bitumen and aggregate mix.
- 4- The resilient modulus test being non-destructive test is in fact is a good indicator of mixture cohesion. This study recommended this non-destructive test as a significant indicator for both rutting and fatigue test.
- 5- This study recommended the use of ANFIS model for estimating the mechanical properties of SMA mix. The main advantages of the ANFIS model are: it is computationally efficient, and it is adaptable to optimization and adaptive techniques. In current study input data was CRM content, binder content and environmental conditions of temperature and stress. It recommended using other input, for future studies, such as modulus of elasticity and poisons ratio.

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Appendix A: Marshall Test Results

Table A.1 Marshall Stability

<i>Source of Variation</i>	<i>SS</i>	<i>df</i>	<i>MS</i>	<i>F</i>	<i>P-value</i>	<i>F crit</i>
Rows	276.0973	4	30.67748	4.01787	0.005826	2.456281
Columns	20530	5	10265	1.420796	0.2455	3.554557
Error	137.4347	20	7.635259			
Total	20943.53	29				

Table A.2 : Marshall Flow

<i>Source of Variation</i>	<i>SS</i>	<i>df</i>	<i>MS</i>	<i>F</i>	<i>P-value</i>	<i>F crit</i>
Rows	257.3657	4	28.59619	4.009082	0.005891	2.456281
Columns	21560.45	5	10780.23	1.348694	0.8893	3.554557
Error	128.3913	20	7.132852			
Total	21946.21	29				

Table A.3: Compacted Mix Density

<i>Source of Variation</i>	<i>SS</i>	<i>df</i>	<i>MS</i>	<i>F</i>	<i>P-value</i>	<i>F crit</i>
Rows	250.6257	4	27.8473	4.031577	0.005727	2.456281
Columns	20549.21	5	10274.61	14.875	1.03E-20	3.554557
Error	124.3313	20	6.907296			
Total	20924.17	29				

Table A.4: Void in Mix

<i>Source of Variation</i>	<i>SS</i>	<i>df</i>	<i>MS</i>	<i>F</i>	<i>P-value</i>	<i>F crit</i>
Rows	282.104	4	31.34489	4.01206	0.005869	2.456281
Columns	21578.7	5	10789.35	13.81007	0.00221	3.554557
Error	140.628	20	7.812667			
Total	22001.43	29				

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