Evaluation of Fatigue Resistance for Modified Asphalt Concrete Mixtures Based on Dissipated Energy Concept

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Bewertung der Ermüdungsresistenz von modifiziertem Asphaltbeton

mit dem Konzept der Dissipierten Energie

Vom Fachbereich Bauingenieurwesen und Geodäsie der Technischen Universität Darmstadt zur Erlangung des akademischen Grades eines Doktor-Ingenieurs (Dr. -Ing.) genehmigte Dissertation

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Abstract

The performance of asphalt concrete pavement depends on the bitumen properties, asphalt concrete mixtures volumetric properties and external factors such as traffic volume and environment. Bitumen is a visco-elastic material where temperature and rate of load application have a great influence on its behavior. Conventional bitumen is exposed to a wide range of loading and weather conditions; it is soft in a hot environment and brittle in cold weather. Higher traffic volume produces high stress within pavement layer, which is one of the main causes for pavement distress. Fatigue cracking and permanent deformation is considered as most serious distresses associated with flexible pavements. These distresses reduce the service life of the pavement and increase the maintenance cost. To reduce the pavement distresses there are different solutions such as adopting new mix design or by using asphalt additives. Using of asphalt additives in highway construction is known to give the conventional bitumen better engineering properties as well as it is helpful to extent the life span of asphalt concrete pavement.

In this research an investigation was made on the fundamental studies of modified asphalt binder and mixtures in order to understand the influence of modifiers on the rheological properties and fatigue resistance with the aim of preventing fatigue cracking in asphalt pavement. The conventional bitumen (70/100) penetration grade was used in this research, modified with crumb rubber (CR) and styrene-butadiene-styrene (SBS) at four different modification levels namely 3%, 5%, 7% and 10% by weight of the bitumen. The rheological properties and fatigue resistance tests for asphalt binder were performed using a dynamic shear rheometer apparatus. Fatigue life for asphalt binder and mixtures were calculated based on the dissipated energy concept as well as a procedure for modifying of conventional bitumen was developed to find the suitable blending time and the optimum modifier content.

From the results at low rubber content 3% and 5%, the behaviour of the modified bitumen remains close to that of the conventional bitumen and the optimum crumb rubber content for good rheological properties and long fatigue life was found to be 10% by the weight of bitumen. At higher (SBS) polymer content 7% and 10%, the behaviour of the modified binders remains close to that of the modified bitumen with 5% (SBS) and the optimum (SBS) content was found to be 5%. The fatigue behavior of modified bitumen was found to be significantly improved compared to conventional bitumen.

Fatigue test using dynamic shear rheometer was found to be costly and time consuming. 3D finite element model for dynamic shear rheometer has been developed and was used for dissipated energy calculation. The experimental result and the model result showed excellent fit between dissipated energy for the same tested bitumen. On the other hand, a shift factor was found between the dissipated energy per volume from the bitumen specimen in dynamic shear rheometer and dissipated energy per volume for asphalt concrete mixtures in indirect tensile fatigue test.

Abstrakt

Das Gebrauchsverhalten von Fahrbahnbefestigung aus Asphaltbeton wird von den Bitumeneingeschaften, den volumetrischen Eigenschaften des Asphaltmischguts und externen Einflüssen aus Verkehr und Umwelt. Bitumen ist ein viskoelastisches Material, dessen Verhalten stark beeinflusst wird von der Temperatur und die Frequenz der Lastaufbringung. Es ist einem großen Spektrum aus Belastung und Wetterbedingungen ausgesetzt. Bei hohen Temperaturen ist Bitumen weich und bei niedrigen Temperaturen spröde. Hohe Verkehrsbelastungen führen zu einer hohen Beanspruchung der Schichten der Fahrbahn, was einen der Hauptgründe für das Versagen von Straßenbefestigung darstellt. Ermüdungsrisse und bleibende Verformungen gelten als die am häufigsten eintretende Schäden flexibler Fahrbahnbefestigungen. In der Folge wird die Nutzungsdauer von Fahrbahnbefestigungen reduziert und die Erhaltungskosten steigen. Zur Reduzierung dieser Schäden existieren verschiedene Lösungsansätze wie die Anpassung der Asphaltmischgutzusammensetzung oder Verwendung von Additiven. Die Verwendung von Additiven im Asphaltstraßenbau ist bekannt sowohl zur Verbesserung der Eigenschaften von konventionellem Bitumen (Straßenbaubitumen) als auch zur Verlängerung der Nutzungsdauer der Fahrbahnbefestigung aus Asphaltbeton.

In dieser Untersuchung wurde eine grundlegende Betrachtung von modifiziertem Bitumen und Asphaltmischgut durchgeführt, um den Einfluss der Modifizierung auf die rheologischen Eigenschaften und Ermüdungsresistenz zu analysieren und in der Folge Ermüdungsrissbildung in Asphaltbefestigungen vermeiden zu können. Es wurde Straßenbaubitumen 70/100 eingesetzt, das mit jeweils 3 %, 5 %, 7 % und 10 % (bezogen auf das Bitumengewicht) Gummi (aus Altreifen) und dem Polymer Styrol-Butadien-Styrol (SBS) modifiziert wurde. Die rheologischen Eigenschaften und die Ermüdungsresistenz der Bitumen wurden mit dem Dynamischen Scher-Rheometer bestimmt. Die Ermüdungsdauer des Bitumens und der Asphaltmischung wurde über die Theorie der dissipierten Energie berechnet und eine Vorgehensweise zur Bestimmung der optimalen Mischzeit und der optimalen Modifizierungsmenge für Straßenbaubitumen entwickelt.

Die Ergebnisse der Bitumenuntersuchungen zeigen, dass die Eigenschaften des mit 3 % und 5 % Gummi modifizierten Bitumens vergleichbar zu den Eigenschaften des Straßenbaubitumens sind. Der optimale Gummianteil hinsichtlich der rheologischen Eigenschaften und der Ermüdungsdauer zeigte sich bei 10 %. Bei der Modifizierung des Bitumens mit 7 % und 10 % SBS waren die Eigenschaften vergleichbar zu den Eigenschaften des mit 5 % SBS modifizierten Bitumens, der optimale SBS-Gehalt wurde daher mit 5 % bestimmt. Das Ermüdungsverhalten der modifizierten Bitumen war gegenüber dem Straßenbaubitumen deutlich verbessert.

Ermüdungsuntersuchungen mit dem Dynamischen Scher-Rheometer sind kostenintensiv und zeitaufwändig. Daher wurde ein 3D-Finite-Element-Modell für das Dynamische Scher-Rheometer entwickelt und zur Berechnung der Dissipierten Energie verwendet. Die Versuchsergebnisse und die mit dem Modell berechneten Werte zeigen eine gute Übereinstimmung für die einzelnen Bitumen. Weiterhin wurde ein Verschiebungsfaktor festgestellt zwischen der Dissipierten Energie pro Volumen der Bitumenproben, die mit dem DSR bestimmt wurden, und der Disspierten Energie pro Volumen der Asphaltbetonmischung, die mit dem Spaltzugversuch bestimmt wurde.

This dissertation is dedicated to:

My parents and my family for their love,

My wife for her endless help and continuous support,

My daughters

Rofida,

Yomna,

Jody

for their sweet smile that give me energy to work.

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M.Sc. Eng. Farag Khodary

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Abbreviations and Symbols

AASHTO	American Association of State Highways and Transportation Officials, Washington
ASTM	American Society of Testing Materials
FHWA	Federal Highway Administration
HMA	Hot Mix Asphalt
HPDIM	Highway Pavement Distress Identification Manual
M-E	Mechanical –Empirical design approach for asphalt concrete mixtures.
NCHRP	National Cooperation Highway Research Program
SHRP	Strategic Highway Research Program
TUD	Technical university Darmstadt
ZTVT-StB 95	Zusätzliche Technische Vertragsbedingungen und Richtlinien für Tragschichten im Straßenbau (Specification for Unbound Granular Materials used in Pavement Constructions -Bundesministerium für Verkehr, Bonn, 1995.
DSR	Dynamic Shear Rheometer
IDT	Indirect Tensile Tester
IDFT	Indirect Tension Fatigue Test
CR	Crumb Rubber
CRMM	Crumb Rubber Modified Mixtures
CRMB	Crumb Rubber Modified Bitumen
SBS	Styrene-Butadiene-Styrene
SBS PMB	Styrene-Butadiene-Styrene Polymer Modified Bitumen
SBS PMM	Styrene-Butadiene-Styrene Polymer Modified Mixtures
RBTM	Recommended Blending Time Minutes
MBTM	Maximum Blending Time Minutes
RBTD	Recommended Blending Temperatures Degree
MBTD	Maximum Blending Temperatures Degree
DER	Dissipated Energy Ratio
DSCT	Disk-Shaped Compact Tension Test
SBR	Styrene-Butadiene- Rubber

τ_{max}	Absolute value of the peak-to-peak shear stress $[Pa]$
γ_{max}	Absolute value of the peak-to-peak shear strain $[\%]$
h	Specimen height $[mm]$
E	Young's modulus
G*	Complex shear modulus $[Pa]$
G'	Storage modulus $[Pa]$
G''	Loss Modulus $[Pa]$
δ	Phase angle degree
N_F	Number of cycles to failure
ε_o	Initial strain.
σ_o	Initial stress
S_o	Mixtures stiffness
wi	Dissipated energy at load cycle
da/dN	Incremental change in a crack length
ΔK	Stress intensity factor
T_{max}	Maximum applied torque
θ_{max}	Maximum deflection angle $[rad]$
υ	Poisson's ratio
Δ h	Recoverable Horizontal Deformation
VMA	Volume of voids in mineral aggregate
Va	Air Voids in Compacted Mixture, Percent of Total Volume
N	Newton
^oC	Degree Celsius
Wt %	By The Weight of Bitumen
Pa	Pascal

Chapter 1

Introduction

In highway construction engineers must consider the primary user requirements of safety and economy and it is essential to construct asphalt pavement that remains with acceptable service level for maximum possible time. In order to achieve highway construction requirements pavement designer should take into account environmental factors, traffic flow and asphalt concrete mixtures materials. It is believed that the properties of the designed mixtures play an important role in controlling pavement distresses. There are different types of distresses appear in asphalt concrete pavement such as moisture damage, rutting and fatigue cracking. Pavement performance is greatly affected by the bitumen properties; it is known that conventional bitumen has a limited range of rheological properties and durability that are not sufficient to resist pavement distresses. Therefore, asphalt researchers looking for different types of bitumen with excellent rheological properties, which directly affect asphalt pavement performance.

1.0 Problem Statement

Over the past 25 years, Egyptian Government had invested a huge sum of money in the field of highway construction to reach excellent pavement performance. However, these roads show early signs of distress such as rutting and fatigue cracking. The pavement distress is due to change in weather and high traffic loads. Environmental condition and heavy loads affect directly the durability and pavement performance. Therefore, pavement distress needs urgent solutions that become necessary and does not accept the delay. On other hand, the weakness of highway networks affects directly the national economy. There are numerous studies introduce pavement distress causes and solution. Terrel, R. L (1971) reported that cracking can be assumed as one of the major pavement distresses and it occurs due to different reasons. Fatigue is the process of cumulative damage resulting from repeated traffic loading.

This type of distress occurs at moderate to low temperatures under repetitive traffic loading and it occurs over the long term of pavement life, but once it is initiated, it progresses rapidly and leads to a total structural collapse. Miller and Bellinger (2003) presented pavement distresses and failure mechanisms in Highway Pavement Distress Identification Manual (HPDIM). To minimize asphalt concrete pavements distress there are several ways, which could extent pavement service life:

- Produce a new binder type with better physical, chemical and rheological properties.
- Improve the pavements and mix design.
- Improve the construction methods and maintenance techniques.

Modified bitumen is assumed to be one of the most important solutions for pavement distress. To produce modified bitumen there are several methods by using different materials at different modification level. In recent years, using polymer is considered to be a common method in asphalt concrete pavement industry that can be used to improve bitumen properties. Modified bitumen provides the diversified properties needed to build better performing roads. Addition of polymer to asphalt cement is the most important form of modification due to its wide range of application and potential for use. Nowadays, polymer technology is considered as a permanent part of the highway construction. On the other hand, using crumb rubber from scrap tiers as asphalt modifier helps to solve serious environmental problems and to improve the pavement performance.

Commercial polymer offer the possibility to produce mixtures that can resist both rutting and cracking. Using polymer modifier for a highway construction project depends on many factors such as cost, construction ability, availability, and expected performance. Polymer modification especially in developing countries is more expensive since the polymer is imported from foreign countries. To reduce the cost of highway construction and maintenance asphalt researcher look for alternative materials such as scrap tires. Scrap tires are waste materials, which contribute to be one of the most serious environmental problems. Large numbers of scrap tries are thrown away daily which affects the soil and ground water. Therefore, using such waste materials are not only reducing the cost but also to keep the environmental clean and help to achieve the natural balance. Bahia (1995) reported that using commercial polymer improves the pavement performance and at the same time increases the construction cost. Studying the effect of different types of modifiers to improve fatigue resistance of asphalt pavement is a field of interest for many asphalt researchers, but most efforts concentrated on the fatigue resistance for asphalt concrete mixtures. On the other hand, the fact that fatigue is a phenomenon in asphalt binder should not be ignored. Therefore it is appropriate to measure fatigue of asphalt binders and to correlate it with the mixtures fatigue. However, several critical questions associated with modifiers remain to be answered.

- To what extent using waste materials as asphalt modifiers improve the rheological properties of bitumen?
- To what extent using waste materials as modifiers improve fatigue resistance comparing to commercial polymer?

For this purpose two experimental tasks are have been used for better understanding of the rheological properties of modified bitumen and to the influence of improvement in bitumen rheology on fatigue resistance of asphalt mixtures. The first task is fatigue properties of conventional bitumen and modified one using dynamic shear rheometer (DSR) based on dissipated energy concept. The second task is fatigue resistance for asphalt concrete mixtures, manufactured with the same bitumen using indirect tensile fatigue test based on dissipated energy concept. A better understanding of the rheological properties binders strengthens the ability to produce durable asphalt concrete pavements and to increase pavement life.

1.1 Research Objectives

The aim of this work is to provide excellent rheological and physical properties for modified bitumen and mixtures. Waste materials such as scrap tires are creating costly disposal problem and using these materials was proven to be economical, environmentally sound and effective to improve the pavement performance. The main objective of this research is to study the influence of waste materials scrap tires (Crumb rubber) on rheological properties and fatigue resistance of asphalt mixtures, and to compare them with the commercial polymer such as styrene-butadiene-styrene (SBS) in order to understand fatigue mechanism with the aim of preventing fatigue cracking in asphalt pavement. Therefore to achieve the main aim of the study, the following objectives tasks were performed:

1. Review the literature on the effect of waste materials and commercial polymer on the rheological and fatigue resistance of modified bitumen.
2. Identify the Rheological characteristics of binders that have the greatest influence on fatigue behavior using a dynamic shear rheometer (DSR) apparatus in wide range of temperature and frequency.

3. Quantify the effect of using SBS and CR as compared to conventional mixtures in terms of increasing pavement fatigue life.
4. Correlate the dissipated energy per volume for asphalt binder with the dissipated energy per volume for asphalt concrete mixtures that can be used to predict fatigue life for asphalt concrete pavement.

1.2 Research Methodology

The research methodology for this study involved the following major tasks:
Literature Review, experimental design and materials selection, laboratory testing program, laboratory test data analysis, comparison and evaluation of the fatigue life for modified and unmodified asphalt binder and mixtures based on dissipated energy concept. Finally conclusions and recommendations for future work are presented.

1.3 Scope of the Study

The study work investigates different areas including asphalt additives as polymer technology characterization. Real comparison between waste materials and commercial polymer from the point of it is rheological behaviour and fatigue resistance. The rheological and fatigue tests were undertaken using a dynamic shear rheometer (DSR) apparatus based on the fundamental of dissipated energy approaches. Three-dimensional finite element model for dynamic shear rheometer (DSR) were used to calculate the dissipated energy for viscoelastic materials. The rheological properties of modified binders help to appreciate the main advantages and disadvantages of using modifiers in asphalt pavement industries.

1.4 Thesis Layout

The work is organized in seven Chapters. Chapter 1 is an introductory chapter outlining the problem statement and the objectives of the research work. The scope of the study is clearly stated in this chapter as well as a layout of the thesis.

Chapter 2 provides an extensive literature review beginning with an introduction and brief summary regarding to composition and chemistry of asphalt cements. An overview is given for using polymer as asphalt modifier in the filed of highway construction as well as using waste materials. Evaluation of the rheological and visco- elastic properties of asphalt binder using dynamic mechanical analysis as well as characterization of the mechanical properties of asphalt concrete mixtures using fundamental tests are presented in this chapter.

Asphalt rheology is the main theme in Chapter 3. Experimental measurements to characterize the rheological behavior of modified and unmodified bitumen, evaluate the fatigue resistance and investigate the effect of modifier content and type, on the viscoelastic properties using mechanical analysis based on fundamental dissipated energy concept is outlined. As well as procedure for mixing modifier with base bitumen were presented. Layout of laboratory testing program are presented in figure (1-1)

Chapter 4: describes the rheology and fatigue laboratory test results for modified and unmodified asphalt binder.

Chapter 5: includes the experimental design for the asphalt concrete mixtures as well as material properties for the bitumen and aggregates. The methods used for asphalt concrete mixtures fabrication is outlined, and the experimental measurements used to characterize the mixtures in terms of fatigue resistance are also discussed.

Chapter 6: in this chapter the experimental result for asphalt concrete mixtures presented with evaluation of the effects of polymer types and polymer content on fatigue resistance. A summary of the discussion is given at the end of the chapter.

Chapter 7: includes a list of conclusions as results of the research work as well as recommendations for future work.

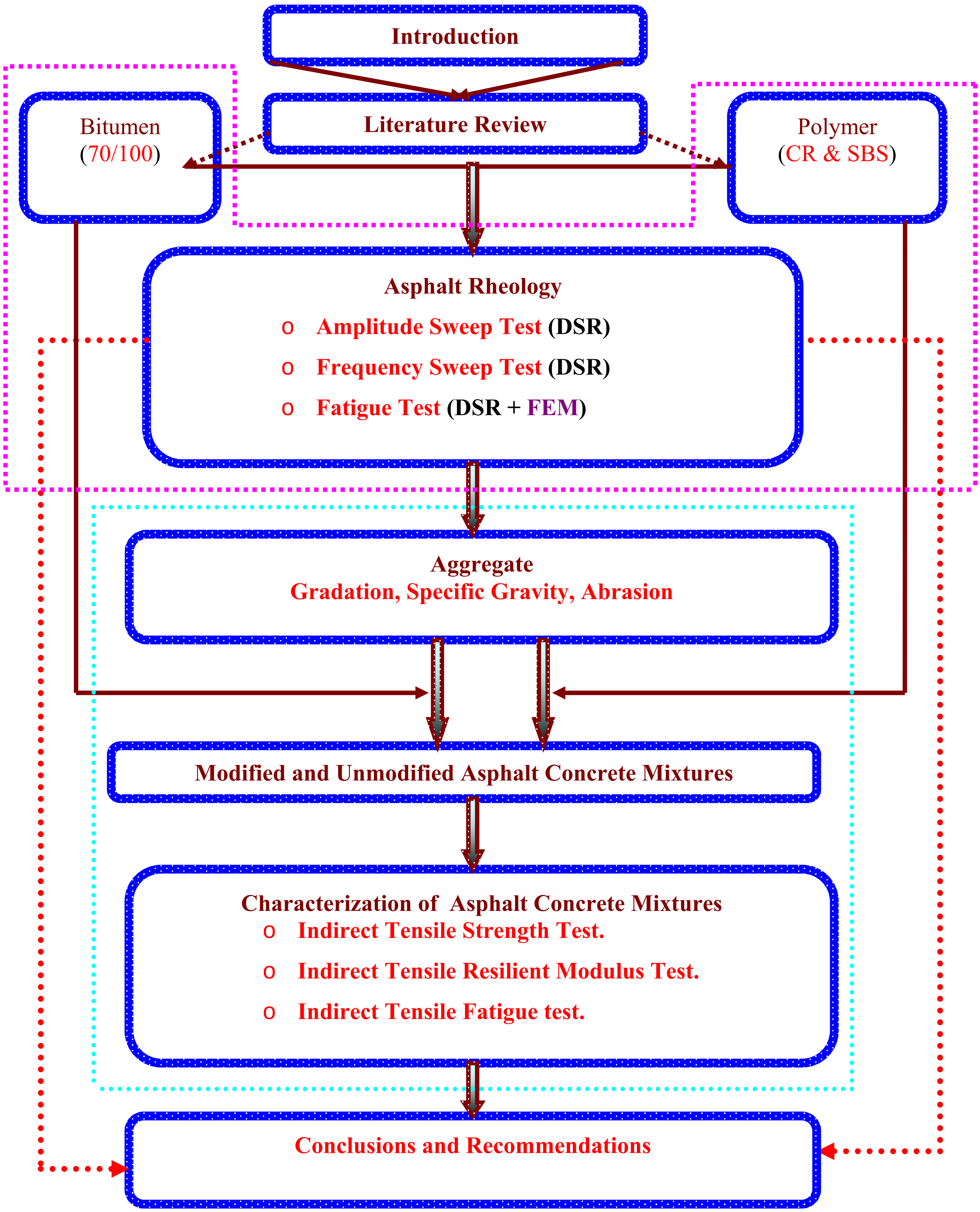


Figure (1-1): Layout of laboratory testing program.

Chapter 2

Literature Review

2. Introduction

The purpose of this chapter is to review the literature of using polymer technology in asphalt concrete pavement industry. This literature review consists of five parts; the first part describes the chemical composition and viscoelastic properties of bitumen's material. The main topic in the second part is to classify the common flexible pavement distresses. Type of asphalt additives and its importance as well as the fundamental rheological properties of bitumen are presented in the next two parts. Last part describes the mechanistic approaches used to evaluate asphalt concrete mixtures properties.

2.1 Asphalt Concrete Pavement

Asphalt pavements are designed to resist rutting, fatigue, low temperatures cracking and other distresses. The most serious distresses associated with flexible pavement are cracking, which occurs at intermediate and low temperatures, and permanent deformation, which occurs at high temperatures. These distresses reduce the services life of the pavement and increase the maintenance costs. In recent years, there is a rapid increase in using additives in asphalt concrete mixtures to improve its properties. Current research is focused on increasing the fatigue resistance of asphalt concrete mixtures. Polymer modification is suggested to improve the fatigue resistance of asphalt binder and mixtures.

Asphalt cement binds the aggregate particles together, enhancing the stability of the mixture and providing resistance to deformation under induced tensile, compressive and shear stresses. The performance of asphalt mixture is a function of asphalt cement, aggregate and its volumetric properties. Bitumen is the main component, which controls the viscoelastic properties during production in the plant and service on road. Bitumen's materials are viscoelastic material and their mechanical behavior is dependent on both the temperature and rate of loading. At low temperatures and short loading times asphalt cements behave as elastic solids, while at high temperatures and long loading times they behave as simple viscous liquids. At intermediate temperatures and loading times, the behavior is more complex. The relationship between modified asphalt binders and field pavement performance is still being researched because many modified binders are rheologically complex However, it is clear that asphalt binder and concrete modification is an effective method for preventing pavement distress (King et al., 1999).

Polymer is a common method used to modify bitumen and addition of polymers has gained popularity in recent years. This is because modification provides the diversified properties needed to build better performing roads. Polymeric modifiers have been introduced as potential source of specific improvements in the characteristics of asphalt binder and mixtures. The main reasons that asphalt modification has become more accepted are the traffic factors, which have increased including heavier loads, higher volumes and higher tire pressures. In order to understand effect polymer modification on pavement performance, one must understand the nature of bitumen's materials.

2.2 Elementary Analysis of the Bituminous Materials

The bituminous materials were defined as dark brown to black cementations material which is composed principally of high molecular weight hydrocarbons. Cementations material is in the bottom of the vacuum distillation columns in the crude oil refineries. Bitumen is considered to make up of asphaltenes, resins, and oils. Elementary analysis of the bitumen manufactured from a variety of crude oils shows that most bitumen contains carbon, hydrogen, sulphur, oxygen, and nitrogen (Whiteoak C. D., 1990). The elementary analysis of the bitumen is presented in table (2-1).

Table (2-1) Elementary analysis of the bituminous materials

Component	Percentage %
Carbon	80 - 88%
Hydrogen	8 - 11%
Sulphur	0 - 6%,
Oxygen	0 - 1.5%
Nitrogen	0 - 1%

2.3 Chemical Groups of Bituminous Materials

It is widely acknowledged that the chemical composition of bitumen has a large influence on the performance of bitumen. Robert N. H. et al (2000) explained that the bitumen consists of two chemical groups called asphaltenes and maltenes. The maltenes can be further

subdivided into three small groups saturates, aromatics and resins. The main Chemical component groups for bitumen are presented in figure (2-1).

Bitumen = Asphaltenes + Maltenes (Saturates + Aromatics + Resins)

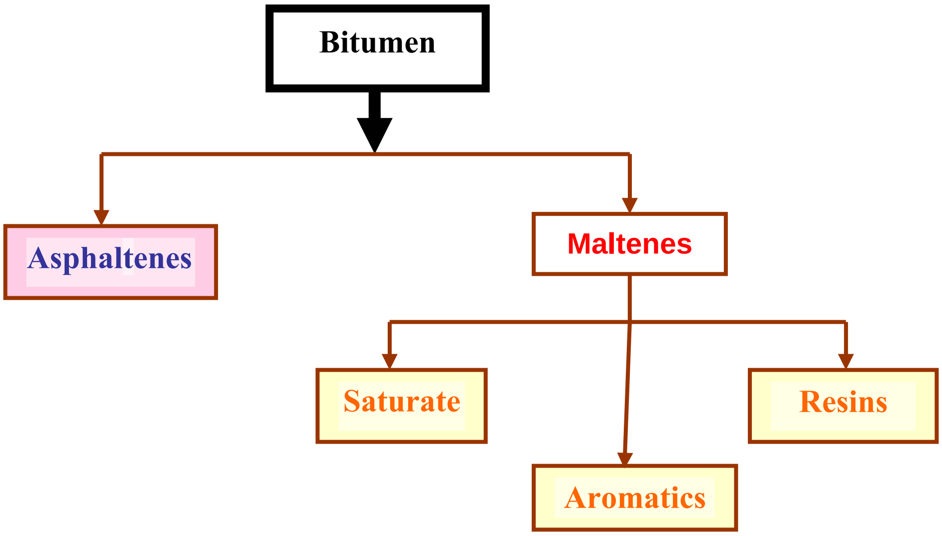


Figure (2.1): Chemical component groups for bitumen.

2.3.1 Aphaltenes

Robert N. H., (2000) defined asphaltenes as highly polar, complex aromatic materials, having high molecular weight more than Maltenes. The asphaltene content of bitumen may range between 5% and 25% and has significant effect on the over all properties of the bitumen. Bituminous Materials with high asphaltene content will have higher softening points, higher viscosities and lower penetrations than those with low asphaltene contents. Increasing the asphaltene content and reducing the maltene content of bitumen will result in harder bitumen.

2.3.2 Maltenes

As described before maltenes is divided into three groups, which give the bitumen its characteristic. "Saturates comprise straight and branched-chain aliphatic hydrocarbons together with alkylnaphthenes and some alkyl-aromatics. The components include both waxy and non-waxy saturates and form 5% to 20% of the bitumen (Shell Bitumen Handbook, 1990)". Robert N. H, (2000) explained that the polar nature of the aromatics gives the bitumen its viscosity as well as its fluidity. The polar nature of the resins gives the bitumen its adhesive properties and they act as dispersing agents for the asphaltenes. Resins provide adhesion properties and ductility for the bituminous materials.

2.4 Viscoelastic Properties of bituminous material

Viscoelastic material defined as material, which store and dissipate mechanical energy in response by a mechanical stress. Robert N. H., (2000) described that the asphalt cement is a viscoelastic materials and its mechanical behaviour depend on both the temperature and the duration of loading. At low temperatures and short loading times asphalt cements behave as elastic solids, while at high temperatures and long loading times they behave as simple viscous liquids. At intermediate temperatures and loading times, the behaviour is more complex. The response of elastic, viscous and viscoelastic material under constant stress loading are presented in figure (2-2). Figure 2-2(a) shows a constant load is applied to an elastic material, the strain of the material is proportional to the applied stress and when the applied stress is removed from the material, there is a complete recovery to the original position. Figure 2-2(b) describes the behavior of a viscous material in which the strain of the material increases over time under constant stress. Figure 2.2 (c) demonstrates the behavior of a viscoelastic material in which a constant stress increases the strain over a long time and when the applied stress is removed, the material fails to attain its original position leading to permanent deformation.

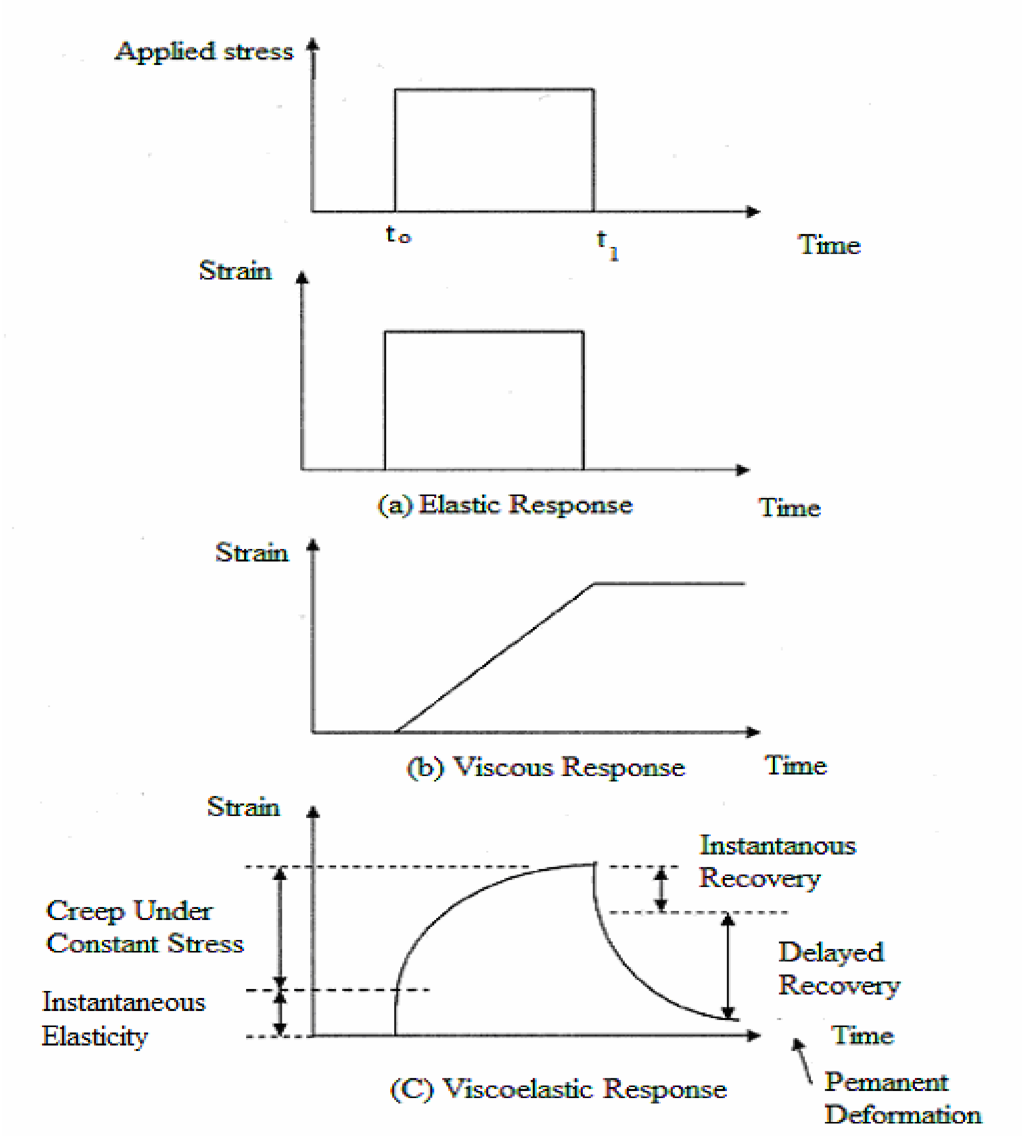


Figure (2-2) Idealized response of elastic, viscous and viscoelastic material under constant stress loading (Van der Poel, 1954).

2.5 Stress and Strain within Flexible Pavement

Flexible pavements are defined as asphalt bound layers built over a granular base that rests on natural road bed soil. These types of pavements are called flexible because the total pavement structure bends or deflects due to traffic loads. A flexible pavement structure is generally composed of several layers of materials which can accommodate this flexing. Generally, pavement structure is divided into three layers namely: bituminous surfacing (surface course), road base (base course) and sub-base (AASHTO, 2002). Asphalt concrete mixtures should have high stiffness to be able to resist permanent deformation. On the other hand, the mixtures should have enough tensile stress at the bottom of asphalt layer to resist fatigue cracking after many load applications. Figure (2-3) presented the orientation of principal stresses with respect to position of rolling wheel load.

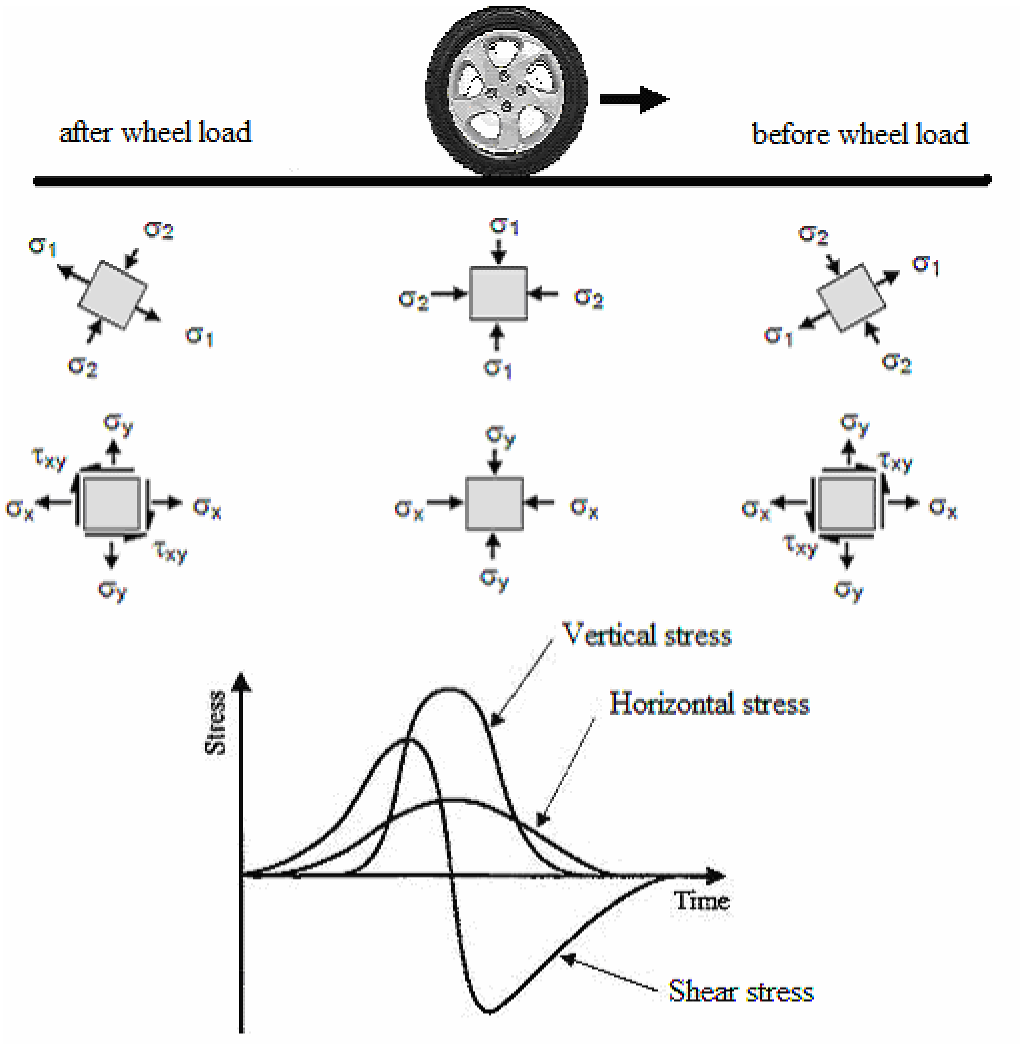


Figure (2-3) Stress beneath a rolling wheel load after (Shaw, 1980)

2.6 Asphalt Pavement Distresses

Terrel. (1971) mentioned that to improve the fatigue performance of asphalt pavements it is important to acquire a better understanding of the cracking mechanism of asphalt pavements. Fatigue failure is the result of flexural cracking of asphalt bound layer and there are a lot f factors affect the fatigue mechanism such temperature, loading rate and aging. The complex interaction of these variables leads to use advanced mechanics theories such as viscoelasticity, damage mechanics, and fracture mechanics in order to understand the failure mechanisms. Monismith., et al. (1985) divided the structural damage in flexible pavements appears in two main forms: cracking and deformation, both of which are due to load repetitions or adverse environmental conditions. Fatigue cracking has been known to decrease the service life of pavement and lead to pavement structure collapse. Different factors affect pavement performance and lead to pavement distress such as magnitude and frequency of loads density, duration of load cycle and variation of temperatures.

Lytton et al. (1994) described fatigue cracking stages with two stage process the first is crack initiation and the second is crack propagation. Crack initiation can be described as a process by which initial cracks on the asphalt pavement. Crack propagation can be described as the period for the dominant cracks to grow and further develop to form a critical size leading to pavement distress. The subject of pavement distress and failure is considered complex as several factors contribute to the pavement deterioration and failure. At high temperatures under traffic loading the asphalt is not able to maintain the original shape of the pavement, which lead to permanent deformation, know as rutting. At low temperatures the asphalt gets brittle and tends to crack because the stiffer structure is unable to relax the internal stresses originating from traffic load (AASHTO, 2002). The main general causes of the pavement failure are:

- Defects in the quality of materials used.
- Defects in the construction method and quality control during construction.
- Surface and subsurface drainage.
- Increase in the magnitude of the wheel loads and the number of load repetition due to increase in the traffic volume.
- Settlement of the pavement foundation
- Environmental factors including heavy rainfall, snow, frost action and high water content.

Asphalt pavement distresses are categorized into three main types cracking, surface deformation, and surface defects. There are many reasons for asphalt concrete damage such as mixtures disintegration, fracture and viscoplastic flow. The classifications of asphalt pavement distress according to (Miller and Bellinger, 2003) are presented in table (2-1).

Table (2-1): Common flexible pavement distresses (Miller and Bellinger, 2003)

Category	Distress Type
Cracking	Longitudinal, Fatigue, Transverse, Reflective, Block, Edge
Deformation	Rutting, Corrugation, Shoving, Depression, Overlay Bumps
Surface defects	Potholes, Patching, Ravelling, Stripping

Fatigue cracking is one of the main modes of asphalt pavement deterioration caused by traffic and environmental factors. Bahia. H. (2006) explained that fatigue cracking of flexible pavements is based on the horizontal tensile strain at the bottom of asphalt concrete layer. In the stage of crack initiation water trapped in the cracks and this led to reduction of the materials strength under repeated loading. Due to the strength reduction crack start to propagate and lead to pavement collapse.

Figure (2-4): Flexible pavement distresses fatigue cracking (Bahia, 2006)

2.7 Asphalt Additives

A conventional bituminous material does not have the performance requirements for the road construction, which are increasingly subjected to heavy loads, heavy traffic and several environmental conditions. When the produced asphalt does not meet climate, traffic, and pavement structure requirements, modification has been used as one of the attractive alternatives to improve its properties. Modification offers one solution to overcome the pavement distress deficiencies of bitumen and thereby improve the performance of asphalt concrete pavement. Isacsson U. (1995) reported that using of polymer modified bitumen's to achieve better asphalt pavement performance has been observed for a long time. The main objective of the bitumen improvement is to produce ideal modified bitumen's materials with high resistance to permanent deformation, and fatigue cracking.

2.7.1 The Need of asphalt additives

There are many researchers looking for the reasons to modify bituminous materials. Lewandowski, L.H. (1994) mentioned that the main reasons to modify bituminous materials with different type of additives could be summarized as follows:

- To obtain softer blends at low service temperatures and reduce cracking,
- To reach stiffer blends at high temperatures and reduce rutting,
- To increase the stability and the strength of mixtures,
- To improve fatigue resistance of blends,
- To reduce structural thickness of pavements.

King, et al. (1986) defined asphalt modifier as material, which would normally be added to the binder or the mixtures to improve its properties. The choice of modifier for a particular project can depend on many factors including construction ability, availability, cost, and expected performance. Roberts et al. (1991) described that the technical reasons for using modifiers in asphalt concrete mixtures are to produce stiffer mixes at high service temperature to resist rutting as well as to obtain softer mixtures at low temperature to minimize thermal creaking and improve fatigue resistance of asphalt pavement. Improvement in the performance of asphalt concrete mixtures that contain polymer is largely due to the improvement in the rheological properties of the asphalt binder. The rheological properties of a binder that allow flexibility under load controls resistance to fatigue. The modified mixtures are less brittle at lower temperatures and it has higher stiffness at higher temperatures compared to normal mixtures. This makes polymer modification extremely attractive for pavement designers and highway agencies.

Epps, Jon A (1994) explained that asphalt exposed to a wide range of load and weather conditions, however, does not have good engineering properties, because it is soft in a hot environment and brittle in cold weather. Therefore, asphalt is usually reinforced by polymers to improve its mechanical properties. The main advantage of using modified bitumen is the effect on the pavement performance in terms of permanent deformation, fatigue cracking, and moisture susceptibility. The stiffer asphalt concrete mixture is considered to be more resistance resistant to permanent deformation. Brule (1996) mentioned that Polymer modification increases binder stiffness and elasticity at high service temperatures and low loading frequencies with the degree of modification being a function of bitumen source, bitumen–polymer compatibility and polymer concentration.

2.7.2 Classification of Asphalt Additives

Srivastava, et al. (1992) presented clear definition for polymer as long chemical chains that are made up of many smaller chemicals (monomers) that are joined together. Polymer is divided to two main basic type elastomers and plastomers that used to modify bitumen for road applications. Plastomers modify bitumen by forming a tough, rigid, three-dimensional network to resist deformation, while elastomers have a characteristically high elastic response and, therefore, resist permanent deformation by stretching and recovering their initial shape.King et al. (1986) reported that Elastomers exhibit increased in tensile strength with elongation and have the ability to recover to the initial condition after an applied load is removed. Bahia, et al. (1997) conducted research program and classify asphalt modifier, according to their composition. Asphalt modifiers are divided to different main groups such as polymers (elastomeric and plastomeric), fillers, fibers, hydrocarbons, antistripping agents, and crumb rubber. These additives vary significantly in their physical and chemical characteristics and are expected to have widely variable effects on asphalt concrete pavement performance. Asphalt additives increase the stiffness of the mix at higher temperatures, decrease the stiffness at lower temperatures and increase the elasticity in the medium range temperatures.

Table (2-2): Types of asphalt additives after (Giavarini, 1998)

Type of Modifier	Purpose	Example
Filler	-Fill voids - Increase stability - Improve bond between aggregate and binder	- Lime - Portland Cement - Fly Ash
Elastomers	- Increase stiffness at higher temperatures. - Increase elasticity at medium range temperatures to resist fatigue cracking. - Decrease stiffness at lower temperatures to resist thermal cracking.	-Natural rubber -Styrene-butadiene-styrene (SBS) -Crumb rubber (TR) -Styrene-butadiene rubber (SBR).
Fiber	-Improves tensile strength -Improve cohesion -Allow for higher asphalt content without drain down	- Asbestos - Polyester - Fiberglass
Plastomers (Thermoplastics)	- Increase high temperature performance - Increase structural strength - Increase resistance to rutting	-Polyvinyl chloride (PVC) -Ethyl-vinyl-acetate (EVA) -Ethylene propylene (EPDM)
Waste Materials	- Replace aggregate with a cheaper product	-Recycled tires

2.7.3 Benefits of Using Asphalt Additives

There are two methods to modify bitumen properties. The first method is to stiffen the bitumen so that the total visco-elastic response of the asphalt is reduced. The second option is to increase the elastic component of the bitumen, which reduces the viscose component of the bitumen and directly affect the pavement performance. Modified asphalt mixtures were observed to be stiffer, more resistant to permanent deformation, and had higher resistance to fatigue cracking (Whiteoak C. D., 1990). The area of asphalt additives is a somewhat complex, that the improvement in the pavement performance is related to the binder rheology and depend on the modifier type with respect to polymer content. Bahia, (1995) studied the effect of polymer modification using scanning electron microscope images. The result showed that the modified asphalt concrete mixtures have better binder-aggregate adhesion, which led to increase in its toughness. Polymer modification affects the binder's flexibility that leads to fatigue resistance and increases the viscosity of the asphalt binder, which improve the tensile, and the compressive strengths of the mixtures. The role of modified bitumen is to increase the resistance of asphalt to permanent deformation at high temperatures.

The main advantage of using polymer technology is to improve the adhesion properties between the binders and aggregate. Khattak and Baladi (2001) conducted a research to study the influence using polymer as modifier that can make the bitumen more resistant to loading and less susceptible to temperature variations. In addition, some polymers improve adhesion of the bitumen to the stones, and improve the resistance to cracking. On the other hand, an ideal asphalt binder should have excellent cohesion and adhesion. The properties of modified bitumen depend on the modifier type with respect to modifier content and bitumen type. The main advantage of elastomers such as (SBR) and (SBS) is that they can provide a higher strength to the modified bitumen or mixtures (King et al., 1999).

It was illustrated in a number of studies that the use of crumb rubber modified binders in paving mixes enhances fatigue life of pavements. (Bahia and Davies, 1994; Bahia, 1995) studied the effect of crumb rubber on asphalt concrete mixtures properties. Rubberized mixtures have higher increase of resistance rutting more than the unmodified asphalt concrete mixtures. Two type of blending process are used wet process or dry process. The wet process consists to mix rubber to asphalt before adding the aggregates, while the dry process replaces some of the aggregate in the asphalt mixture. Raad., et al. (1993) studied that the use of crumb rubber modified binders in asphalt concrete mixtures enhances fatigue life of pavements.

Fatigue damage is a distress mechanism observed in asphalt pavement, particularly at moderate to low temperatures. Preliminary studies have shown that unmodified asphalt is sensitive to fatigue and the use of asphalt additives can dramatically improve the binder's response to fatigue. Green and Tolonen (1977) reported that using crumb rubber in flexible pavements need a better understanding of its effects on the physical, chemical and rheological properties of crumb rubber modified bitumen binders. The percentage of improvement in the properties of crumb rubber modified binders depends on the interaction between crumb rubber and bitumen. Hanson, D.I., et al. (1995) mentioned that crumb rubber can be used as asphalt modifier to minimize waste tires pollution and improve properties of asphalt mixtures. Based on results of rutting tests and indirect tensile tests, the addition of recycled tire rubber in asphalt mixtures could improve the engineering properties of asphalt mixtures.

2.8 Rheological Properties and Fatigue Resistance of Bituminous Material

Bituminous material deforms when subjected to loads and the properties of bituminous material change with change of temperatures during day and night. It has been well established that the rheological properties of the bitumen binder affect the asphalt pavement performance. Vinogradov, et al. (1980) defined rheology scince as a part of continuum mechanics and it is the study of material deformation. Rheology is the description of the mechanical properties for different materials under various deformation conditions. Bahia and Davies (1994) used the rheological properties as indicator for the pavement performance, at high temperature the rheological properties are related to the rutting performance of pavements. The rheology at intermediate temperatures impacts on the fatigue cracking of pavements. The low temperature properties of the binder are related to the low-temperature thermal cracking of the pavement. Reduced rutting, improved fatigue life, and lower low-temperature stiffness values have been measured in asphalt mixtures made with binders with improved rheological properties. Anderson DA, et al (1994) mentioned that the properties of asphalt binder play an important role in asphalt concrete pavement performance. There are many asphalt pavement distresses, which are believed to be related to the rheological properties of asphalt binder. The fundamental rheological characterisation of the modified and unmodified asphalt binder can be used to predict asphalt pavement performance.

The rheological properties of asphalt binder can be evaluated useing dynamic shear rheometer (DSR) apparatus. Different tests can be used to characterize the viscous and elastic behavior of asphalt binder at high and intermediate service temperatures.

Airey, G.D (1997) used dynamic shear rheomter apparatus to evaluate the bituminous materials properties. The result of these tests can be used to evaluate the specimen's response to the sinusoidal stresses and calculates several parameters of the bitumen sample, such as complex shear modulus, dynamic viscosity, phase angle and accumulated strain.

Bahia and Anderson (1995) present a description of the purpose and scope of the dynamic shear rheometer test. The dynamic shear rheometer (DSR) used to characterize the viscoelastic behavior of bituminous material at intermediate and high service temperatures. Stress-strain behavior defines the response of materials to load. Asphalt binder's exhibit aspects of both elastic and viscous behaviors; hence they are called viscoelastic materials. Bahia et al. (1993) conducted a time sweep test using dynamic shear rheometer. The test provides a simple method of applying repeated cycling of stress or strain loading at selected temperatures and loading frequency. The initial data collected were very promising and showed that the time sweeps are effective in measuring binder damage behavior under repeated loading in shear. The advantage of time sweep test that can be used to calculate fatigue life of asphalt binder based in dissipated energy approaches.

2.9 Asphalt Mixtures Characterization

Different testes and approaches were used to evaluate asphalt concrete mixtures properties. Several materials properties can be obtained from fundamental and mechanistic tests that can be used as input parameter for asphalt concrete performance model. Hadley, W. (1970) evaluated the properties of asphalt concrete mixtures using the indirect tensile test. The main terms, which can be characterized using indirect tensile test, are resilient elastic properties, fatigue cracking and the properties related to permanent deformation. The elastic stiffness of the asphalt concrete mixtures can be measured using the indirect tensile test.

2.9.1 Indirect Tensile Strength Test

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

Kim (2003) evaluated conventional and crumb rubber modified asphalt mixtures using indirect tensile strength tests and indirect tensile resilient modulus (M_R) tests. Othman. A. et al (2007) conducted a research to evaluate the performance of Polyethylene (PE) modified asphaltic mixtures based on physical and mechanical properties. Physical properties were evaluated in terms of penetration and softening point. On the other hand, the mechanical properties were evaluated based on the indirect tensile strength. The result presented that Polyethylene enhance both physical and mechanical properties of modified binder and mixtures.

2.9.2 Resilient Modulus Test

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

2.9.3 Indirect Tensile Fatigue Test

There are different test methods used throughout the world to measure fatigue resistance for asphalt concrete mixtures. Pell and Cooper (1975) mentioned that there are three main methods used to evaluate and predict the fatigue characteristics of asphalt mixes. They are initial strain – fatigue life, dissipated energy – fatigue life and fracture mechanics. Read et al. (1996) used the indirect tension fatigue test to evaluate the fatigue life of asphalt concrete mixtures. The horizontal deformation during the indirect tension fatigue test is recorded as a function of load cycle. The test specimen is subjected to a different level of stress so that a regression analysis on a range of values 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. Kim (2003) reported that fatigue cracking is a pavement distress that typically occurs at intermediate temperatures. Due to this fact, 20 °C was chosen as test temperature to characterize the fatigue lives of asphalt concrete mixtures. The testing frequency had chosen to be 10Hz, which is approximately equivalent to a vehicle speed of 50 mph. Raad and Saboundjian (1998) studied the fatigue resistance for cylindrical specimen with a compressive load, which acts parallel to and along the vertical diametric plane. This loading configuration develops a reasonably uniform tensile stress in the specimen perpendicular to the direction of the applied load and along the vertical diametric plane.

2.10 Fatigue Resistance Evaluation Approaches

It has been generally accepted that fatigue is a process of cumulative damage and one of the major causes of cracking in asphalt concrete pavement. The traditional fatigue approach assumes that damage occurs in a specimen from dynamic repetitive loading that leads to fatigue failure of the specimen. The number of load to failure equal to the fatigue life can be calculated based on can be based on stress, strain or energy.

2.10.1 Fatigue Approach Based on Stress or Strain

Monismith et al. (1985) mention that the fatigue characteristics of asphalt mixtures can be expressed as relationships between the initial stress or strain and the number of load repetitions. Different test methods were used to determine the fatigue life of asphalt concrete mixtures using repeated flexure, direct tension, or diametral tests performed at several stress or strain levels.

Khattak and Baladi (2001) reported that two types of controlled loading can be applied: control stress and control strain. In the control stress test, the stress remains constant but the strain increases with the number of repetitions. In the control strain test, the strain is kept constant, and the load or stress is decreased with the number of repetitions. The use of constant stress has the further advantage that failure occurs more quickly and can be more easily defined. The relationship between the number of cycles to failure and initial stress or strain can be presented using the following equation:

$$N_f = a\left(\frac{1}{\varepsilon_o}\right)^b * \left(\frac{1}{S_o}\right)^c \tag{2.2}$$

$$N_f = d\left(\frac{1}{\sigma_o}\right)^e * \left(\frac{1}{S_o}\right)^f \tag{2.3}$$

Where:

N_F : Number of cycles to failure.

ε_o :Initial strain.

σ_o : Initial stress.

S_o : Mixtures stiffness.

a, b, c, d, e, f : experimentally determined coefficients.

2.10.2 Fatigue Approach Based on Energy

During dynamic-load repetition, amount of energy is carried into the material. Part of this energy is stored in the material and dissipated when the load is released. Ghuzlan, K.A. (2001) reported that the energy approaches can be used to predict the fatigue behavior of the asphalt mixtures. Fatigue damage is related to the amount of energy dissipated in the specimen during testing. Several researches have also used energy-dependent models for predicting the fatigue behavior of asphalt mixtures. This is considered appropriate for asphalt mixtures, as the dissipated energy can be used to explain the decrease in mechanical properties, such as flexural stiffness, during testing.

Cheng, (2002) explained fatigue damage in viscoelastic materials can be due to stored and dissipated energies. The energy balance is influenced by rheological properties of the mix and the binder, which is in turn, functions of temperature, frequency and loading. Development and accumulation of damage is evaluated in terms of dissipated energy and number of cycles. The initial phase angles between stress and strain waveforms are indicative of the viscous or elastic nature of the material. During a dynamic bending test in controlled stress or strain dissipated energy per cycle per volume change due to the change in the mix behaviour and damage accumulation. The dissipated energy per unit volume per cycle for viscoelastic material is given by the following equation:

$$W_i = \pi \, \sigma_i \, \varepsilon_i \sin \delta_i \tag{2.4}$$

Where

Wi : Dissipated energy at load cycle i ,

σi : Stress amplitude at load cycle i,

εi : Strain amplitude at load cycle i, and

δ : Phase angle between stress and strain wave signals, degree.

Dissipated energy versus the number of cycles to failure could be characterized as follows:

$$N_f = K_1 \left(\frac{1}{W_i} \right)^{K_2} \tag{2.5}$$

Where:

Nf : Number of load application to failure

Wi : Dissipate energy

K1, K2 : Experimentally determined coefficients

2.10.3 Fatigue Approach based on Fracture Mechanics

Facture mechanics tests measure the energy required to break the mechanically loaded asphalt concrete mixtures specimen. Two important parameters can be obtained from these Facture mechanics test namely fracture energy and fracture toughness. In fracture mechanics, fatigue is considered to develop progressively through the three phases of crack initiation, stable crack growth and unstable crack propagation. Marasteanu et al. (2004) investigated several different fracture mechanics based test methods. The three major fracture test geometries used in this study were the Disk-Shaped Compact Tension Test (DSCT), Semicircular Bending Test and a bending beam test.

Paris law of crack propagation relates the increase in crack length per load cycle to the stress intensity factor; ΔK. A logarithmic linear relationship between the incremental change in a crack length (da/dN) and the amplitude of the crack driving force has been widely used. Walubita F. L. (2006) mentioned that Paris and Erdogan (1963) developed a crack rate law for use in linear elastic homogeneous materials. Paris' Law is defined as:

$$\left(\frac{da}{dN}\right) = c * (\Delta K)^{n} \tag{2.6}$$

Where

a : Crack length,

N : Number of load repetitions,

K : Stress intensity factor,

c, n : Empirical material parameters.

Chapter 3

Asphalt Rheology

3. Introduction

Fatigue cracking, due to repeated traffic load, is the most common mode of failure for asphalt pavements. As it is well known fatigue distress is mainly related to the rheological properties of asphalt binder. To understand the influence of asphalt binder on fatigue behavior of mixture, two experimental investigations were carried out with evaluatation to the effects of cyclic loading on the rheological and mechanical properties of asphalt binder and mixture. In the first phase from the experimental study, modified bitumen was produced by mixing base bitumen with two types of modifier commercial polymer and crumb rubber. The major objectives of the first phase are to characterize the rheological behavior of modified and unmodified bitumen, to evaluate the fatigue resistance and to investigate the effect of modifier content and type, on the viscoelastic properties using mechanical analysis based on fundamental dissipated energy concept. The second phase is to evaluate the mechanical properties and fatigue resistance of modified and unmodified asphalt concrete mixtures. Procedure for mixing modifier with base bitumen will be described later in this chapter. In the literature there are different studies, which present how the rheological properties for asphalt binder can be evaluated under wide range of load and frequency by means of dynamic mechanical analysis using a dynamic shear rheometer (DSR).

3.1 Evaluation of Bitumen Properties

Goodrich J.L. (1998) reported that bituminous material is considered as a viscoelastic material and their performance depend on time or rate of loading as well as temperatures. There are two types of tests that can be used to evaluate bitumen properties; conventional physical testing methods and fundamental rheological tests. Using conventional physical testing methods at specific temperature, which give only one parameter, for example designer would specify a stiffer binder to reduce the rutting problems in asphalt concrete pavements. This may be lead to crack at low temperature.

The empirical tests to evaluate the bitumen properties fail to characterise the performance of polymer modified bitumen. The performance for modified bitumen can be characterised by means of fundamental rheological characterisation because the tests can be done under wide range of temperature and frequency. The importance of rheology in the field of bitumen properties evaluation is presented in this chapter.

Bahia, et al. (1993) presented in the research conducted for the Strategic Highway Research Program (SHRP), a new testing method to characterize the rheological, failure, and durability properties of asphalt binders based on the rheological properties. The research results were discussed in four main points: (A) The viscoelastic nature of asphalts and its relation to pavement performance; (B) the types of conventional measurements that are used now and the fundamental problems related to these tests; (C) the concept behind selecting the new test methods and the new characteristic properties; and (D) how the new measured properties compare to the conventional properties.

Using rheological properties as performance parameter have advantages and disadvantage. The advantage of this technique is that it allows measurement of physical properties at high and low frequency with wide temperature range and this is likely to be experienced in the field due to traffic. Dynamic shear rheometer need qualified person with high experience in the field of dynamic tests to get good rheological results. In this chapter a brief description of the dynamic shear rheometer apparatus as well as specimen fabrication and specimen dimension will be presented. Finally all rheological test procedures adopted for the materials characterization are listed in details in this chapter.

3.2 Dynamic Shear Rheometer

Dynamic shear rheometer was used to measure visco-elastic properties, fatigue and rutting resistance at high and intermediate temperature. DSR measures both viscosity and the elastic properties of the asphalt binders. It is also defined as a binder characterization procedure and used to determine the fail temperatures of asphalt binders. This device used to make dynamic oscillatory load, where sinusoidal shear stress or strain is applied in the form of sinusoidal time function. DSR device was utilized to measure different binder properties before and after using the modification. Test results at intermediate and high temperatures, which can be used to predict resistance to rutting and fatigue cracking in asphalt concrete pavements. The test can be done when the bitumen's sample is sandwiched between two parallel plats and then the rheological parameter is recorded. DSR method contains many difficult details that must be carefully considered in order to obtain reproducible results. Dynamic shear Rheometer is reported to evaluate the specimen's response to the dynamic load. The dynamic load can be presented as sinusoidal time function, which is given as presented in the following equation.

$$\tau = \tau_o \, \mathrm{Sin}(\omega T) \tag{3-1}$$

Goodrich (1988) studied the effect of temperatures and frequency on the stiffness and viscosity of the bitumen's tested sample. The test simulates the shearing action of traffic at a certain speed and determines two important parameters used to predict pavement performance. Rheological parameter are divided into terms, the first one is the complex shear modulus (G*) and the second one is the phase angle (δ). Clear definition for the rheological parameter are presented and described in detail in the following sections. Final conclusion about the benefit of using (DSR) is to determine shear resistance of the bitumen tested sample in wide range of temperature and frequency, where rutting and fatigue occur in asphalt pavement.

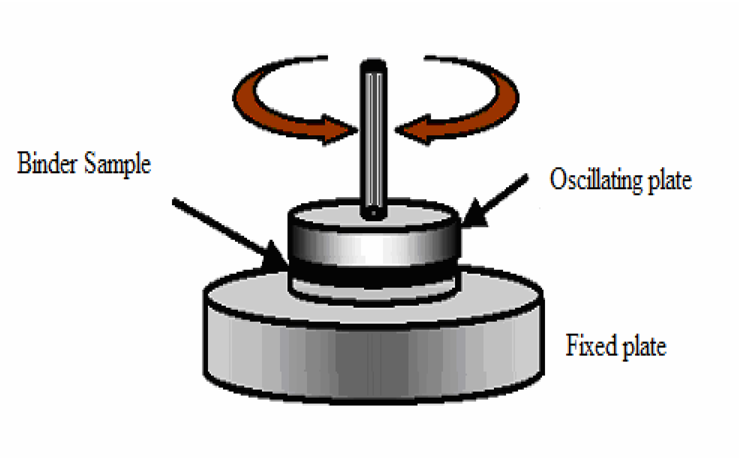


Figure (3-1) Dynamic Shear Rheometer after (Bahia, 1993)

3.2.1 Theory of Analysis and Data Collection

Gebhard, (2004) presented that dynamic tests provide data on viscosity and elasticity related to the rate of applied load and temperature. DSR test mechanism as a circular specimen mounted between two circular plates. The upper plate rotate around a vertical axis plate and the lower plate is constantly fixed. The specimens are subjected to specific shear stresses or strain at a range of frequencies by transfer the resulting torque to the upper plate. Dynamic testing provides an indication of the tested sample to resistance deformation. The performance of the tested sample can be presented as elastic or viscous component in the form of rheological parameter complex modulus and phase angle. Roberts, et al., (1996) mentioned that the data acquisition unit records the test temperature, applied load, loading frequency and deflection angle during the test cycles, which directly sends the test data to the personal computer. The computer software calculates the rheological parameters such as the shear stress, shear strain, complex modulus and phase angle and present it in the form of table and figure.

3.2.2 Rheological Properties

Gershkoff D. (1995) demonstrated that all modified bitumen standardization should be based the rheological properties of the used binder. Anderson et al. (1994) reported that the rheology science is to study and evaluate of the time/temperature dependent response of materials, which are subjected to an applied force. The rheological properties of the bitumen have a major influence on the bond properties between asphalt and aggregate. It is believed that asphalt pavement distress may be are related to the rhological properties of the used bitumen in asphalt concrete mixtures. All tests were done to evaluate the rheological properties take into account the climatic and loading conditions of the pavement because this type of test can be done in a wide range of temperatures and frequency. Numerous studies decide that the fundamental rheological parameter for modified and unmodified bitumen can be used directly as pavement performance indicator.

Briscoe (1987) described the benefit of the (DSR) that is used to characterize the viscous and elastic behavior of asphalt binder at high and intermediate service temperatures. The purpose of the experiment was to study the effects of modifier type and content on the rheological properties of asphalt binder. From the rheological test there are two main rheological parameter can be determined such as complex shear modulus (G*) and phase (□). Clear definition in this study were presented Complex modulus (G*), which is considered as the total resistance of the binder to deformation when repeatedly sheared. The complex shear modulus (G*) consists of two components: the first part is storage modulus, (G′), which can be defined as the elastic (recoverable) component, and the second part is loss modulus, (G″), which can be defined as the viscous (non-recoverable) component. The elastic component or storage modulus is related to the amount of energy stored in the sample during each testing cycle. The viscous component or loss modulus is related to the energy lost during each testing cycle through permanent flow or deformation. From the definition of main rheological parameter scientists can control this parameter, which directly control pavement distress.

It is known that the rheological criteria can provide a more reliable prediction of asphalt performance for modified, unmodified. Once the rheological parameters were determined there are several models that can be used to predict pavement performance. The rheoloical parameter can be used also as indicator of mix performance for the purposes of comparing binders in a given mix. It is advisable to control the rheological parameter by using different type of modifiers. Once the history of temperature range in highway construction area was known the type of the used bitumen can be determined. The aim of choosing suitable bitumen for asphalt concrete is to control pavement distress as possible.

3.2.2.1 Dynamic Complex Shear Modulus (G*)

Bahia et al (1995) described the relationship between the complex shear modulus (G*), storage modulus (G′), loss modulus, (G″) and phase angle (δ), graphically as presented in figure (3-2) and described mathematically as shown from equation (3-2) to (3-6). Time lag between applied stress and resulting strain, which is define as phase angle can be used to describe the viscoelastic bahavior 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.

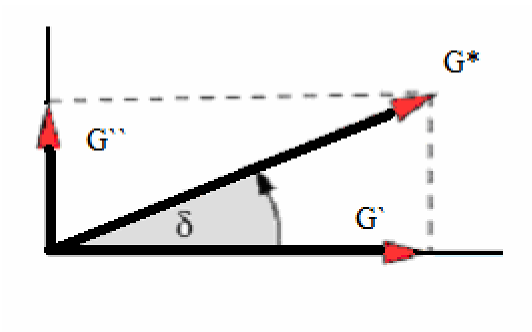


Figure (3-2) Relationship between Complex Shear Modulus (G*), Storage Modulus (G′), Loss Modulus (G″), and Phase angle (δ) after (Bahia, 1993)

$$G' = \mathrm{Cos}(\delta)(\tau / \gamma) \tag{3-2}$$

$$G'' = \mathrm{Sin}(\delta)(\tau / \gamma) \tag{3-3}$$

$$G^* = \frac{|\tau_{max} - \tau_{min}|}{|\gamma_{max} - \gamma_{min}|} \quad \mathrm{Pa} \tag{3-4}$$

$$\tau_{max} = \frac{2T_{max}}{\pi r^3} \quad \mathrm{Pa} \tag{3-5}$$

$$\gamma_{max} = \frac{\theta_{max} \ r}{h} \quad \mathrm{Pa} \tag{3-6}$$

$$\tan\delta = G' / G'' \tag{3-7}$$

Where:

τ_{max} : Absolute value of the peak-to-peak shear stress $[\mathrm{Pa}]$

γ_{max} : Absolute value of the peak-to-peak shear strain $[\%]$

T_{max} : Maximum applied torque (load) $[\mathrm{Pa}]$

G' : Storage modulus $[\mathrm{Pa}]$, G'' = Loss Modulus $[\mathrm{Pa}]$

r : Radius of specimen plate $[mm]$

θ_{max} : Maximum deflection angle $[rad]$

h : Specimen height $[mm]$

3.2.2.2 Phase Angle (δ)

Bahia et al (1995) defined phase angle as immediate elastic and the delayed viscous responses of the binder, obtained from the lag between the induced hear stresses and measured strains. The time lag between applied stress and resulting strain in oscillatory deformation test is shown graphically in figure (3-3). In case of elastic materials no phase difference between applied stress and resulting strain were found. At low temperature and higher frequency small phase angles are found since the bitumen approximates elastic behavior. On the other hand, at high temperature and lower frequency higher values for phase angle are exhibited because the bitumen nearly viscous.

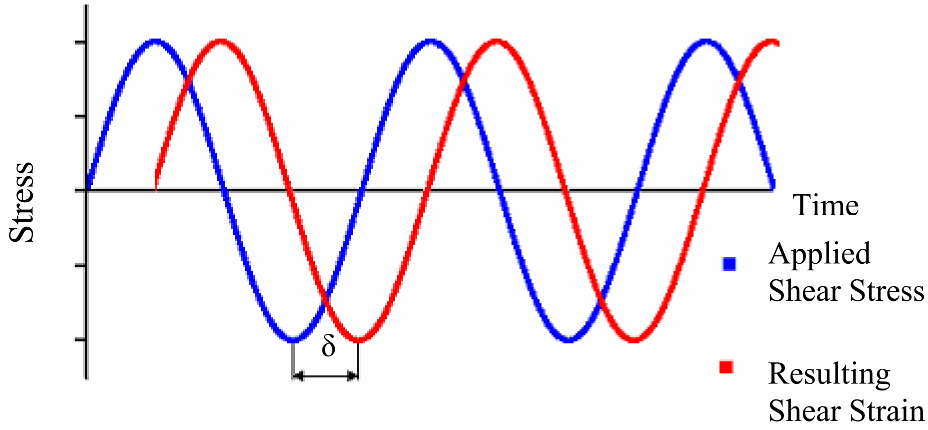


Figure (3-3) Viscoelastic material behavior for dynamic sinusoidal loading after (Bahia, 1993)

3.3 Test Specification and Objects

The rheological tests were done according to the European standard EN 14770 (Methods of test for petroleum and its products, Bitumen and bituminous binders - Determination of complex shear modulus and phase angle - Dynamic Shear Rheometer). The scope of the test method that the European standard specifies a number of methods using a dynamic shear rheometer (DSR) capable of measuring the rheological properties of bituminous binders. The procedure involves determining the complex shear modulus and phase angle of binders over a range of test frequencies and test temperatures when tested in oscillatory shear. The main object of using dynamic shear rheometer (DSR) is to understand the rheological properties of asphalt binder, which help to predict asphalt concrete pavement performance. Dynamic shear rheometer, temperature unit and data acquisition unit are presented in figure (3-4)

Figure (3-4) Dynamic Shear Rheometer (TU-Darmstadt, Asphalt lab)

3.4 Summary of Method

Bahia and Anderson, (1995a) described dynamic shear rheometer test summary and clear definition for oscillation test were presented. Oscillation tests are known as dynamic tests to evaluate the specimen's response to the sinusoidal stresses and to calculate rheological parameter. Complex shear modulus and phase angle of a binder, which are indicators of asphalt's resistance to shear deformation, help to predict pavement distress such as rutting and fatigue. Viscoelastic properties of asphalt are determined by evaluating the behavior of asphalt specimen, when it is subjected to oscillatory stress. Data acquisition unit records the test temperature, applied torque, loading frequency and deflection angle every 10 cycles of the test and send the data to the personal computer. The computer software calculates the shear stress, shear strain, complex modulus and phase angle. The software presents the measured and calculated value in the form of tables and figure.

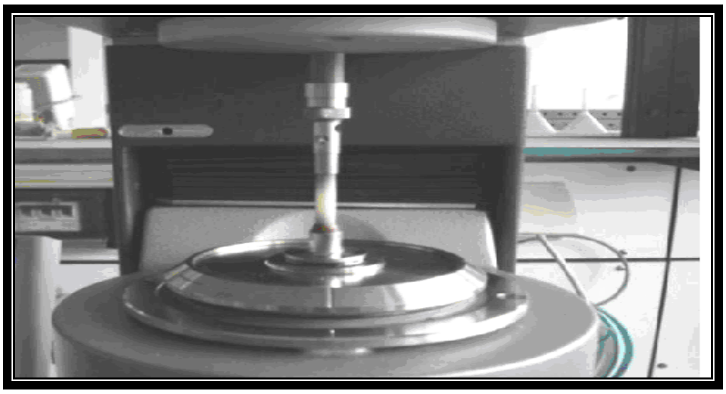


Figure (3-5) Plate-Plate test in dynamic shear rheometer (TU-Darmstadt, Asphalt lab).

3.5 Base Bitumen and Polymer Type

It is known from the previous studies that the degree of modification depends on the base bitumen type and modifier type. Numerous studies have been done in the field of polymer technology and there are several explanations for the need of using modifier in asphalt industry. There are different reasons for using asphalt modifier in asphalt industry started with improve the pavement performance, increase the service life of the pavement, meet the heavy traffic demands, and ended by saving cost of maintenance. Physical and chemical properties of base bitumen presented in table (3-1).

Table (3-1) Physical properties of base bitumen

Physical state	Solid at ambient temperature
Density	1010 to 1050 kg/m3 at 25 °C
Softening point	43 °C minimum.
	51 °C maximum.
Penetration	70 to 100 x 10-1 mm at 25 °C.
Flash point	> 230 °C
Storage temperatures	170 °C maximum.

The performance of asphalt concrete pavement can be improved by using either polymer or crumb rubber modified bitumen. Asphalt binder used in this study was (70/100) pertation grade from shell bitumen company penetration grade, which is the most widely used for intermitted temperature. Once the base bitumen were chosen, the second step for bitumen modification started by looking for suitable modifier. Two types of modifiers were used in this study to compare between the effect of waste materials and commercial polymer on fatigue resistance and the rheological properties of the produced binder. The first one is styrene butadiene styrene (SBS), which is a thermoplastic rubber. SBS is a tri-block polymer with a butadiene block in the middle of two styrene blocks (Kraton). Using SBS significantly increases strength at higher temperatures as well as flexibility at lower temperatures. The physical properties of styrene butadiene styrene (SBS) are presented in table (3-2).

Table (3-2): The physical properties of styrene butadiene styrene (SBS)

Physical properties / price (SBS)	Unit
Denestiy	1240 kg/m^3
Young's modulus (E)	2.800-3100 MPa
Tensile strength (σ_t)	30-50 MPa
Elongatian @ break	10-200%
Melting point	180 °C
Price	0.50-1.0 €/kg

The second modifier is crumb rubber powder (CR) from recycled passenger car tyres, particle size 0 – 600 µm. 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 (3-3).

Table (3-3): The physical properties of Crumb rubber (CR)

Physical properties Crumb Rubber	Unit
Denestiy	1320 kg/m^3
Young's modulus (E)	2600-2900 MPa
Tensile strength (σ_t)	40-70 MPa
Elongatian @ break	25-50%
Melting point	200 °C
Price	0.25-0.50 €/kg

Hanson, D. I., and Duncan (1995) studied the advantage of using crumb rubber, which is considered as polymer type. Crumb rubber modification mechanism depends on that rubber absorbs the solvent, which increases the dimensions of the rubber network. Due to increase in rubber net work dimension the bitumen start to be stable and uniform equilibrium swelling is achieved. An extensive review was made to get indication about the type of modifier that can be used and the modification level. Kim et al. (2001) mentioned that rubber produced from the scrap tires, known as crumb rubber, can be used in asphalt mixtures either as a binder modifier (wet process) or as a fine coarse aggregate replacement (dry process). In both wet and dry processes, rubber particles react with bitumen at high temperatures during the manufacturing stage. Compared to the wet process, the reaction time in the dry process is considerably less (maximum six hours) and slower due to the larger particle sizes.

Consequently, it is generally assumed that the effect of rubber-bitumen reaction in the dry processed mixture is less and, therefore, has a limited effect on the mixture performance. Clear definition for the polymer concentration range was given by literature review, expert opinion and manufacturer recommendation. In this study within this range, four modification levels were chosen as very low, low, middle and high modification level from the recommended range. SBS was incorporated into base bitumen with four different percentage namely 3wt%, 5wt%, 7wt% and 10wt% percent by total weight of the base bitumen. Crumb rubber was incorporated into base bitumen with four different percentages namely 3wt%, 5wt%, 7wt% and 10wt% percent by total weight of the base bitumen. In this research using more than 10% produce non-homogenous modified binder. Therefore, 10% were chosen as maximum modification level for both CR and SBS. The used modifiers in this research are presented in figure (3-6).

Figure (3-6) Crumb rubber and styrene butadiene styrene.

The modification level was chosen for phase (II) in experimental program acceding to degree of modification in the rhological parameter in phase (I). If the rheoloigical parameter values have significantly improved, then it is possible to use this modification level in phase (II). If the phase angle (δ) and complex shear modulus (G*) have no difference than neat bitumen then modifier content level will not included in phase (II). Once the optimum modifier content was determined from phase (I), directly the second phase start to evaluate the mechanical properties and fatigue resistance for modified and unmodified asphalt concrete mixtures. Figure (3-7) present the modifier type and modification level.

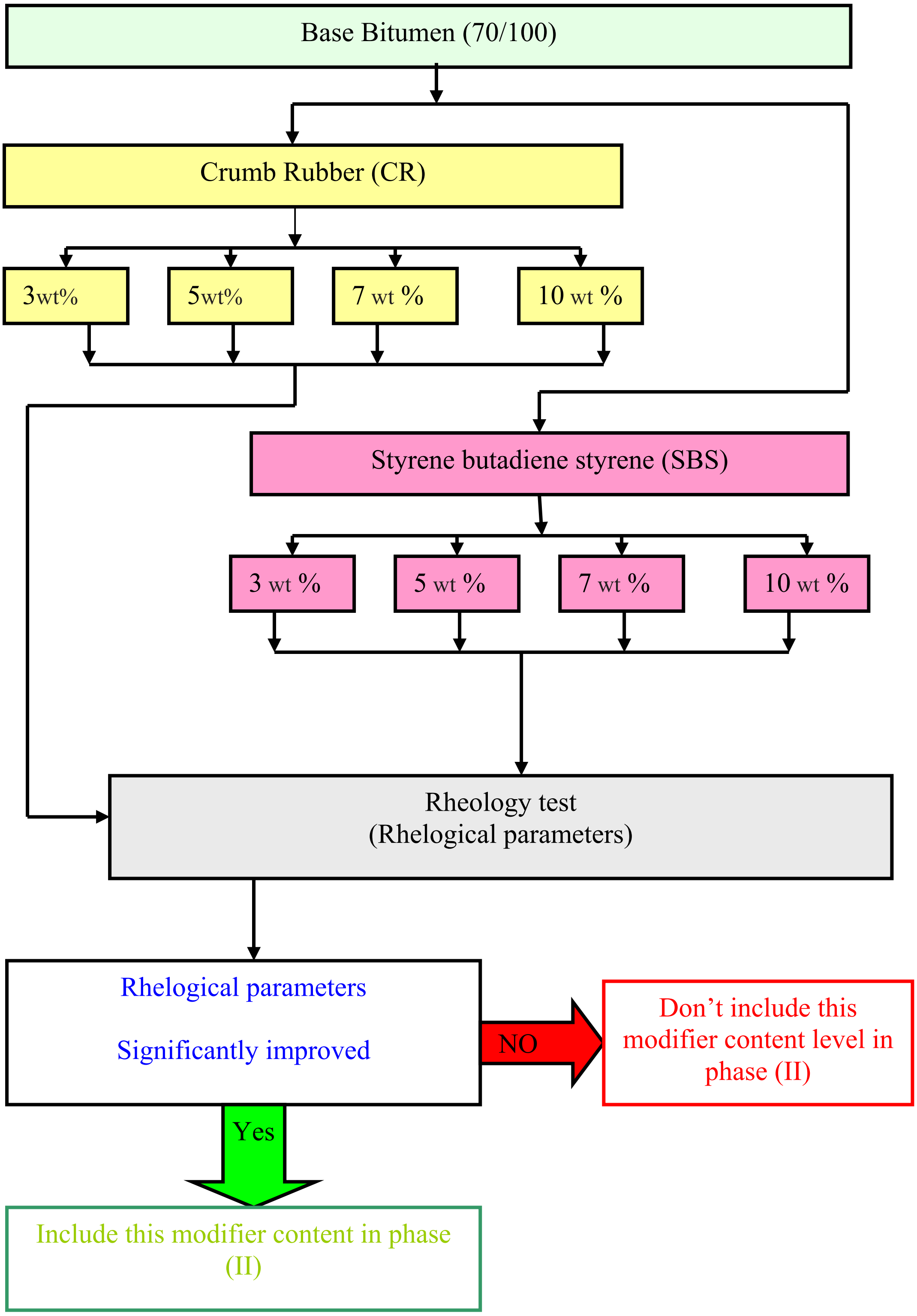


Figure (3-7) Schematic presentation of modification level and modifier types.

3.5.1 Asphalt Polymer Blending Requirements

Gordon D A, (1997) studied the factors, which affect the properties of modified bitumen. The properties of the modified bitumen depend on the polymer type, polymer content, and temperature. Each polymer type has its own way of mixing and special treatment. Different methods of mixing polymer with the bitumen were used and this is known from the experience or from old written report. The blending mechanism, which were used to blend bitumen with polymer depend on the level of shear rate, which induces into the bitumen. Clear definitions of the factors, which affect the properties of the modified bitumen, were presented in this study. There is a recommended shear blending speed is assumed to be 2500 rpm. Final conclusion from the previous study is that the blending time depends on the blender configuration and polymer type.

Al-Abdul-Wahhab, H and Al-Amri, G. (1991) reported that the blending temperatures depend on the molecular weight of the used polymer. It is clear that polymer with higher molecular weight needs higher temperature than the polymer with low molecular weight. The following blending sequence was used to modify bitumen materials with crumb rubber or styrene butadiene styrene:

- Asphalt cement was heated in an oven at a temperature of at least 160 ^{o}C.
- The stainless steel beaker used for mixing was cleaned and kept in the oven at a temperature of at least 160 ^{o}C.
- The required amount of asphalt was weighed into the beaker; then the amount of additive required to yield the desired additive-to-asphalt ratio was weighed.
- Eight blends were prepared with 3%, 5%, 7%, and 10% SBS and CR, respectively, by total weight of bitumen.
- The mixer was started, and the prepared amount of additive was added gradually to the beaker while stirring.
- The mixing temperature was controlled during mixing using heater and it is 180 ^{o}C for crumb rubber and 160 ^{o}C for styrene butadiene styrene.
- The ready modified bitumen was used to prepare the tests sheet, which were used for making DSR specimen with different diameter using special tools.

There are different methods to determine the optimum blending time. One of the most known methods to determine the optimum blending time is to measure one physical property for the tested material and when this physical property start to be constant with increasing mixing time then this is the optimum blending time. Complex shear modules at uniform interval time were considered during the blending process. The uniform interval time was 5 minutes and when the complex shear module does not show increase with time, the blending process should be stopped. Longer blending time for production of SBS modified binder and rubber-modified binders lead to an increase in the complex shear modulus; this is due to homogenous and stable network formation in the modified binder.

The optimum blending time is illustrated by the relationship between the complex shear modules versus the time as shown in figure (3-7). It is clear from the figure that the longer blending time in the control binder was found to have little effect on the complex shear modulus. The optimum blending time for production of 10% rubber-modified binders is about 60 minutes and the optimum blending time for production 5% SBS modified bitumen is about 45 minutes. The blending time is dependent on the level shear rate of the blender and polymer type, it is recommended to find the recommended time for each modifier.

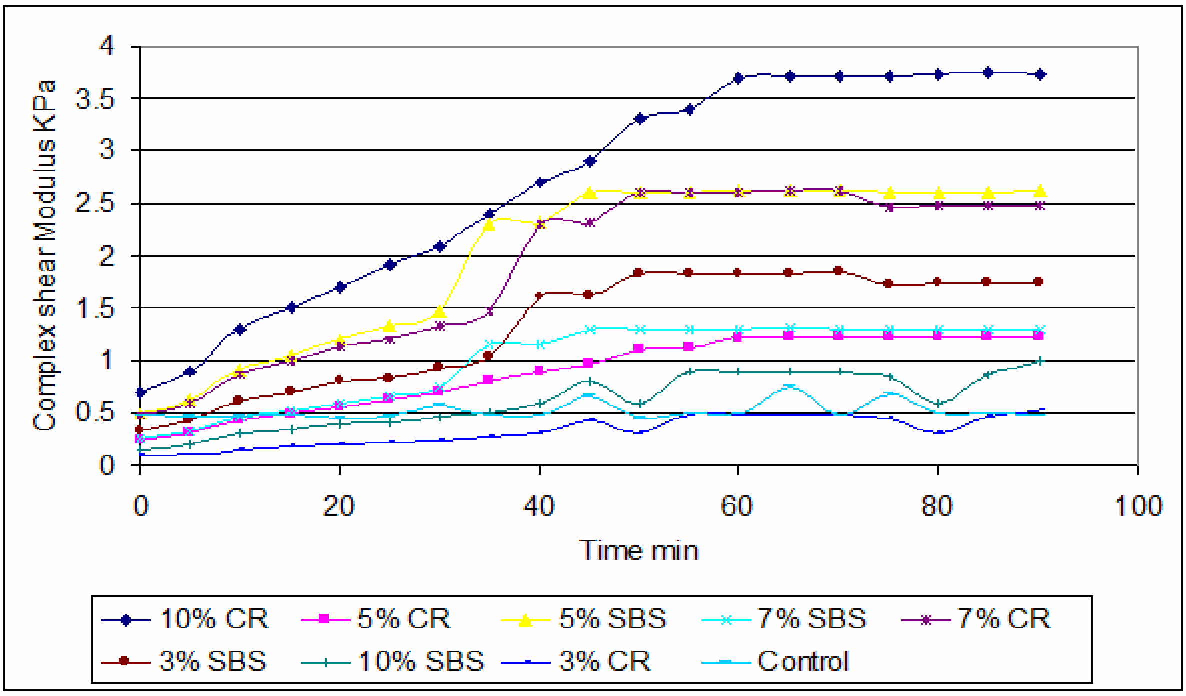


Figure (3-7) Relationship between the complex shear modules versus the time at 160 ^{o}C.

Table (3-4) presents the recommended blending time and temperatures for both crumb rubber (CR) and styrene butadiene styrene (SBS). It is clear from the table that crumb rubber needs more blending time and more temperatures than styrene butadiene styrene.

Table (3-4): Recommended blending time and temperatures for crumb rubber and styrene butadiene styrene

Type of Modifier	RBTM (*)	MBTM (**)	RBTD (***)	MBTD (****)
styrene butadiene styrene(SBS)	60	80	160	200
Crumb rubber (CR)	45	65	180	200

(*) - Recommended Blending Time Minutes

(**) - Maximum Blending Time Minutes

(***) - Recommended Blending Temperatures Degree

(****) – Maximum Blending Temperatures Degree

This is because the blending temperature depends on the molecular weight of the modifier and it is known that the molecular weight of crumb rubber is more than the molecular weight of SBS. The required blending temperature for (CR) is higher than the required blending temperatures for SBS. Different mixers were used to blend bitumen with polymer according the level of shear rate. The level of shear rate is defended as the speed of the blinder head. The higher shear blending speed is recommended to be 2500 rpm.

3.5.2 Asphalt Binder Specimen Fabrication

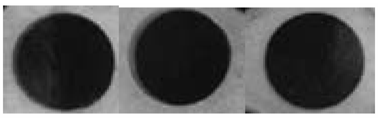
The mixing temperature of bitumen with polymer is generally between 150 ^{o}C and 200 ^{o}C or above. The binder is heated in the oven for about two hours at pouring temperatures of 135 ^{o}C for unmodified bitumen and 150 ^{o}C – 165 ^{o}C for modified bitumen until the binder is hot enough to pour. Steel plate is covered with non-stick paper then four small steel pieces with 1-mm and 2-mm according to the needed thickness were placed over the non-stick paper. Then the heated asphalt binder is poured in the space between the four small steel pieces.

Another non-stick paper was placed over the binder, which was intercepted between the four small steel pieces. The upper steel plate was covered by the second non-stick paper. The upper steel plate is pressed by hand to get uniform thickness for the heated binder. Finally the bitumen sheet was kept in refrigerator at (4 ^{o}C to 10 ^{o}C) to be solid enough. DSR specimen was prepared by using special tools of 25-mm and 8-mm diameter to cut the sample from the bitumen sheet.

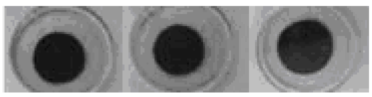
Figure (3-8) Asphalt blinder specimen fabrication

3.5.3 Specimen Geometry

The specimen geometry was chosen according to the test type, condition and specification. The specimen geometry at high temperature should have big diameter to save the specimen from melting. At low temperature the specimen should have small diameter with high thickness to prevent it from brittle crack. Two type of testing plate geometries are used with the dynamic shear rheometer. The first specimen geometry is 25-mm diameter spindle with 1-mm testing gap for intermediate to high temperature. The second specimen geometry is 8-mm diameter generally used at low temperature from -5 ^{o}C to 20 ^{o}C.



25 mm diameter - 1mm thickness
High Temperature



8 mm diameter - 2mm thickness
Intermediate Temperature

Figure (3-9) (DSR) test samples for high temperature and intermediate temperature

3.6 Asphalt Binder Rheology Tests

The rheological properties and fatigue resistance were performed using three different tests. The tests were done under range of frequency and temperature. Dynamic shear rheometer test types are presented in the following paragraph:

- Amplitude Sweep Test
- Frequency sweep at constant stress
- Fatigue Life for Asphalt Binder Using Dynamic Shear Rheometer

3.6.1 Amplitude Stress Sweep Test

The oscillation stress sweep test is used to determine a material's linear visco-elastic range. In this case the stress amplitude has a linear relationship which can be described by the following equation:

$$\tau_O = G^* \times \gamma_O \qquad (3\text{-}7)$$

Where:

τ_0 = Initial shear stress

G^* = Complex shear modulus

γ_0 = initial resulting strain

Complex shear modulus (G*) versus stress plot was used to determine the linear visco-elastic region. It is important that all rheological measurements are undertaken in the linear visco-elastic region of the response. The main reason to do the test within the linear visoelastic region is that the relation between stress and strain is influenced only by the frequency or temperature and not by the magnitude of the stress or strain because the behavior of the material is linear. Anderson et al. (1994) defined the liner visco elastic limit as the point where complex modulus, (G*), decreased to 95% of its initial value. All asphalt binder specimens were subjected to a stress sweep to determine the maximum stress that can be applied as input parameter during the frequency sweep test. Figure (3-10) presents load behavior during the test, it is clear that the stepwise increase with the increase of stress amplitude.

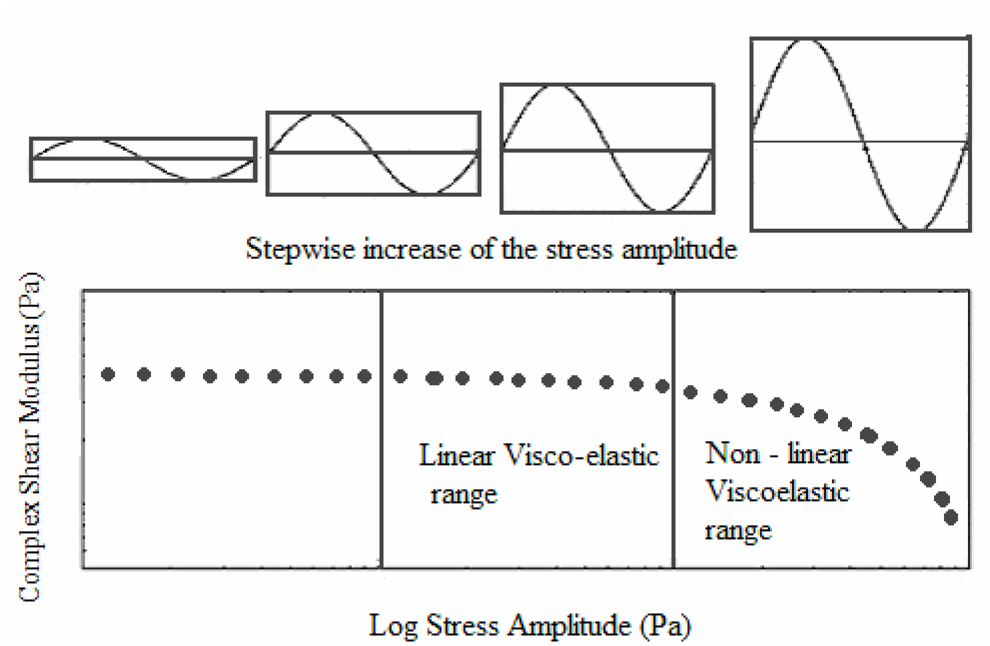


Figure (3-10) Amplitude stress sweep test after (Gebhard, (2004)

Gebhard, (2004) gave a clear definition for the benefits of oscillatory test, which can be used to characterize the stress – strain response of the tested material. It is easy by using this test to describe the difference between elastic, viscous and viscoelastic behavior according to the material response. Amplitude stress sweep parameter and condition are presented as shown in table (3-5).

Table (3-5) Amplitude stress sweep test conditions.

Test parameter	Test conditions	Units
Mode of loading	Stress mode	
Stress (τ)	1- 15000	Pa
Temperature (T)	50 ^{0}C	^{0}C
Frequency (F)	10	Hz
Bitumen thickness (t)	1	mm
Spindle diameter (D)	25	mm

The elastic material exhibits stress with proportion to the strain. On the other hand, viscous material exhibits stress proportion to the rate of strain. When a viscoelastic material is loaded, it exhibits both elastic and viscous behaviour and displays a time independent relation between the applied stress and result strain.

3.6.2 Frequency sweep at constant stress

Frequency sweep test is very important test to evaluate the rheological properties of asphalt binder. Two main rheological parameters, which measured from frequency sweep test over a range of frequencies, are useful to determine the viscous and elastic properties of the tested binder. As described before rheological parameters are complex shear modulus and phase angle. These two parameters can be used as input parameter in asphalt model to describe the pavement performance. Frequency sweep test is used to construct the stiffness master curve and black diagrams for the tested sample. The load from the linear visco-elastic limits was used at different temperatures. The test parameter and condition are presented as shown in table (3-6).

Table (3-6) Frequency sweep test specification.

Test Parameter	Test Conditions	Units
Mode of loading	Stress mode	
Stress (τ)	1000	Pa
Temperature (T)	20,35, 45, 55	^oC
Frequency (F)	0.1 to 30	Hz
Bitumen thickness (t)	1	mm
Spindle diameter (D)	25	mm

Gebhard, (2004) reported that for real visco-elastic materials both the complex modulus and the phase angle (δ) are frequency and temperature dependent. Therefore, in frequency sweep test the measured values of G^* and δ can be plotted as a function of frequency. Relation between angular velocity and complex shear modules are presented in figure (3-11). When the frequency increases stepwise each step give two rheological measurements. The first one is complex shear modulus and the second one is phase angle and both are important to evaluate asphalt binder properties.

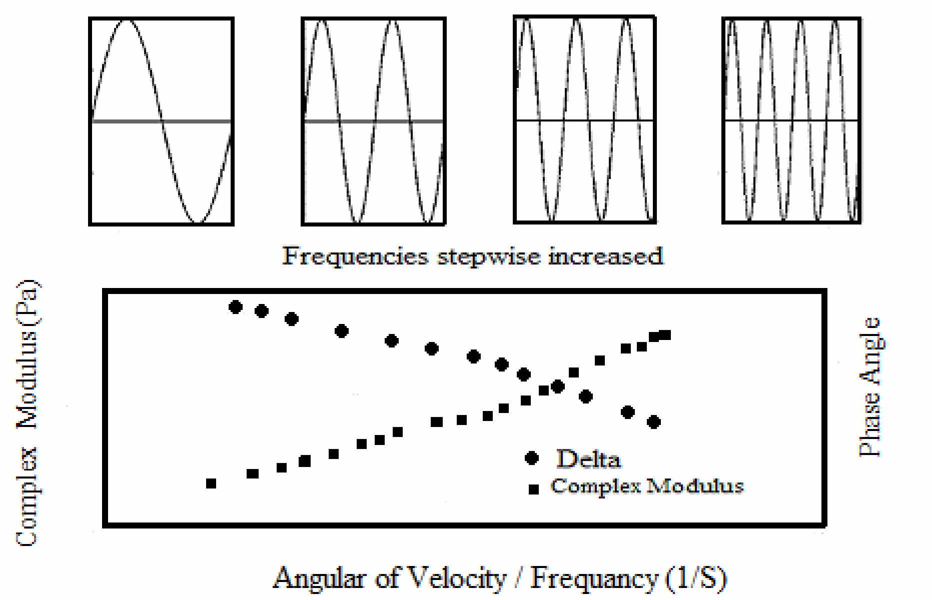


Figure (3-11) Dynamic test frequency sweep after (Gebhard, 2004)

3.6.3 Fatigue Life for Asphalt Binder (time sweep test)

Asphalt pavements are known to exhibit fatigue distress or deterioration of binder stiffness under repeated traffic load in the intermediate temperature range from about 10 ^oC to 30 ^oC. One of the most important tests used to determine fatigue test for asphalt binder is oscillatory time sweeps test. This test is designed to determine the resistance of asphalt binder to fatigue under repeated oscillating load. Oscillatory time sweeps are important when testing materials, such as viscoelastic materials, that may undergo macro- or micro-structural rearrangement with time. Due to rearrangements in the materials structures the rheological properties for the tested materials are affected directly. Also, the test result provided the necessary information about how a material changes with time depending on the tested sample properties.

It is logical and known to study fatigue behavior for asphalt concrete pavement using asphalt concrete mixture. Measuring fatigue behavior of asphalt mixtures has been the subject of numerous studies, which includes both laboratory and field studies. In literature, a little attention has been given to a possible fatigue property of the asphalt binder itself. In the same time asphalt researchers don't ignore of the fact that fatigue is a phenomenon in asphalt binder; therefore it seems more appropriate to measure fatigue of asphalt binders. Last 10 years asphalt researchers start to study fatigue tests for the bitumen itself. Fatigue test provides the advantages that no need to prepare asphalt concrete mixtures and to avoid the influence of different mixes designs and the effect of compaction. Moreover, the proposed test method for bitumen fatigue test is relatively simple and by tanning the test result for the same bitumen under the same condition start to be nearly the same. On the other hand, fatigue test for asphalt binder have some disadvantages, the test is costly and time consuming. However, it is challenging to handle pure asphalt and apply repeated loading to this sticky visco-elastic material. Therefore it is very appropriate to measure fatigue of asphalt binders.

The fatigue test was performed on un-aged asphalt binder to determine the effect of modifiers on fatigue properties using dynamic shear rheometer. Fatigue life of asphalt binder can be considered as number of cycles until damage. Fatigue tests can be performed with 8-mm plates and 2.0 mm thinness. A time sweep was performed, in which an oscillating stress of 10000 and 100000 kPa was applied with constant frequency 10 Hz. The DSR performed a continuous oscillation procedure, and the complex modulus (G*) was monitored until it dropped sharply. Fatigue test time is about 2 hour to 4 hour and the test time is considered as one of fatigue test disadvantage using dynamic shear rheometer.

Different methods were used to evaluate the fatigue damage for asphalt binder. Bahia, H., and Anderson, D. A. (1993) explained that fatigue damage in viscoelastic materials can be evaluated using stored and dissipate energies. The energy balance is influenced by rheological properties of the binder, which are in turn functions of temperature and the rate of loading. Damage can be evaluated by dissipated energy and number of cycles to failure. In this study dissipated energy ratio concept is used to determine the fatigue life for modified and unmodified asphalt binder. For using dissipated energy as method to evaluate the fatigue properties for asphalt binder there are different steps needed. The first step is to find out the initial dissipated energy and the second step is to make clear definition for failure point. The point of failure is obtained from analyzing the variations of dissipated energy ratio per loading cycle.

3.6.3.1 Initial Energy Input

Two input energies are needed to determine the fatigue relationship according to Bahia, et. al. (1993). The first input energy represents weak pavement where the binder is considered to be loaded in the non linear range. The second input energy represent strong pavement where the binder is loaded in the non – linear range as shown in figure (3-11). To check the linear and non – linear properties of the binder amplitude sweep testing was done. The strong pavement was defined as the middle of the linear limit and the weak pavement was picked up slightly above the linear limit.

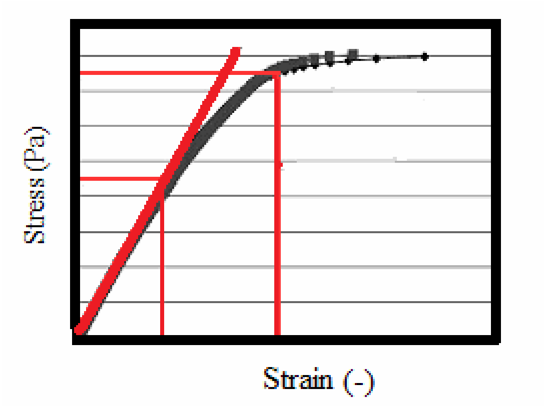


Figure (3-11) Typical example for choosing input Energy after (Kitae, 2004).

Fatigue data was processed according the dissipated energy ratio (DER) and typical example for DER calculation are presented in appendix (C). Time sweep test parameter and condition are presented as shown in table (3-7).

Table (3-7) Time Sweep Test Specification.

Test parameter	Test conditions	Units
Mode of loading	Stress mode	
Stress (τ)	10000-100000	Pa
Temperature (T)	20	^{o}C
Frequency (F)	10	Hz
Bitumen thickness (t)	2	mm
Spindle diameter (D)	8	mm
Test Time	120 to 240	Minutes

The dissipated energy ratio can be calculated as a function of the number of loading cycles from the selected initial energy, loading rate, and temperature during the fatigue test. From table (3-6) summery of test input parameter that used to evaluate fatigue resistance for modified and unmodified asphalt binder are presented in the next chapter:

- Energy input: two levels of energy input were used. One in the middle of the linear range and one slightly over the linear limit, as explained earlier.
- Testing temperature was chosen as 20 ºC, which corresponds to the average of winter Egyptian climate.
- Testing frequency used was 10 Hz, which correspond to the fast traffic speed.

3.6.3.2 Crack Initiation and Crack Propagation

Bahia, et al (1993) explained a clear definition for damage that can be characterized by a decrease in the energy dissipated per loading cycle. Fatigue life of the binder is represented by the number of cycles required to undergo the crack initiation without reacting the crack propagation. Kitae et al (2004) presented the result for fatigue test and explained that the number of cycles to failure can be divided into two groups: the number of cycles to initial crack, and the number of cycles during crack propagation to failure.

Khalid. A. et al (2006) presented clear definition for fatigue damage as the number of cycle (Nf) where the dissipated energy deviates straight line this point was selected because it provides independence of the mode of loading. If the initial input energy is the same, the (Nf) value obtained using stress controlled or strain controlled test will be the same. Figure (3-12) presented crack initiation and crack propagation for the tested sample. The number of cycles to failure (Nf) occurs after the crack initiation point and usually before the crack propagation point, so it provides reasonable criteria for defining the fatigue failure of binders. The point where the trend line leaves the linear slope corresponds to the point where the fatigue cracks are initiating in the binder. This point is refered as crack initiation point. However, after this point, if the binder is unloaded and enough resting time is given, it is still capable of recovering. After more cycles of testing the crack propagation point is reached. When the binder goes beyond this point no more healing is possible and the fatigue damage is not reversible.

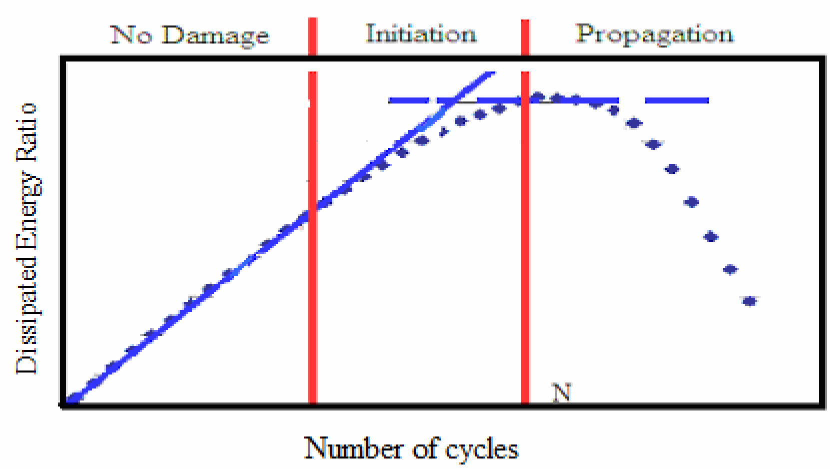


Figure (3-12) Variation of dissipated energy ration using control stress test after (Kitae, 2004)

3.6.3.3 Measurements and Calculation:

The required frequency is related to the traffic speed for fast speed 60 mph the testing frequency should be 10 Hz and for the slow speed 15 mph the testing frequency should be 2.5 Hz. To determine number of cycles, the frequency should multiply with the test time and the number of load cycles to failure depends on the test time. Traffic speed affects the fatigue life mainly by influencing the binder and mixture modulus. Higher speed results in a higher modulus; higher modulus will result in a lower strain for a fixed loading application. The effect on fatigue life however is not simple and could depend on binder sensitivity to loading rate and fatigue dependency on energy input. Some binders are more sensitive than others in terms on modulus change but are less sensitive in terms of fatigue change with energy input.

Yun Liao (2007) presented graphically the relation between stress and strain for different materials. Figure (3-13) graphically illustrates stress-strain behavior for elastic and viscoelastic materials during a loading and unloading cycle, where (σ) is the applied stress and (ε) is the resulted strain. Pronk and Hopman (1990) defined the dissipated energy as the difference between two stress-strain circles which may be induced by energy loss, micro-damage. Bahia et al (1995) presented a new definition for dissipated energy and its relation with damage. The dissipated (loss) energy is the amount of energy which is dissipated by viscous flow and/or plastic flow, and leads to potential damage when bituminous materials is subject to repetitive loading. From figure (3-13) the red hysteretic loop represents either time-dependent viscoelastic behavior (strain accumulation due to viscoelasticity only) or, if the damage occurred, the amount of energy loss due to frictional resistance. When the load is applied to elastic materials there is no hysteretic loop (energy loss).

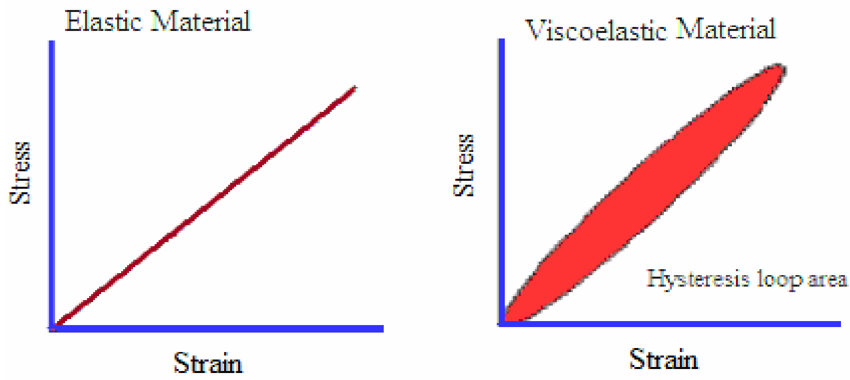


Figure (3-13) Stress-strain behavior for elastic and viscoelastic materials (source Yun Liao 2007).

Yun Liao (2007) reported that dissipated energy is the area under the stress-strain curves of hystressis loop. Clear definition for microcrack and crack initiation as dissipated energy during the micro-crack development stays fairly constant and after the mickrocrakes reach a certain threshold this point defines as crack initiation. After crack initiation a dramatic shift in the in the rate of dissipated energy occurs that means the crack propagation phase start. The energy balance is influenced by rheological properties of the binder, which are in turn functions of temperature, frequency or loading and stress/strain. Development and accumulation of damage is evaluated in terms of dissipated energy and number of cycles. The dissipated energy and the dissipated energy ration were outlined by Pronk.

$$W_i = \frac{\pi . \tau^2 . \sin(\delta)}{G^*} \qquad (3\text{-}8)$$

Where:

Wi : Dissipated energy per cycle

τ : Applied stress (Pa)

G^* : Complex modulus (Pa)

δ :Phase angle (Degree)

Fatigue assumed to be minimized by controlling the dissipated energy. Lower value of (G*/sin (δ)) led to lower value of dissipated energy per loading cycle. The concept of dissipated energy ratio allows the determination of the fatigue properties for both the asphalt binder and the asphalt concrete mixtures. This test can be done under both control stress or control strain. Equation (3-9) presented dissipated energy ratio.

$$DER = \frac{\sum_{i=1}^{n} Wi}{Wi_n} \tag{3-9}$$

Where

Wi_n = Dissipated energy at cycle n

The number of cycle of load application (Nf) to reach the crack propagation is the parameter that represents the binder fatigue life. The conventional power low model expressed as the number of cycles to failure in fatigue as function of the initial dissipated energy.

$$N_f = k_1 \times \left(\frac{1}{w_i} \right)^{k2} \tag{3-10}$$

The relationship between the fatigue life of a binder (Nf) and the energy input (Wi), when plotted in a log-log scale, can be approximated by equation (3-10). To determine the parameters K_1 and K_2, two points are needed. This means that, for a fixed pavement temperature, the binder has to be tested at least at two different energy levels in order to obtain the fatigue relationship. After the fatigue parameters are calculated, the fatigue relationship can be used to calculate the fatigue life at any input energy for the selected pavement temperature. K1 and K2 are used to describe the fatigue behavior of the asphalt binder.

Chapter 4
Rheological Properties and Fatigue Analysis for Bitumen

4. Introduction

This chapter presents the rheological properties and fatigue resistance test result for asphalt binder as well as brief analysis test data. The testing framework was selected in order to investigate the influence of CR or SBS in the bitumen properties subjected to different loading parameters, as explained with more detail in Chapter 3. To select the type of modifier, which can be used in pavement industry this should be based on the rheological properties of modified bitumen, which assumed to be the finger print for optimum modifier content and modified type selection. Both neat and modified asphalt binders were tested in this study, all results are presented below. The effect of modifier type and content were also evaluated and the rehological test results were divided into three parts. The first part presents the results of stress sweep test for both modified and unmodified binder to find the linearity limits. The second part presents the effect of waste materials such as crumb rubber and commercial polymer such as styrene–butadiene–styrene (SBS) on asphalt binder rheology. The third part presents fatigue test result as well as the effect of the modifier type and content on fatigue resistance. The test procedure, specifications and methodology were described before in details in chapter (III). The rheological properties of the modified binders were characterized using dynamic mechanical analysis over wide ranges of temperatures and frequencies. A summary of the test results is presented in tables and graphical form in the following sections.

4.1 Stress Sweep Test Result

Stress sweep testing was carried out using the dynamic shear rheometer, which has been successfully used to determine the linearity limits as a percentage decrease of the initial complex shear modulus value at selected temperature, frequency and the stress. Figure (4-1) and Figure (4-2) presented the complex shear modulus G* versus stress for CRM bitumen and SBSPM bitumen, respectively.

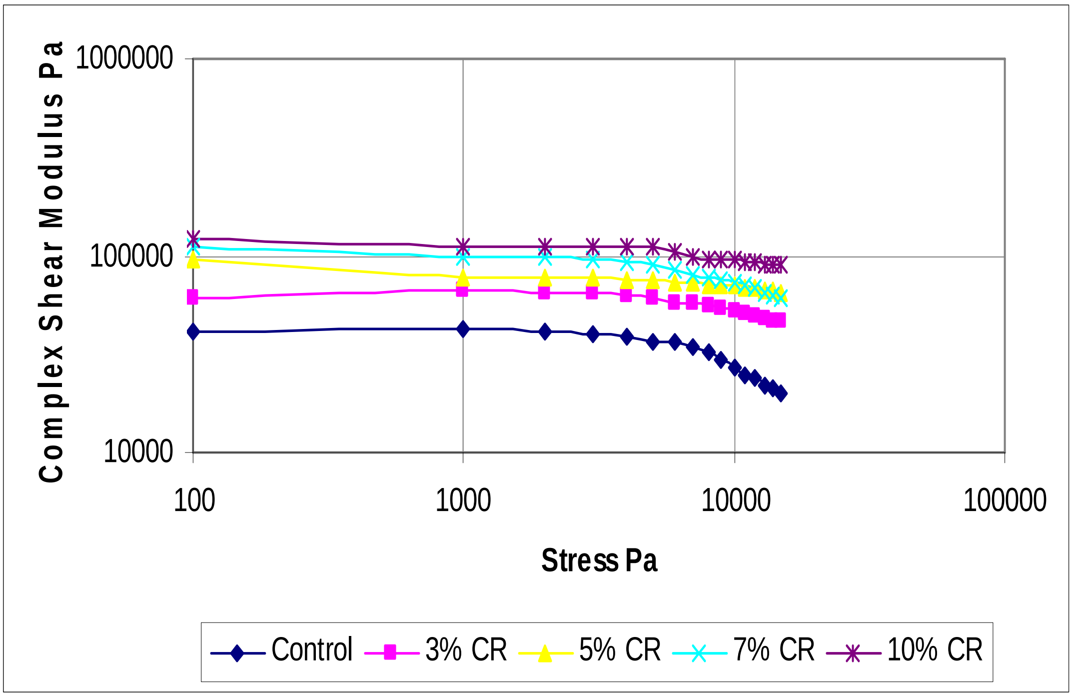


Figure (4-1) Complex shear modulus G* versus stress for CRM bitumen.

The test has been proven to provide reliable information on linearity limits related to response to dynamic loading. Stress sweep test result for all tested binders at various loading frequencies is presented in this section. Measurements were performed at 35°C with a shear stress range from 100 Pa to 15000 Pa and at frequency equal to 1 Hz.

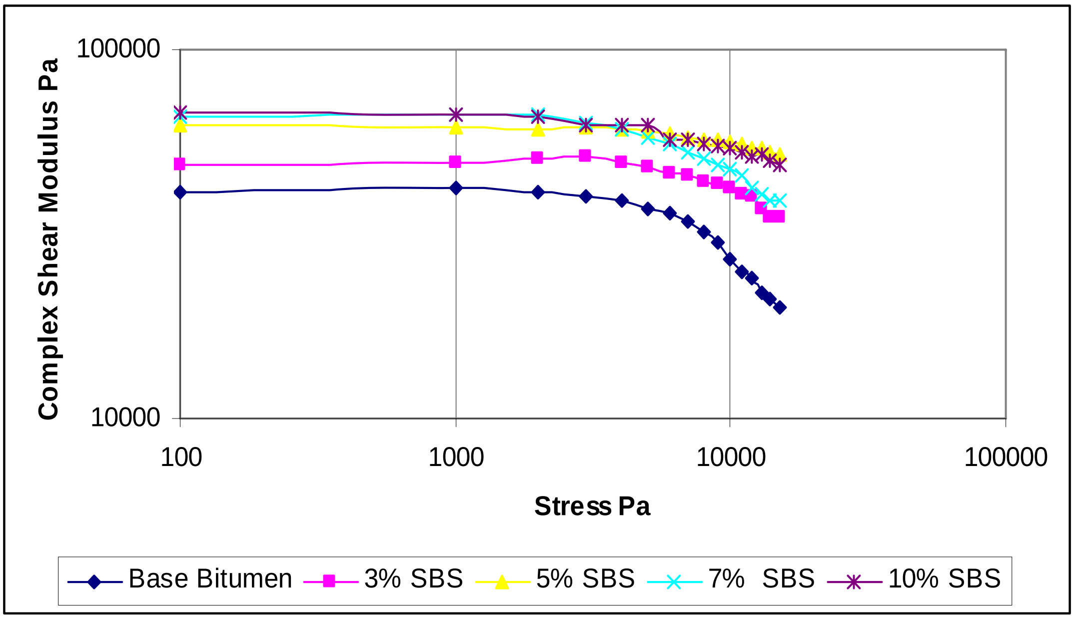


Figure (4-2) Complex shear modulus G* versus stress for SBS PM bitumen

The linear viscoelastic range (LVR) is illustrated by the relationship between the complex shear modulus G* versus shear stress and shear strain. In SHRP study it is reported the linear viscoelastic range (LVR) was defined as the point where complex modulus decrease to 95% of its initial value. The results were to compare between asphalt binders, which could be help to choose the input parameter for frequency sweep test. Based on these liner limits, rheological tests using the DSR were performed at stress level well inside the linear region and the liner rheological properties for modified and unmodified bitumen.

The test was repeated at different temperature and frequency in ordered to choose suitable shear stress or stain as input parameter for all asphalt binder.For the majority of binders; it has been found that the linear range lies between 100 Pa to 8000 Pa. To make the tests much easier, the same stress level was chosen for both control bitumen and modified bituminous binders. Therefore, 1 kilopascal was chosen as the constant stress used as input parameter for frequency sweep test for all binders.

4.2 Frequency Sweep Test Result

The frequency sweeps could be used for the construction of both master curves and black diagrams. It is known that frequency sweep test used to obtain complex shear modulus in Pascal and phase angle in degrees for the given test temperature and frequency for both modified and unmodified bitumen. There were three replicate specimens tested for each asphalt binder. Complex shear modulus and phase angle values were calculated for each specimen under the same test conditions and the average value was calculated.

4.2.1 Frequency Sweep Test Result for CRM Bitumen

Stiffness was measured at four different temperatures: 20 ^{o}C, 27 ^{o}C, 35 ^{o}C and 45 ^{o}C over a range of frequencies from 0.1 Hz to 30 Hz. The highest loading frequency was selected because it is intended to simulate highway traffic speeds, and the lowest testing frequency was selected because it simulates loading in slow moving traffic conditions. A summary of complex shear modulus and phase angle for modified and base bitumen are presented in appendix (A). The test results, complex shear modulus and phase angle at various frequencies for crumb rubber modified bitumen are illustrated in Figures (4-3) and (4-4). It clear from the result that general trend is that the shear modulus (G*) increases with the testing frequency and phase angle for the bitumen tested sample tends to decrease with the increases of testing frequency. For the same testing frequency, the shear modulus increases as testing temperature decreases. From figure (4-3) and (4-4), it can be seen that the modified asphalt binder have high complex shear modulus and lower phase angle than base bitumen. This result was expected as the G* values for the modified bitumen increase with increases of crumb rubber content and in the same time the phase angle decrease with the increase in crumb rubber percentage. Using crumb rubber with different modification levels 3%, 5%, 7% and 10% help to increase the stiffness of the standard (70/100) binder to some degree by factors of 12%, 25%, 43% and 50% respectively, at 10 Hz and the same temperature 20 ^{o}C.

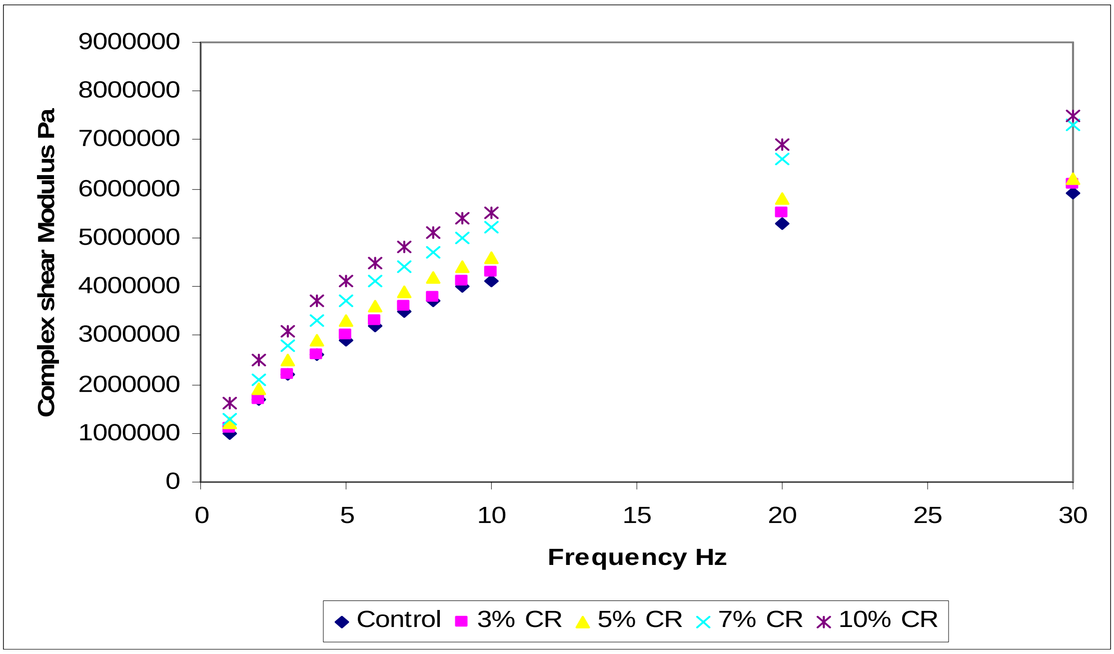


Figure (4-3) Complex shear modulus G* versus frequency for CRM bitumen at 20 ^{o}C.

The rheological properties are strongly dependent on crumb rubber content and the test condition. As indicated in Figure. (4-3), at low rubber content (3%) and (5%), the behaviour of the modified binders remains close to that of the base bitumen this is because of bitumen and modifier are separated. It can be related to uncompleted formation of network structure in the modified binders or the modifier content is not enough to make saturated modified binder by chemical cross linking. By increasing the rubber content the rubber absorbs the solvent, which increases the dimensions of the rubber network until equilibrium swelling is achieved. The increase of complex shear modulus is considered as indications of the development of a network within the binder structure and the binder have higher viscosity. In fact, the complex shears modulus increase by approximately two times when frequency is reduced from 30 Hz to 10 Hz. The increase in phase angles with the increase in frequency indicates that the binders became more viscous. CRM bitumen have low phase angle at high and low temperature than the conventional one that means the modified bitumen tending to become more elastic in behaviour. The purpose of blending CRM with bitumen was to enhance the elastic and resilient properties of the modified bitumen.

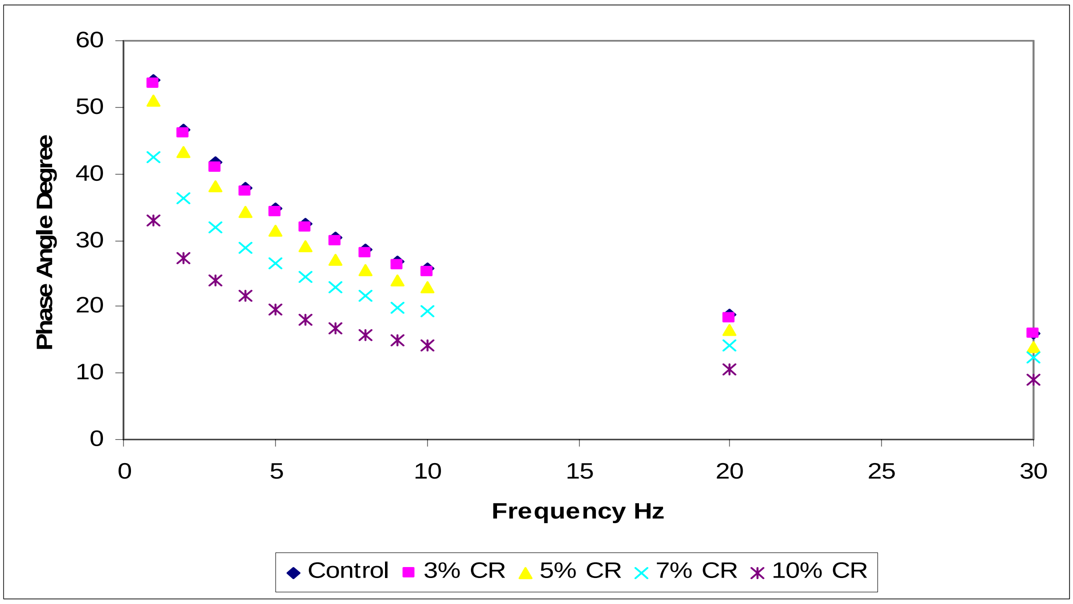


Figure (4-4) Phase angle versus frequency for CRM bitumen at 20 oC.

Oliver 1982 reported that when the crumb rubber is added to base bitumen the rubber particles absorb the aromatic oils from the bitumen. The absorption of aromatic oils causes softening and swelling of the rubber particles. Therefore, the binder consists of soft rubber, lead to produce modified bitumen more elastic than the base one. In the same time the modified bitumen seems to be hard because of the loss of oils. From the previous result it appears clearly that when the rubber was added to the base bitumen decrease of the phase angle which directly affects the elastic recovery properties and helps the bitumen to be fully recovered when the load is removed. In the same time complex shear modulus which represents the stiffness of the bitumen increase by using different modification levels. The increase in stiffness is due to rubber absorb oil from the base bitumen and this physical mechanism produce hard bitumen. The additional of crumb rubber reduce the low molecular weight maltenes from the bitumen, which help to improve the rheological properties of the bitumen. Crumb rubber modified bitumen showed different properties compared to the base bitumen and greater effect on the rheological properties were found. In conclusion, the additional of crumb rubber as modifier to the base bitumen lead to produce hard and elastic bitumen.

4.2.2 Frequency Sweep Test Result for SBS PM Bitumen

Frequency sweep test were done for styrene–butadiene–styrene (SBS) modified bitumen and complex shear modulus G* for different concentration of namely 3%, 5%, 7% and 10% are presented as shown in figure (4-5). It can be seen from the result that adding SBS polymer increases the complex shear modulus and this improvement is due to the formation of polymer network between polymer and base bitumen.

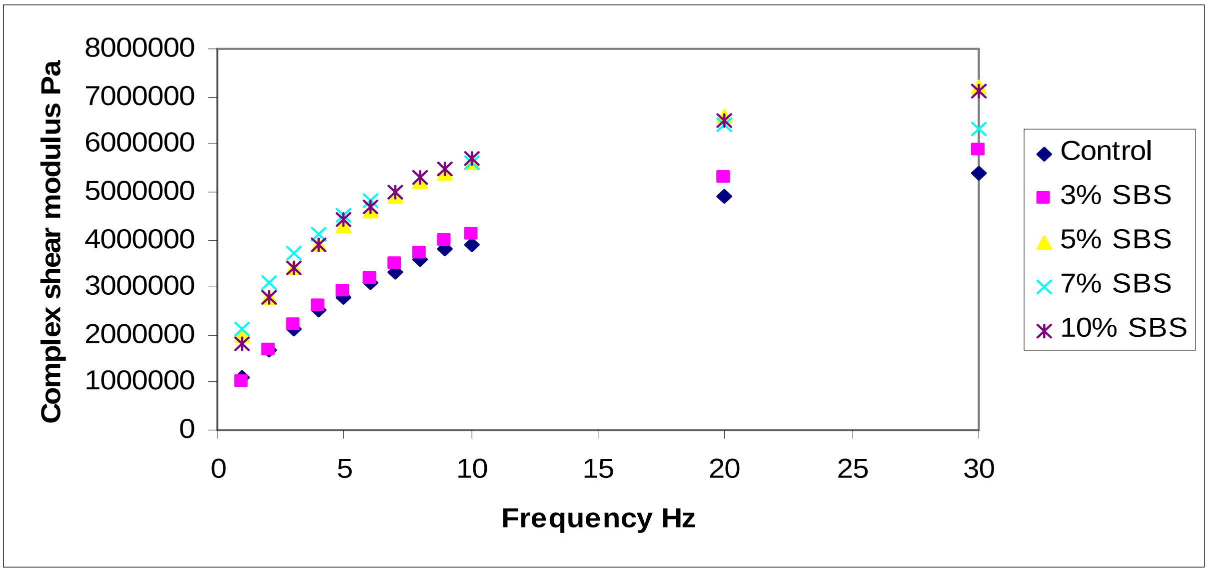


Figure (4-5) Complex shear modulus versus frequency for SBS PM Bitumen at 20 ^{o}C

At higher SBS polymer content (7%) and (10%), the behavior of the modified binders remains close to that of the modified binder with 5% SBS. Modified bitumen with 5% SBS appears to be the optimum polymer content which give stable network and saturated solution with acceptable rheological properties. Adding more polymer as modifier to the base bitumen will not add more improvement in rheological properties. The rheological properties help to choose the optimum polymer content as well as the polymer type. The phase angle for modified and unmodified asphalt binder is presented as shown in Fig. (4-5). From the result it can be seen that there is a difference in phase angle between modified and unmodified binder. The decrease in phase angle with the increase in temperature make the SBSPM-bitumen behaves to some extent as elastic materials and the increase in phase angle at high temperature means that the materials start to be more viscous. The change in the rheological properties of the modified binder because of polymer absorbs the liquids and swells. The amount of liquid that polymer absorb depends on the nature, temperature and viscosity of the used bitumen as well as the polymer type.

To select the correct asphalt modifier, the modifiers should have the following properties:

- Modifiers have to be compatible with asphalt where the compatibility is dispersion between polymer and bitumen.
- Modifiers should keep its properties after mixing and storage.
- Modifiers should not segregate or degrade during mixing.

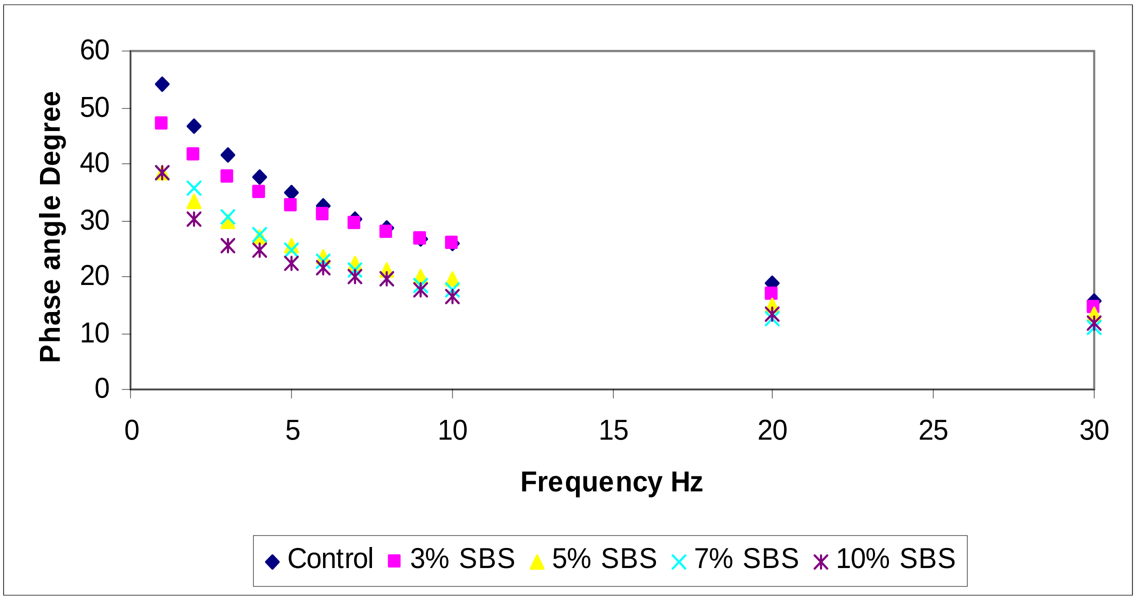


Figure (4-6) Phase angle versus frequency for SBS PM bitumen at 20 °C.

4.3 Temperature Effects on Rheology of Bitumen

Temperature has great effect on asphalt binder properties such as stiffness. The change in materials stiffness depends on materials temperatures susceptibility. There are different methods to improve asphalt binder temperature susceptibility one of these methods is known as using additives as asphalt modifiers. It was proven that addition of additives like polymers can influence the temperature susceptibility of asphalt binder and as a result an improvement in characteristics and performance of asphalt mixtures can be obtained in a wide temperature range. It is known that the temperatures affect the bitumen rheology because of this, the laboratory temperature test conditions are chosen to reflect the expected in-service temperatures. In general, asphalts with higher temperature susceptibility are unable to relieve stresses as easily at low temperatures and thus experience more thermal cracking than softer asphalts. Frequency sweep tests were performed on the binders over the temperature range 20 to 45 ^{o}C and generally in the frequency range 0 Hz to 30 Hz. The physical properties and temperature susceptibility characteristics of the bitumen influence pavement performance.

4.3.1 Temperature Effects on Rheology of CR Modified Bitumen.

A relationship between average complex shear modulus and test temperature is illustrated in Figure (4-7). As expected, shear modulus for all mixes is a function of temperature: the higher the testing temperature, the lower the complex shear modulus. The test procedures and specification were level were presented before in chapter 3. The rheological properties in terms of complex shear modulus and phase angle are presented in Figure (4-7) and Figure (4-8), respectively. The complex shear modulus of crumb rubber modified bitumen is higher than the complex shear modulus of the base bitumen.

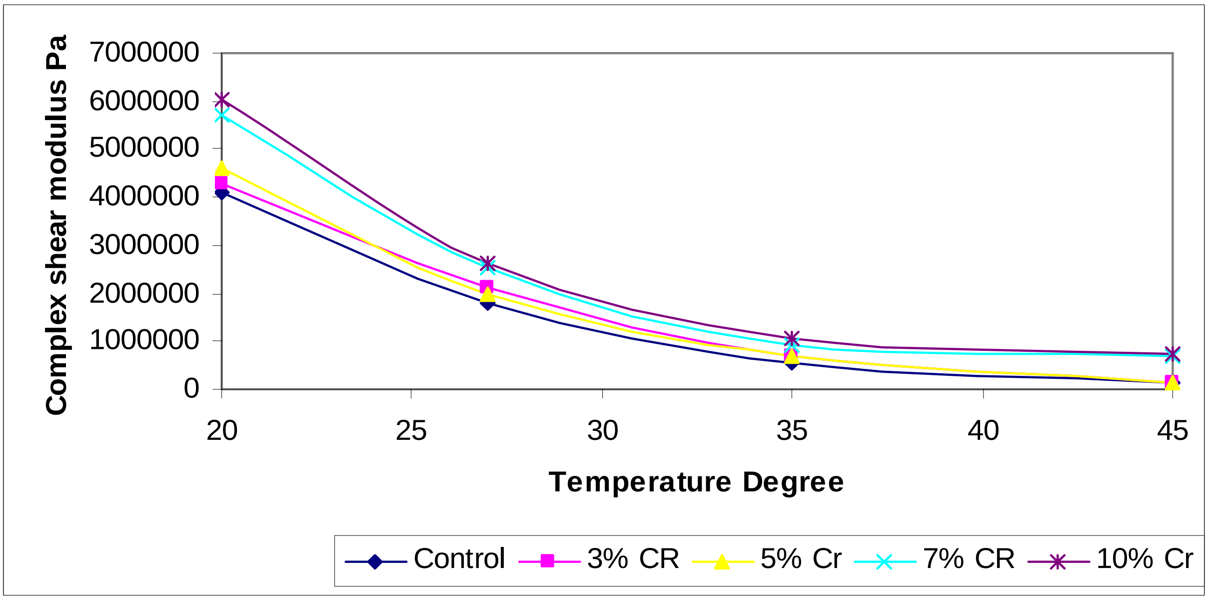


Figure (4-7) Complex shear modulus (G*) versus temperature for CRM bitumen at10 Hz..

It can be seen that there is little difference between the rheological properties verses crumb rubber modifier content from 3% and 5% during the frequency sweep test at the same temperature. However, as the modifier content increases, the rheological properties are significantly improved. Rheological properties are strongly depend on crumb rubber content and it clear that the reduction in complex shear modulus for modified binder is less than unmodified binder by the increase of temperature. In conclusion 10% CRM-bitumen can resist deformation at high temperature more than conventional bitumen. The higher rubber content seems to lead to an increase in complex shear modulus and decrease the phase angle, which is related to increase of the rubber mass through binder absorption.

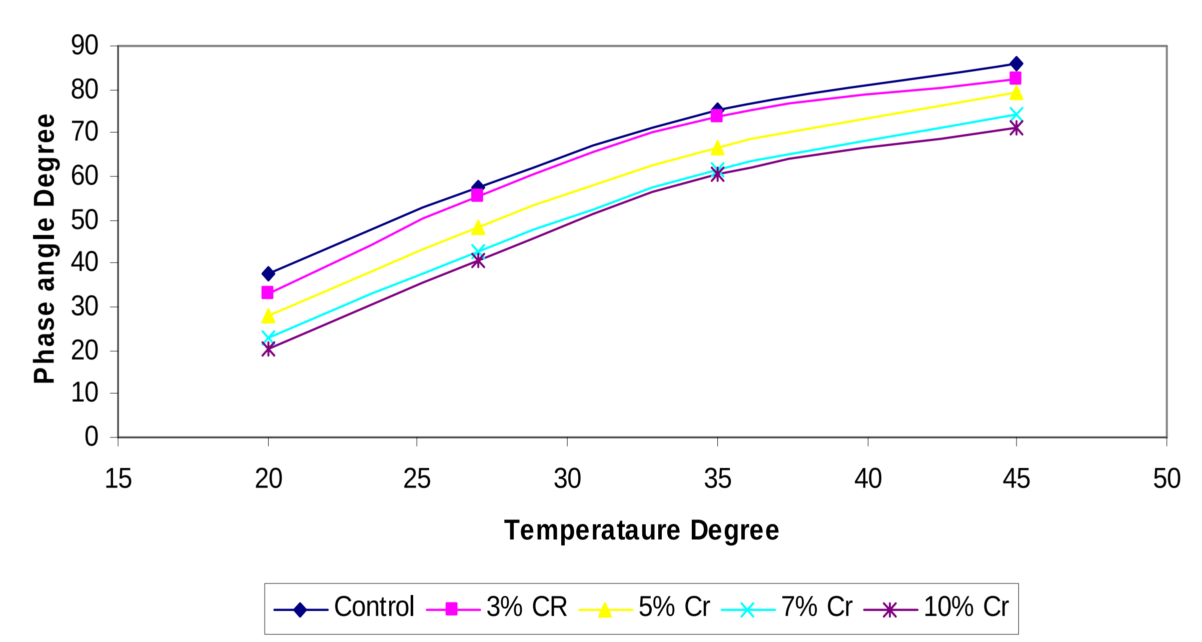


Figure (4-8) Phase angle versus temperature for CRM bitumen at 10 Hz

Comparison between modified and unmodified binder were held at 10 Hz. From the result it is clear that with the increase of temperature the complex shear modulus decrease. Complex shear modulus for modified bitumen is about 1.5 times higher than base bitumen at 20 ^{o}C. At high temperature 45 ^{o}C the difference between complex shear modulus for modified bitumen and base bitumen is about 1000 kPa. For 10% crumb rubber modified bitumen at intermediate temperatures or high frequencies asphalt binders tend to approach an intermitted value of G* of approximately 4.0 GPa and small value of phase angle δ of 20 degrees. At high temperature or low frequencies δ approaches 65 degrees where as well as the values of G* approximately 1.5 GPa. From the result it clear that adding crumb rubber to unmodified bitumen improves the complex shear modules which help to resist rutting. 10% crumb rubber give the best result this is because the oil in the bitumen completely solvate crumb rubber. The modified bitumen is absolutely stable and improvement in the rheological properties is due to the improvement in the elasticity and tensile strength of the bitumen. Using crumb rubber as asphalt modifier increases the binder elasticity at high temperatures and improves the flexibility at low temperatures. It is known that there are four chemical types are used to classify the composition of bitumen. Understanding the chemical composition help to know how the modifier affects the properties of the bitumen. The main chemical compositions are asphaltenes, resins, aromatics and saturates each component affect the properties of the bitumen. Resins, aromatics and saturates form the maltene fraction of the bitumen. To get harder bitumen with less fluid bitumen, asphaltenes content should be increased. When rubber is added to bitumen, the elastomer absorbs all the maltenes components of the bitumen, leaving the residual bitumen containing a higher proportion of asphaltenes. This is why the modified crumb rubber is harder than the base bitumen. The absorption of the lighter fractions of the bitumen into the crumb rubber and the subsequent changes in the rheology of the bitumen also have a detrimental effect on the cohesion the improvements in the rheological properties should lead to an increased resistance to asphalt rutting and cracking at high and low temperatures, respectively. 10% CRM bitumen has the highest complex shear modulus and lowest phase angle compare to the base bitumen as well as it have rubbery characteristics, when it stretched and released they will tend to recover their original shape and to make binder more flexible. Finally the result demonstrates the importance of adding modifiers to base bitumen on both fatigue and rutting.

4.3.2 Temperature Effects on Rheology of SBS Modified Bitumen

From figure (4-9) it can be seen that the modified binder has the highest service temperatures compared to the conventional bitumen. The service temperature is the temperature that the bitumen can give good performance without distress. The best known form using polymer as bitumen modifier is to improve the temperature susceptibility of bitumen by increasing binder stiffness at high service temperatures and reducing stiffness at low service temperatures. The modified binder has the highest service temperatures compared to the conventional. This does not mean that the complex modulus SBS modified bitumen does not decrease when temperature is increased, but that the elastic component is constant in a broader range of temperature than that of the base binder.

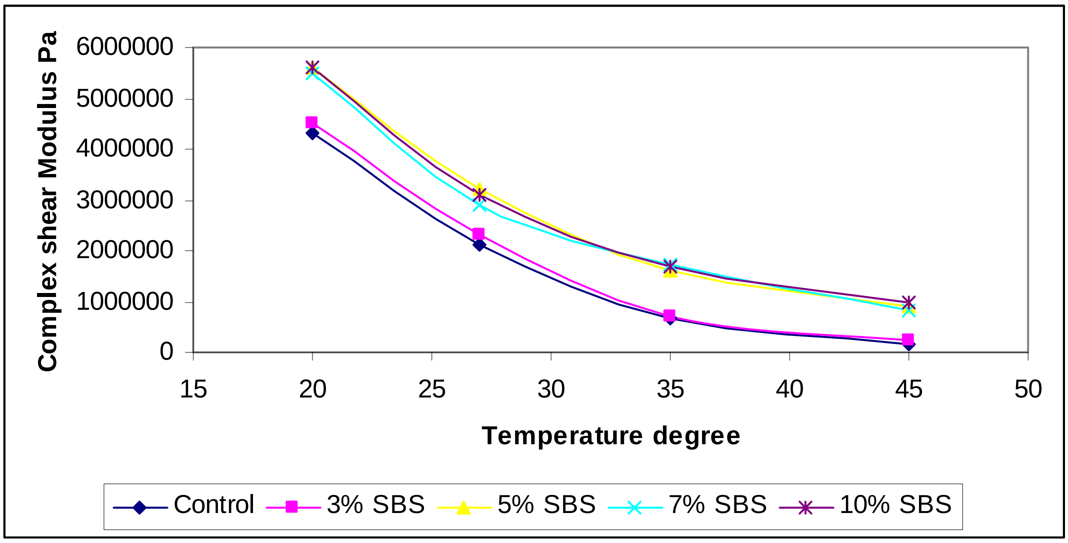


Figure (4-9) Complex shear modulus G* versus temperature for SBSPM bitumen 10 Hz.

The improvement in phase angle helps also to improve the service temperatures and this improvement depends on polymer type and concentration with greater effect by the bitumen type. When the modification levels are 3% SBS or 5% SBS the difference in phase angle between modified and unmodified bitumen at 20 ^{o}C is about 12 degree.

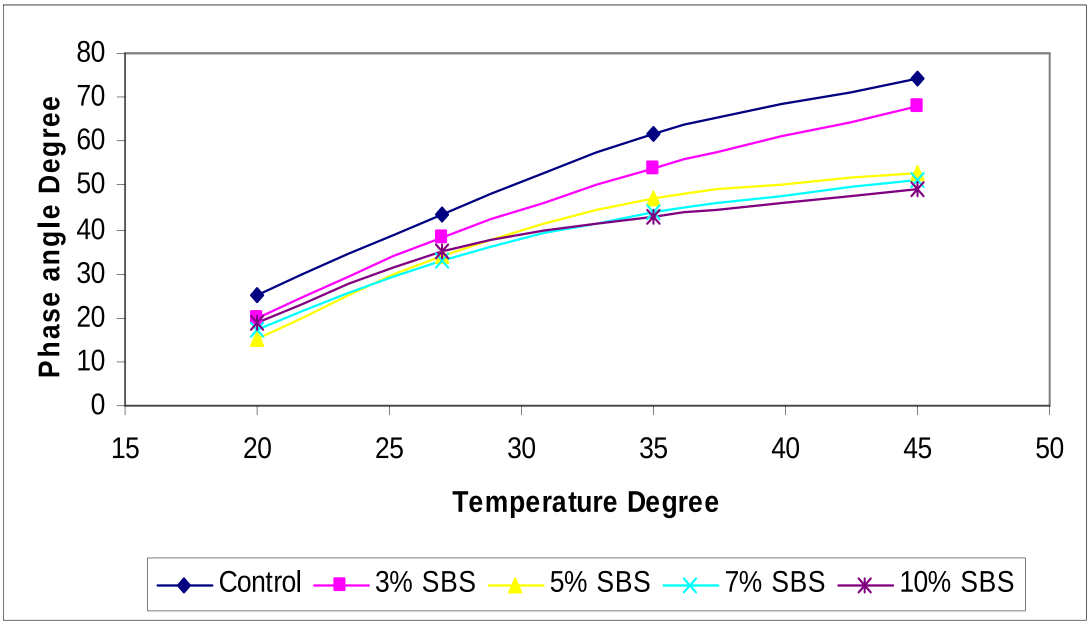


Figure (4-10) Phase Angle versus temperature for SBS PM Bitumen at 10 Hz

When the modification level is more than 5% SBS the difference in phase angle between modified and unmodified bitumen at 45 ^{o}C is nearly 30 ^{o}C. In conclusion SBS was found to improve the rheological properties of the base bitumen. This improvement is due to the formation of polymer net work in the base bitumen. From the result it is clear that the SBS modified bitumen have the ability to maintain their high strength and elasticity at high service temperature, this is due to the physical three dimensional cross linking network. It known that the styrene, provides the strength to the material, while the butadiene, provides the elasticity. The formation of the network depend on the modification level at low modification level SBS acts as a dispersed polymer and dose not significantly affect the properties.

At higher modification level, SBS network start to form and affect directly the rheological properties of the bitumen. With the increase in modification level stable network starts to from and less significant improvement in the rheologicl properties were achieved. In this study, a significant improvement in the rheological characteristics is observed when the SBS content is increased from 3% to 5% and by increase the of polymer content no real improvement occurs. Effects of SBS polymer modification as a decrease in Phase angle and an increase in complex shear modulus depends on the degree of modification. The magnitude and extent of these changes is a function of the base bitumen–polymer combination. Decrease in phase angle and increase in complex shear modulus indicate an increased hardness or stiffness of the modified binders. In addition to this increase in hardness indicate an improvement in temperature susceptibility with polymer modification. The final performance of modified bitumen is determined by chemical nature of base bitumen, source and grade as well as the polymer molecular weight and modification level.

4.4 Black diagram

Black diagram where the dynamic complex shear modulus is plotted as a function of the phase angle containing no reference to temperature or frequency. Individual data points from the frequency sweeps test result at different temperature level are simply plotted on the same graph which gives the change in phase angle with the change in complex modulus. The Black diagrams it can be noticed that the bitumen exhibits simple thermorheological behaviour. The complex shear modulus (G*) that decreases with the increase of the phase angle δ according to the bitumen types. Bitumen-Rubber and SBS-bitumen show very different behaviour compared to conventional bitumen in the Black Diagram and it is clear that the modified bitumen has better performance, especially when high temperatures/low frequencies are considered. According to the time temperature equivalence principle, low angular frequency corresponds to high temperature and high angular frequency corresponds to low temperature. The modified bitumen has a lower phase angle than the unmodified bitumen at low temperature, indicating that it can be more resistance to crack. At high temperatures the modified binder has high complex shear modulus which is taken as reference for stiffness that helps for resisting rutting. It can be seen that the addition of modifiers leads to a binder that performs better under high temperatures/low frequencies (lower values of phase angle) compared to unmodified bitumen.

At higher frequency values or low temperatures the ratio of increment of phase angle values due to modification becomes small. The main conclusion from the rheological properties of styrene–butadiene–styrene (SBS) and crumb rubber modified bitumen that the addition of styrene–butadiene–styrene (SBS) and Crumb Rubber (CR) help to produce a binder that performs more elastic under low temperatures / high frequencies compared to unmodified bitumen. The modifier improves bitumen response to loading at service temperatures and thus the expected performance of the modified bitumen. That means the modified binder can be used successfully at location of high stress and intersection of busy street.

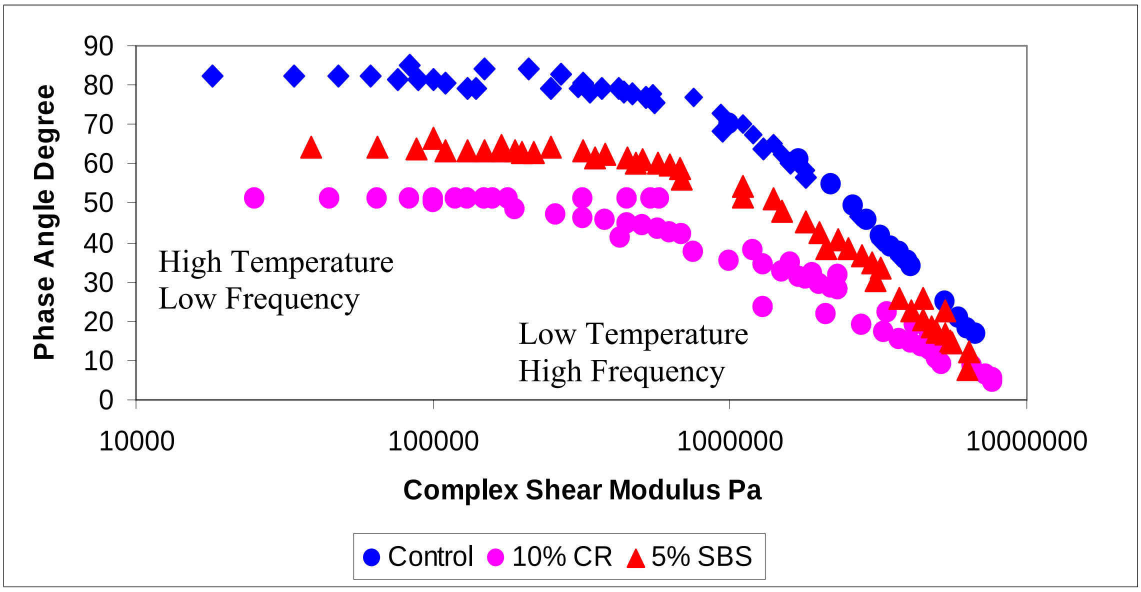


Figure (4-11) Black diagram phase angle versus complex shear modulus.

Based on the results of the experimental investigations conducted on base bitumen, CR modified bitumen and SBS modified bitumen, the following conclusions have been drawn. The change in the rheological properties of the CRM bitumen and SBS PM bitumen can be seen as an increased stiffness and elastic response in the bitumen. The main reason of the changes in the rheological properties of the modified bitumen that the modifier absorbers of the lighter fractions of the bitumen which make the bitumen stiffer. The change in rheological properties also has a detrimental effect on the cohesion properties. The properties of the modified bitumen is function of asphalt polymer net work formation, which is function of many variables including: asphalt composition; chemical structure of polymer; polymer molecular weight; physical properties of the polymer; the nature of the interaction between polymer and asphalt; asphaltene content in the bitumen and mechanical history of blending bitumen with polymer such as mixing time shear rate and mixing temperatures. To select the correct modifiers the modified bitumen should have the following properties:

- Low temperature susceptibility,
- Low loading time susceptibility,
- Better adhesion and cohesion properties.

4.5 Fatigue test result

The fatigue behavior was investigated by applying continuous oscillatory shear loadings using a DSR time sweep, with the 8 mm plate-plate set-up and all tests were done under controlled stress conditions. All testing was done at 10 Hz as the standard loading frequency representing traffic moving at normal highway speed.

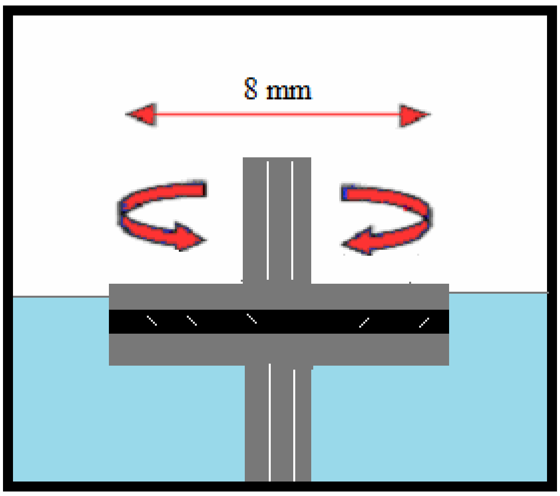


Figure (4-12) Bitumen specimen after time sweep test.

A detailed description of the DSR testes was presented in chapter 3. Bahia et al. (1999) established the proper procedure for testing of binders for their resistance to fatigue damage. The test consists of two steps; the first step is to find the initial input energy and the second step is to find the relation between the dissipated energy and number of cycles to failure. Control stress testing was selected as the most suitable test and it provides more clear measurement. The method involves the measurement of the deterioration of binder stiffness during a time sweep at the intermediate temperatures to provide a specific parameter used for binders fatigue resistance. The damage is characterized by a decrease in the energy dissipated per loading cycle and fatigue life of the binder is represented by the number of cycles required to undergo the crack initiation without reacting the crack propagation. The relation between dissipated energy and number of load cycles to failure is used to get the fatigue test parameter K1 and K2. It is reported that any small change in the test temperature directly lead to big difference in the test result. In order to subject the samples to constant temperature during test, water bath around the sample was used. The samples were subjected to 20°C therefore; all tested samples have the same testing temperature.

4.5.1 Initial Input Energy

Binder fatigue data could be analyzed using the dissipated energy ratio concept (DER) to estimate N_F (number of cycles to crack propagation). Bahia et al. (1999) reported that the binder has to be tested at different energy levels to determine the fatigue relationships.

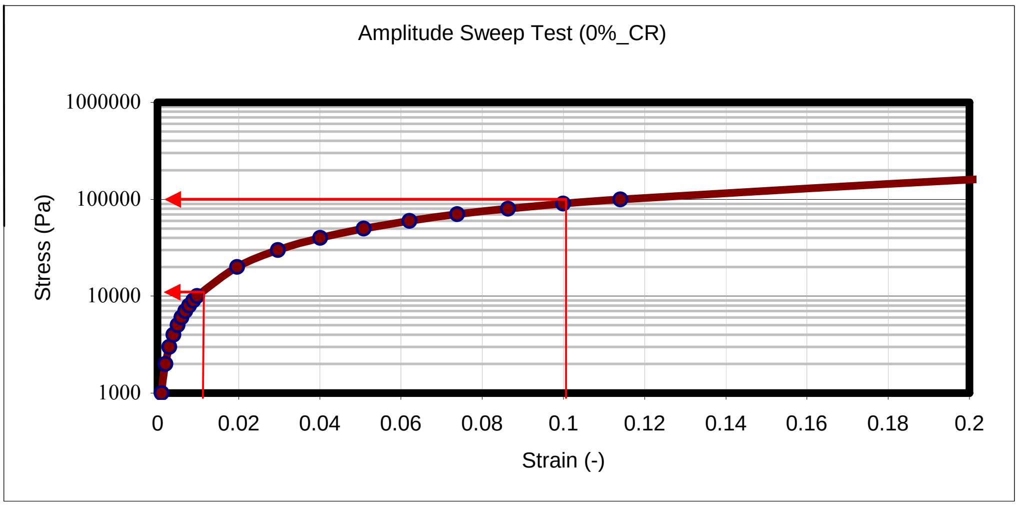


Figure (4-13) Stress versus strain for base bitumen

The concept of dissipated energy ratio allows the determination of the fatigue properties for both the asphalt binder and the asphalt concrete mixtures. The first input energy represents weak pavement where the binder is considered to be loaded in the non-linear range. The second input energy represents strong pavement where the binder is loaded in the linear range. To check the linear and non–linear properties of the binder amplitude sweep test was done. The strong pavement was defined as the middle of the linear limit, which meet the low strain and the weak pavement was picked up slightly above the linear limit, which meet high strain as shown in figure (4-13). The required frequency is related to the traffic speed for 60 m/h the testing frequency should be 10 Hz and the effect of frequency on fatigue life could be included the change of dissipated energy (Wi). To make the test easier 10 and 100 KPa was chosen as the input energy for all types of tested bitumen samples.

4.5.2 Dissipated Energy Ratio for Crumb Rubber Modified Bitumen

Modified and unmodified bitumen sample were tested in dynamic shear rheometer for large number of cycles until failure at 20 ^{o}C. Fatigue occurs in asphalt concrete pavement at intermediate to low temperatures. There are a lot of studies that had been done at a range of temperature between 15 ^{o}C to 30 ^{o}C, where asphalt binders are not brittle and exhibit moderate modulus values. The variation in the DER versus cycles of loading for a crumb rubber binder under cyclic stress controlled load at 10000 Pa and 100000 Pa are shown in Figure (4-14) and (4-15) respectively.

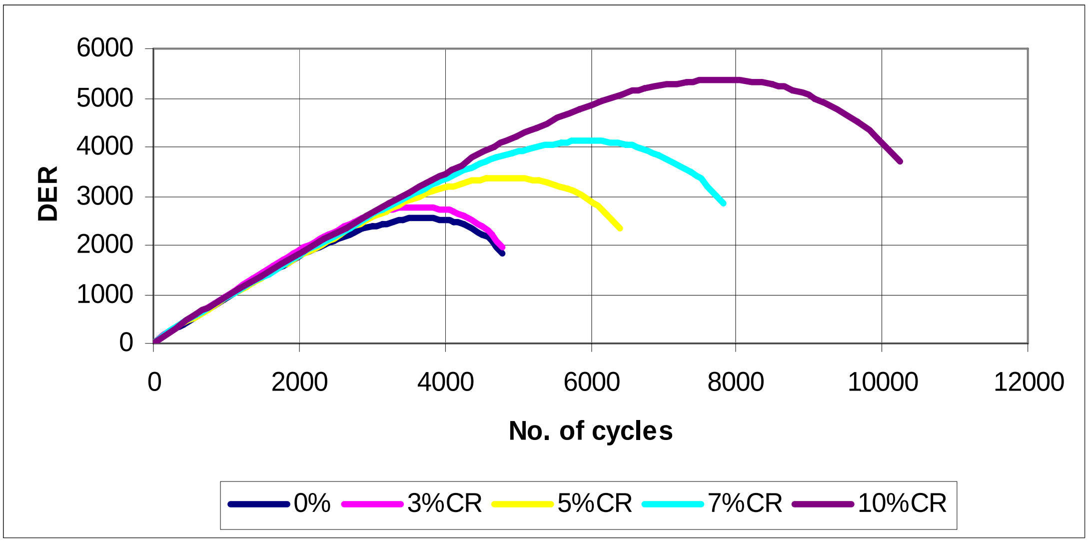


Figure (4-14): The variation in DER versus cycles of loading for crumb rubber modified bitumen at 10000 Pa.

Test results collected include measurements of complex modulus (G*) and phase angle (delta) as a function of the number of cycles in the DSR. The dissipated energy and dissipated energy ratio were calculated and were used to find number of cycles to failure. A lot of studies suggested that the best way to characterize the point when fatigue damage occurs is shown by using the concept of change in the dissipated energy per cycle. Once the test was done the dissipated energy ratio was calculated and the relation between dissipated energy and number of cycles. Excel work sheets were used to calculate dissipated energy and dissipated energy ratio to get number of cycles to failure. In the first stage of the time sweep test the dissipated energy per loading cycle remains constant. However, if the tested sample continues to be loaded and unloaded, the amount of the dissipated energy per cycle will change and then the failure point will be reached.

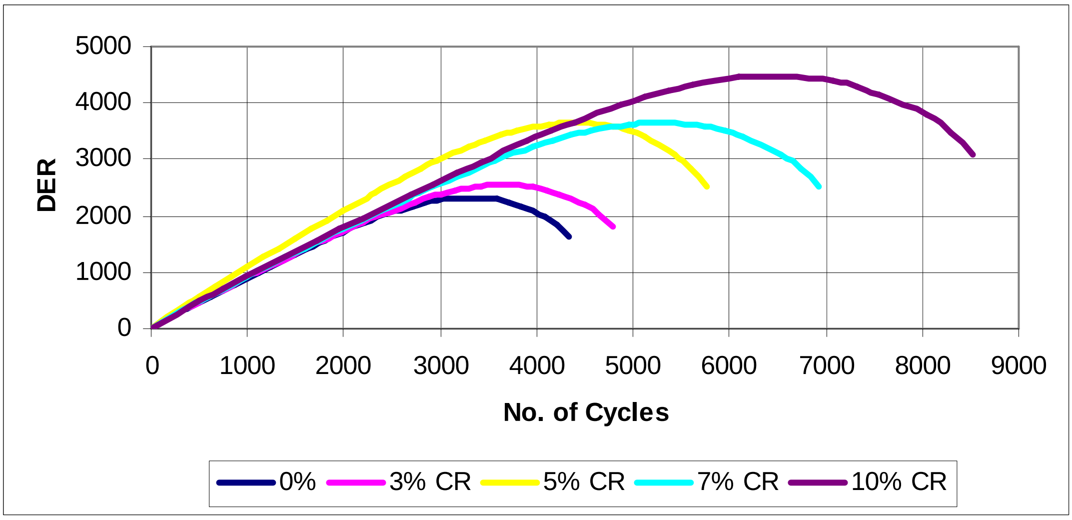


Figure (4-15): The variation in DER versus cycles of loading for crumb rubber modified bitumen at 100000 Pa

The value of Wn increases when fatigue damage is reached, so the DER decreases. The number of cycles for crack initiation where the trend line leaves the linear slope of the DER curve. The relation between DER and number of load cycles to failure were used to determine fatigue life parameter K1 and K2. Average of three samples was tested to evaluate fatigue life for crumb rubber modified bitumen and base bitumen. It is clear from figure (4-14) and (4-15) that by the increase of the crumb rubber modification level the number of load cycles to failure increase. It can be seen that all the binders show a steady behavior in terms of DER values for the first thousand cycles. However there is a certain point where DER starts to decrease, this is a sign of damage occurring in the binder. 10% crumb rubber has the highest number of load cycles to failure compare to the rest of modification level. The numbers of cycles to failure for 3% crumb rubber it is appear to be close to the number of load cycles to failure of base bitumen. The number of load cycles to failure decrease by increase of the initial input energy.

4.5.3 Determination of Fatigue Life Nf for CR Modified Bitumen.

From the result analysis given in figure (4-16), it can be seen that a linear fit is obtained between dissipated energy and number of cycles. The best approach to characterize binder fatigue is to use the classic power-law relationship relating number of cycles to failure to dissipated energy. Based on the linear regression analysis done on the logarithmic values, the relationship between Nf and DER at 20°C is given in table (4-1).

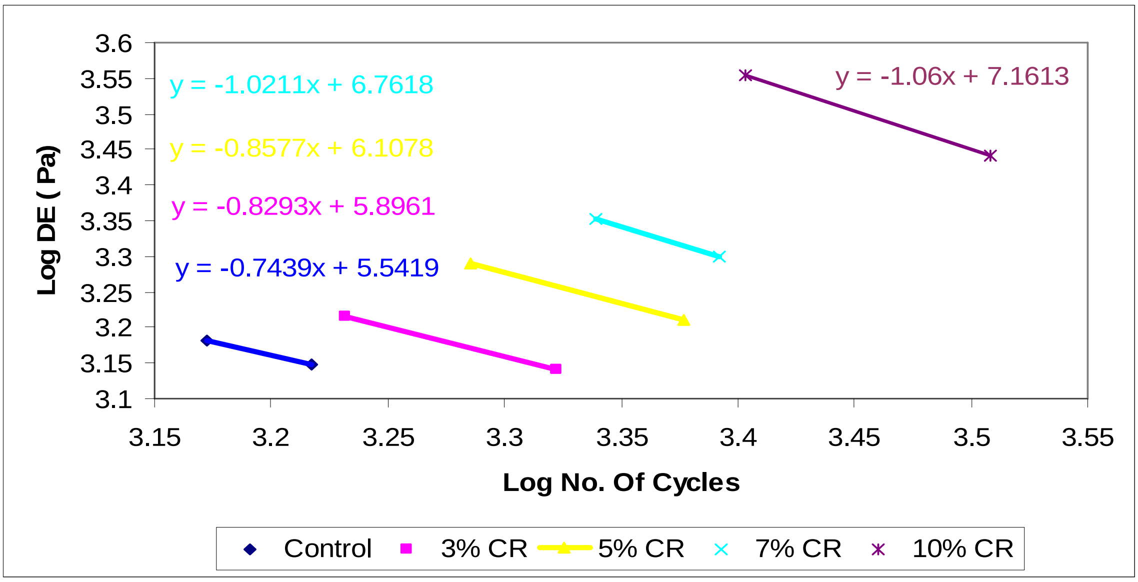


Figure (4-16): Relation between number of cycles and DE for crumb rubber modified bitumen

The fatigue trends were used to fit linear relationship and estimate the K1 and K2 factors at each temperature for each binder. K1 and K2 fatigue life power-law predictor parameters were replaced to calculate the fatigue lives of the binders for different levels of initial energy. After fatigue parameter determination, the fatigue relationship can be used to predict the fatigue life at any input energy for the selected pavement temperature. The comparison of the fatigue life at 20°C of the five modified and unmodified binders types shows that the 10%

modified crumb rubber performed better than the base bitumen for all loading conditions. The fatigue life of the 10% modified crumb rubber is more than 200% of the fatigue life for the base bitumen. The slope of the fatigue line (K2) of the 10% modified crumb rubber is larger than the slope of the base bitumen

Dissipated energy approach allows comparison of fatigue life at a given dissipated energy level that represents combinations of traffic load on pavement. The fatigue life is defined as the oscillation loading time when the specimen is broken (upper and lower plates are no longer connected). The results of the tests are presented in table (4-1).

Table (4-1) Fatigue Test Result for Crumb Rubber Modified Bitumen

Type of Bitumen	Number of cycles	DER	Log Fatigue life	Fatigue life	
Control	1649	1521	Log N = - 0.7439 Log (Wi) + 5,5419	$Nf = 0.348*10^6 (1/Wi)^{0.7439}$	
	1487	1345		$K_1=0.348*10^6$	$K_2=0.7439$
3% CR	2100	1706	Log N = -0.8293 Log (Wi) + 5.8961	$Nf = 0.782*10^6 (1/Wi)^{0.8293}$	
	1644	1449		$K_1=0.782*10^6$	$K_2=0.8293$
5% CR	2383	1930	Log N = -0.8577 Log (Wi) + 6.1078	$Nf = 1.281*10^6 (1/Wi)^{0.8577}$	
	1948	1742		$K_1=1.281*10^6$	$K_2=0.8577$
7% CR	2465	2250	Log N = -1.0211 Log (Wi) + 6.7618	$Nf = 5.778*10^6 (1/Wi)^{1.0211}$	
	2184	1928		$K_1= 5.778*10^6$	$K_2=1.0211$
10% CR	3225	2843	Log N = -1.06 Log (Wi) + 7.1613	$Nf = 14.971*10^6 (1/Wi)^{1.0623}$	
	3187	2769		$K_1= 14.971*10^6$	$K_2= 1.0623$

Fatigue parameter K_1 and K_2 is used to compare between two materials, the higher K1 the higher fatigue life and the lower K_2 higher dissipated energy the lower fatigue life. In conclusion from laboratory fatigue test for crumb rubber modified bitumen, the fatigue behavior bitumen was found to be significantly improved compared to base bitumen. Crumb rubber modified bitumen with 10% has the higher fatigue life; followed by 7% crumb rubber as observed from laboratory fatigue test results, is nearly two times. On the other hand, 3% and 5% crumb rubber modified bitumen shows no significant effect in fatigue life than the base bitumen.

4.5.4 Determination of Fatigue Life Nf for SBS Modified Bitumen.

All result are the average of three samples to evaluate the effect SBS in the fatigue resistance of asphalt binder. Once the test were done the dissipated energy ratio were calculated and the relation between dissipated energy and number of cycles were drown. The variation in the DER versus cycles of loading for a SBS modified bitumen under cyclic stress controlled load at 10000 and 100000 Pa are shown in Figure (4-17) and (4-18) respectively.

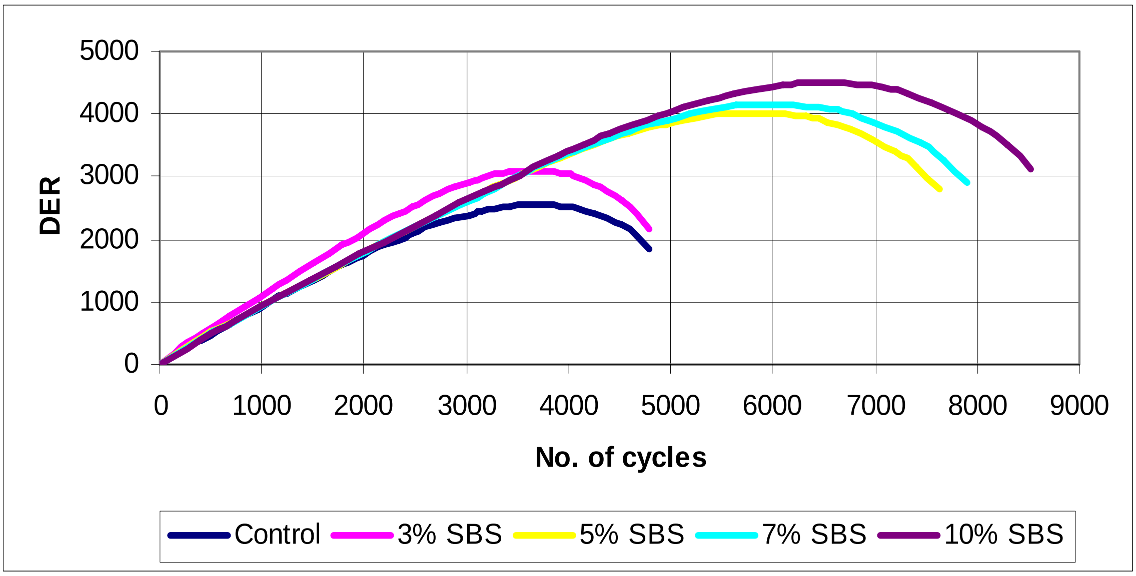


Figure (4-17): Relation between number of cycles and DER for SBS Modified Bitumen at 10000

From the dissipated energy ratio the number of cycles to failure was determined and it is used to draw the S-N curve, which used to determine the fatigue parameter such K1 and K2. Fatigue parameters were used to get the fatigue equation which is used to predict fatigue life for asphalt binder at different initial input energy. From figure (4-17) and (4-18) the number of load cycles to failure is determined from dissipated energy ratio and number of load cycles curve. The number of load cycles to failure is determined when the strait line leaves the curve and this point was chosen carefully. From the result analysis given in figure (4-18), it can be seen that the comparison of the fatigue life at 20 °C of the five modified and unmodified binders types shows that the 5% SBS modified bitumen performed better than the base bitumen for all loading conditions. The fatigue life of the 5% SBS modified bitumen is more than 180% of the fatigue life for the base bitumen. It is appearing from fatigue life curve that for different bitumen's, the fatigue lines looks to be parallel. This fatigue result can provide asphalt engineers modified bitumen with more fatigue resistance.

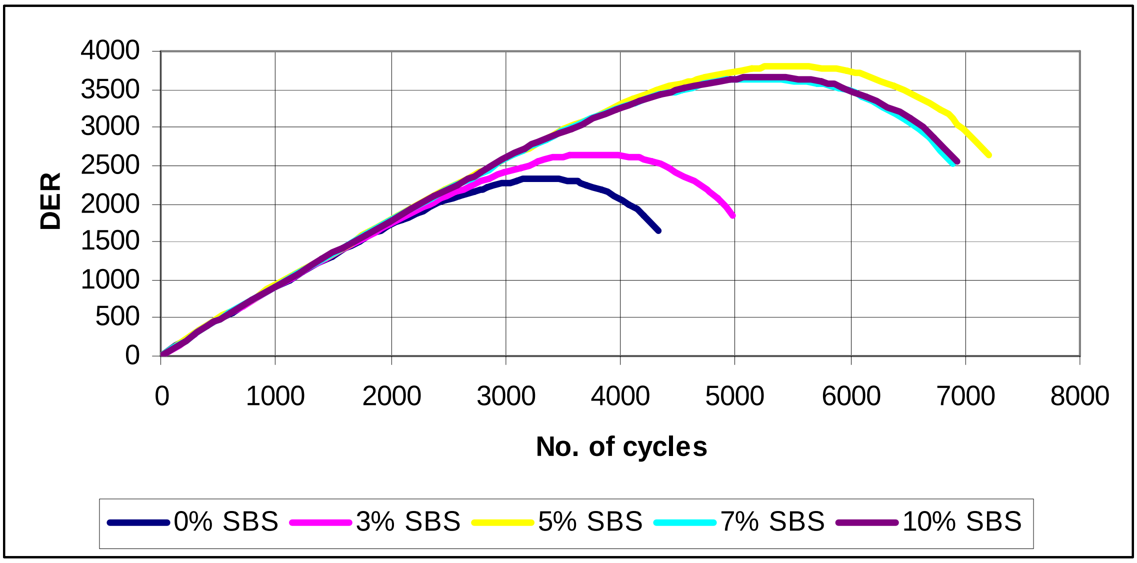


Figure (4-18): Relation between number of cycles and DER for SBS modified bitumen at 100000 Pa

Fatigue life relationships were also expressed in terms of initial dissipated energy for controlled stress tests and fatigue parameter. Fatigue general equation is presented in table (4-3) for modified and base bitumen. The K1-values range between $0.090*10^8$ and $40.888*10^8$. The K2-values for all fatigue test results vary between 1.1892 and 2.150. Comparison with fatigue results obtained by different researchers indicates that the differences are mainly in the K2-values.

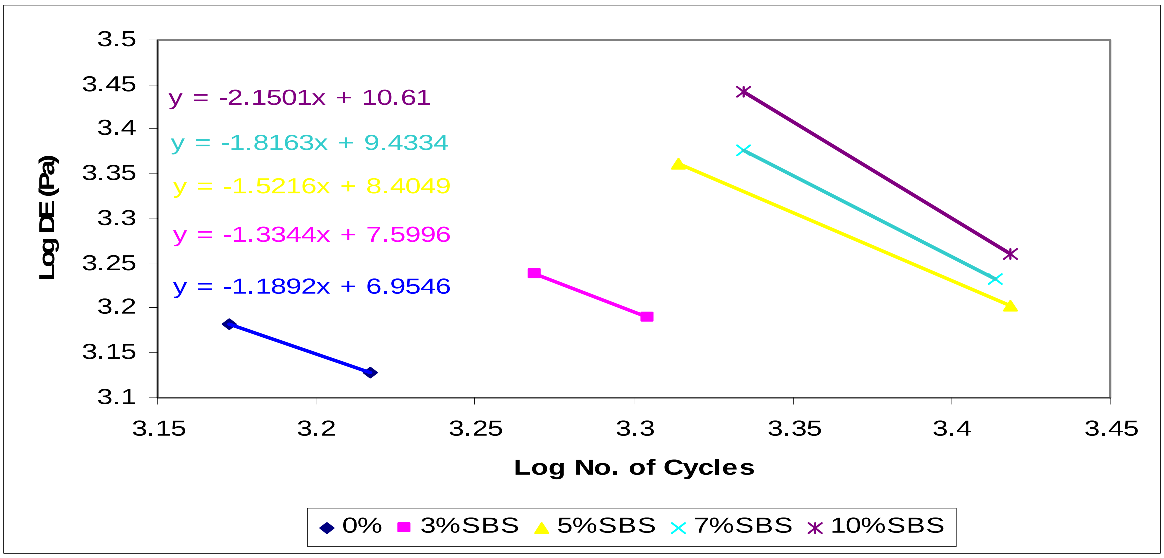


Figure (4-19): Relation between number of cycles and DE for SBS modified bitumen

Based on the linear regression analysis done on the logarithmic values, the relationship between number of cycles and dissipated energy at 20°C is presented in table (4-3).It is thought to be due to differences in loading and in environmental testing conditions, as well as the source of the tested materials. From the test result, dynamic shear rheometer is applicable for characterizing the rheological and fatigue properties of asphalt binder. All test results, which give indicator for pavement performance, suggested that crumb rubber modified binder give good fatigue resistance as well as SBS modified bitumen give less fatigue resistance than crumb rubber modified bitumen.

Table (4-3) Fatigue test result for crumb rubber modified bitumen

Type of Bitumen	Number of cycles	DER	Log Fatigue life	Fatigue life	
Control	1649	1521	Log N = - 1.1892 Log (Wi) + 6.9546	$Nf = 0.090*10^8(1/Wi)^{1.1892}$	
	1487	1345		$K_1=0.090*10^8$	$K_2=1.1892$
3% SBS	2621	1770	Log N = -1.3344 Log (Wi) + 7.5996	$Nf = 0.397*10^8$ (1/Wi) 1.3344	
	1856	1622		$K_1=0.397*10^8$	$K_2=1.3344$
5% SBS	2621	2301	Log N = -1.5216 Log (Wi) + 8.4049	$Nf = 2.540*10^8$ (1/Wi) 1.5216	
	2062	1834		$K_1=2.540*10^8$	$K_2=1.5216$
7% SBS	2715	2383	Log N = -1.8163 Log (Wi) + 9.4331	$Nf = 27.108*10^8$ (1/Wi) 1.8163	
	2170	1919		$K_1= 27.108*10^8$	$K_2=1.8163$
10% SBS	2445	2915	Log N = -2.1501 Log (Wi) + 10.6116	$Nf = 40.888*10^8$(1/Wi) 2.1501	
	2184	2763		$K_1= 40.888*10^8$	$K_2= 2.1501$

4.6 Phase Angle and Fatigue Resistance.

It is known that the phase angle is an indicator of the visco-elastic bahvior of asphalt binder. Figure (4-20) presents the relationship between phase angle for different modified and unmodified asphalt binder. It is clear that the number of load cycles to failure increase when phase angle is decrease. 10% crumb rubber has the lowest phase angle compare to base bitumen or SBS modified bitumen. The phase angle for base bitumen at 20 ^oC is about 1.5 times higher than the phase angle for 10% crumb rubber modified bitumen. Comparison between phase for 5% SBS modified bitumen and 10% crumb rubber were done. The phase angle for 5% SBS modified bitumen is about 37 degree with number of load cycles to failure about 2400 cycles.

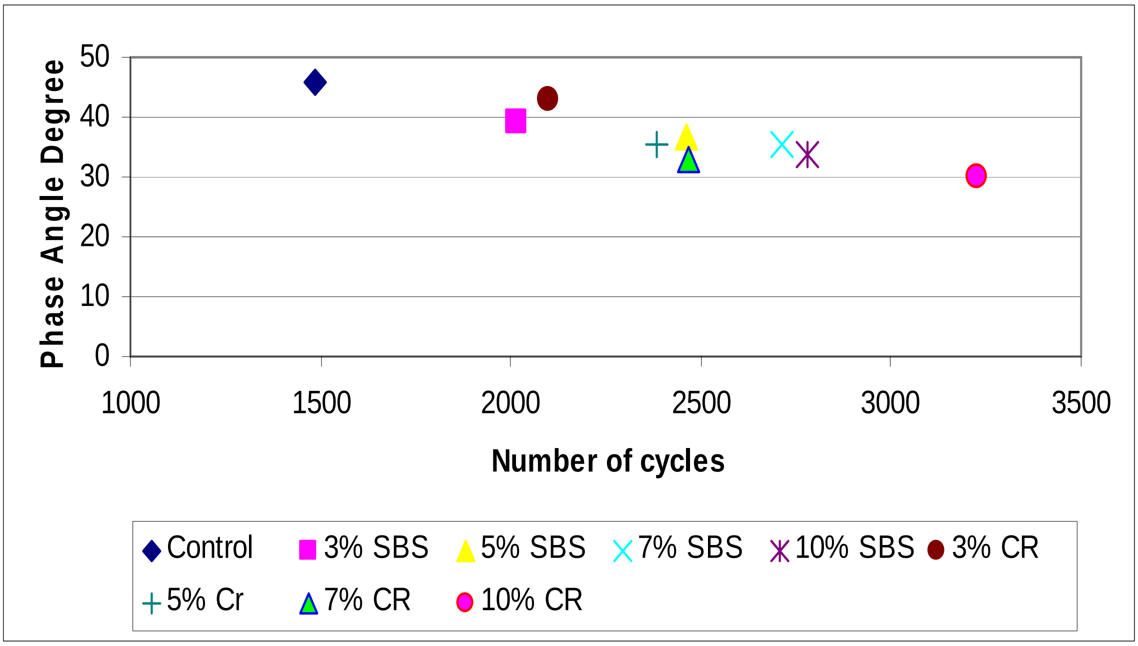


Figure (4-20) Relation between number of cycles and initial phase angle

On the other hand, the phase angle for 10% crumb rubber is about 30 degree and the number of load cycles is about 3500 cycles. The compassion gives indication that there is correlation between number of cycles to failure and phase angle. The conclusion Improvements in the fatigue properties of bitumen due to polymer modifications were indicated by improvements in complex shear modulus (stiffness) and increase the phase angle.

4.7 Cracked Surface after Fatigue Test

When the DSR Fatigue test was done a visual inspection of the specimens cracked surfaces were performed and photograph were taken. Figure (4-21) show a typical cracked surface of bitumen sample from DSR fatigue test. The crack surface is perfectly observed which help to understand the mechanism of DSR fatigue test. From all test result the crack formation depends on the modifier type and modification level. Small cracked surface area was observed when high percent of modifier is used.

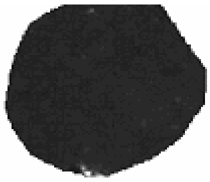
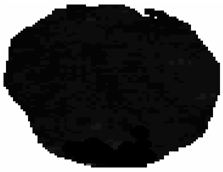
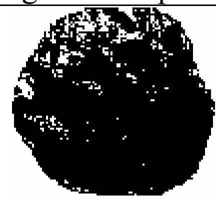
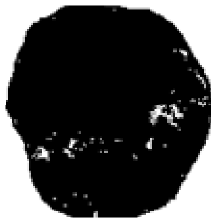
Crack after Fatigue Test	Crack after Fatigue Test using Image techniques
	
Unmodified Sample	Cracked area using Black and white Scale Image Techniques 0%
	
5% SBS	Cracked area using Black and white scale Image Techniques 5% SBS
	
10% Crumb rubber	Cracked area using Black and white scale Image Techniques 10% CR

Figure (4-21): Typical cracked surface area after fatigue test with image techniques

Additional of modifier to the base bitumen improve cohesion properties which improve the materials resistance to crack formation. Damaged surface area for unmodified bitumen is bigger than the damaged area for modified bitumen. Blackscale image techniques were used where the damaged area is marked by white colour and the undamaged area is marked with Black colour. The amount of cracked surface area is different according to the type and percentage of the modifier which were used. It is clear that the modified bitumen with 10% crumb rubber have small cracked area than the modified bitumen with 5% styrene–butadiene–styrene (SBS). Unmodified bitumen has the biggest cracked surface area compare to the modified bitumen with styrene–butadiene–styrene (SBS) and crumb rubber respectively. The black scale image technique is useful to evaluate the effect of modifier type and modification level on fatigue resistance. The bitumen which has less cracked surface area is expected to have long fatigue life. It is difficult to make the visual inspection for all tested sample because the tested sample were damaged and the cracked area is not clearly appear.

4.8 Rheoology and Fatigue Test Result Conclusion

The results of the research draw the main line to choose and modify base bitumen using different type of modifier. The DSR testing conducted in the laboratory study show important changes in the asphalt binder properties when the different types of modifiers were used. From the rheology result it is clear that the improvement in the bitumen properties depends on the modifier type and content as well as the type of the used base bitumen. On other hand, the result of the modified bitumen depends on the way of mixing modifier with bitumen, mixing temperatures and shear rate of the mixer. The rheoloical result will be helpful as input parameter in different finite element model as vicoelastic parameter. The output of the research is the modification methodology and evaluation of the modification effect on the rheoloical properties and fatigue resistance. All improvement was done to in the base bitumen may be satisfy the performance requirements of the Egyptian roads and the environmental condition. From the result it is appear that the modified bitumen with crumb rubber or styrene butadiene styrene nearly doubles characteristic required to resist fatigue cracking at intermediate temperature.

4.9 Finite Element Model

The disadvantage of using dynamic shear rheometre to evaluate fatigue resistance of bitumen that the test is costly and time consuming. 3D finite element model had been developed for dynamic shear rheometer that may be helpful to solve this problem. Excellent results were found from DSR model comparing to the experimental results (Abbas et al. 2004). The DSR model mechanism is a circular specimen mounted between two circular plates. Figure (4-22) Presents the Dynamic Shear Rheometer model using finite element program ABAQUS.

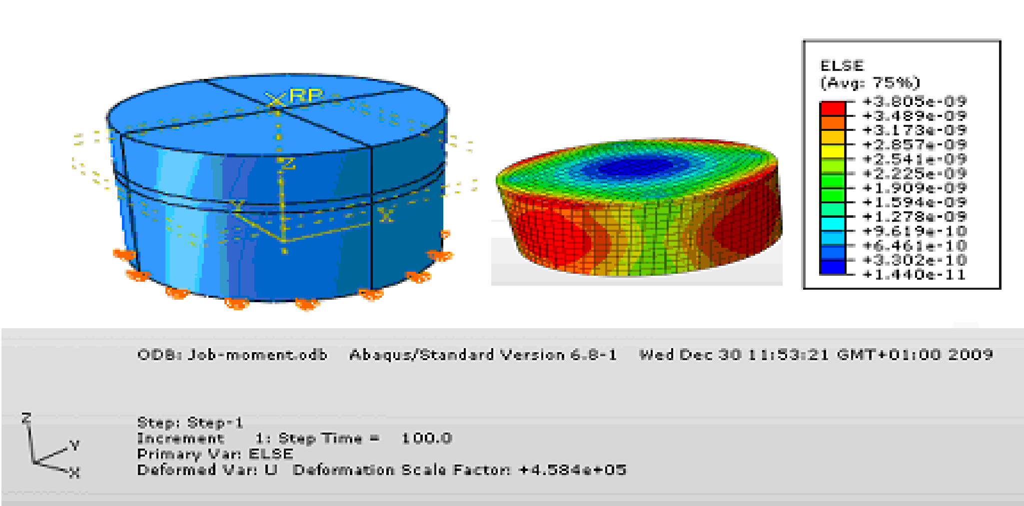


Figure (4-22): Strain energy distribution from dynamic shear rheometer model

The model is generated using ABAQUS with all needed and necessary elements. A dynamic oscillatory load, where sinusoidal shear stress is applied and transfer to the specimen as torque thought to the upper plat. The boundary condition let the upper plate free to rotate around a vertical axis plate and in the same time the lower plate is fixed. The materials input parameter was taking from the result of the first part bitumen rheology.

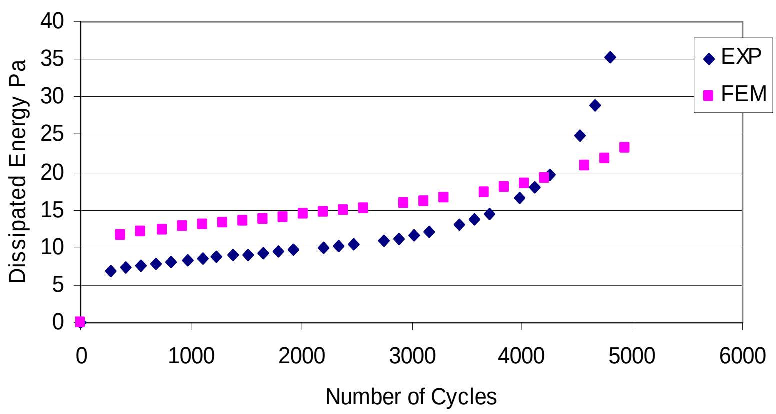


Figure (4-23): Dissipated Energy Result for Conventional Bitumen from Laboratory Results and DSR Model.

Figure (4-23) illustrate the relationship between dissipated energy from experimental test and finite element model for conventional bitumen. The experimental result and finite element result shows excellent fit between dissipated energy for conventional bitumen at the same test condition. The DSR model for asphalt binder can be used to solve the problem of test time and cost. It seems appropriate to used DSR model to calculate dissipated energy for different bitumen types.

Chapter 5

Experimental Design and Material Selection for Asphalt Concrete Mixtures

5. Introduction

The experimental program included in this research was aimed to study the effect of CR and SBS on the mechanical properties as well as fatigue resistance of asphalt concrete mixtures. This chapter provides detailed information about raw materials (utilized to prepare specimens) such as: aggregates and type of mix used in this research. In addition, this chapter highlights the laboratory procedures for the performed tests. The tests used in this study are indirect tensile strength, resilient modulus and fatigue tests. The test procedure tests are described in detail in the following sections.

5.1 Asphalt Concrete Mix Design Methods

Asphalt concrete mix design methods attempt to balance the composition of aggregate and asphalt binder to achieve long lasting performance in a pavement structure. 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. In addition, for the wearing surface, it is also necessary to provide surface texture and skid resistance.

Figure (5-1): Asphalt concrete mixtures (aggregate, asphalt binder and air voids)

The properties of the produced mixtures depend on the physical and chemical properties of the used materials. Each of the component materials needs to be carefully selected and controlled to ensure that they are of a suitable quality for the asphalt mixtures and the expected performance. Bruce Marshall created the Marshall method of asphalt mix design in the 1930's. Aggregate gradation was chosen as well as upper and lower limits for asphalt concrete mixtures 0/11 are presented in figure (5-1).

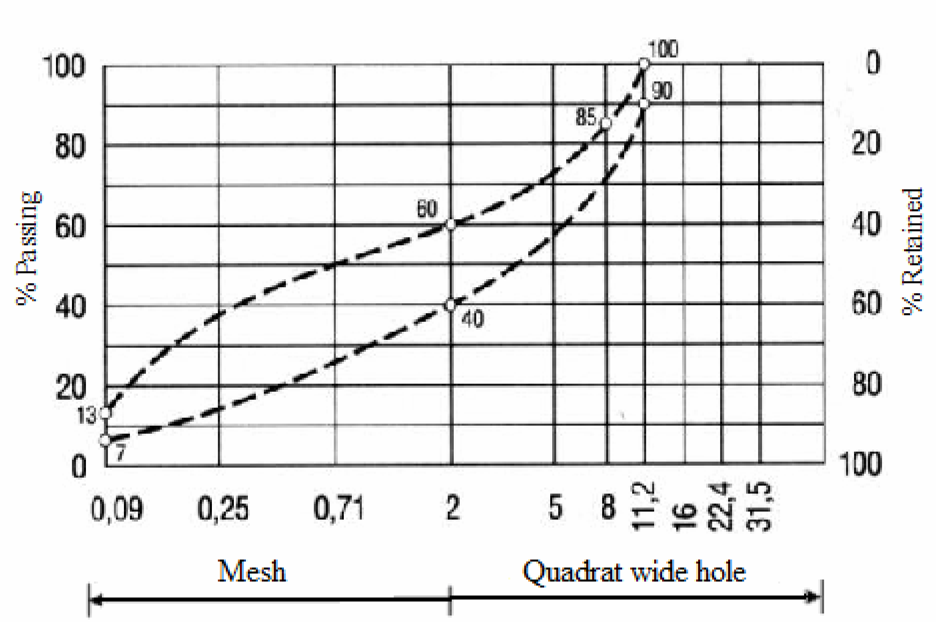


Figure (5-2): Aggregate gradation for asphalt concrete mixtures 0/11
(ZTV asphalt-StB 01)

The amount of aggregate should be calculated in order to satisfy the requirements of specification. Therefore, the properties of the used materials for asphalt concrete mixtures should be acceptable according to both German and Egyptian specification. Numerous factors and associated properties affect the ability of asphalt concrete mixtures to meet these structural requirements:

- Aggregate characteristics;
- Type of bitumen;
- Environmental condition;
- Load;
- Compaction method.

Marshall mix design method was used for the modified and unmodified asphalt concrete mixtures. An impact hammer was used to compact samples in a 102 mm diameter mould to a height of approximately 63.5 mm. Marshall mix design method pays proper attention to voids, strength, and durability (Roberts et al. 1996). The specifications for the designed mixtures are shown in Table (5-1).

Table (5-1): Specification Limits for Asphalt Concrete Mixtures (0/11)

Aggregate	Specification	Temperature °C 135 ± 5	Size of Aggregate	% By weight of Aggregate
< 0,09 mm	7-13		0 / 0,09	10
		% Air Voides ≤ 6,0	0,09/2,00	30
>2 mm	40-60		2,00 / 5,00	20
			5,00 / 8,00	20
> 5 mm	-----	Bitumen Type (70/100)	8,00 / 11,00	20
> 8 mm	≥ 15		Asphalt Content	6,2% -7,5 %
>11,2 mm	≤ 10			

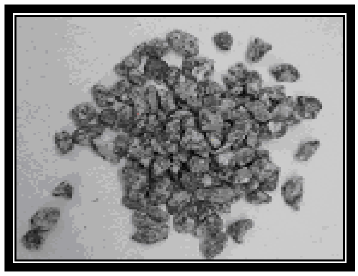
Due to its simplicity, Marshall Mix design method was the most commonly design method in the world. Marshall Mix design procedure consists of the following main elements:

- Selection of aggregates
- Selection of asphalt binder
- Preparation of asphalt mixture specimens
- Compaction of the asphalt mixtures
- Testing of the compacted Marshall specimens
- Computation of volumetric properties of the specimens
- Marshall Mix design criteria to determine the optimum asphalt content.

5.1.1 Aggregate

Aggregate make up 90 to 95% of asphalt concrete mixtures by weight or approximately 75 to 85% by volume. Strong aggregate structure provides asphalt concrete mixtures high resistance to deformation due to repeated load application. As well as aggregate provides most of the loading support in asphalt pavement. Therefore, aggregate and gradations have been shown to have a significant impact on the strength and performance of asphalt concrete mixtures. It is known and clear that the physical properties of aggregates significantly affect the performance of asphalt concrete pavement in service. Aggregates being considered for use in hot mix asphalt concrete should be clean and free of undesirable materials, such as lightweight particles. Aggregates are usually categorized as coarse aggregate and fine aggregate. Coarse aggregate is the mineral aggregate retained on the 2.36-mm sieve. It is produced by crushing rock or natural gravel to obtain angular, rough-textured particles with good mechanical interlock. Sand and course aggregate are the bearing components within asphalt mix. There are many factories affect the bond between the asphalt binder and aggregate such as the particle shape (angular or round), the absorption capacity (porous or not) and the degree of acidity. The basic aggregate properties were determined such as specific gravity, gradation, and all needed properties.

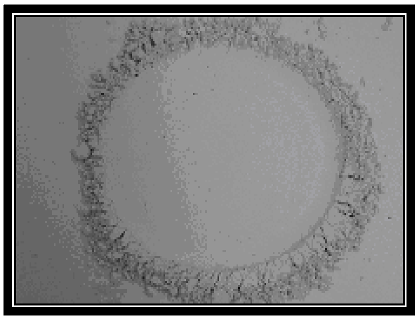
(8 – 11) mm



(5-8) mm

(2-5) mm

(0,09 – 2) mm



(0 - 0,09) mm

Figure (5-3): Coarse aggregate and fine aggregate used in asphalt concrete mixtures (0/11)

Aggregate gradation is the distribution of aggregate particle sizes expressed as a percent of the total dry weight of the aggregate. Aggregate gradation affects many important properties of an asphalt concrete mix including stiffness, stability, durability, permeability, workability, fatigue resistance, frictional resistance, and moisture damage resistance (Roberts et al. 1996). Fine aggregate is material passing the 2.36-mm sieve; it can be crushed rock or natural sand. Filler is a fine material, the majority of which passes a 75µm sieve derived from aggregate or other similar granular material. Hard aggregates (crushed granite) were used to produce modified and unmodified asphalt concrete mixtures. The Physical properties of granite aggregate are evaluated related to the European specification. The results from all of these tests are tabulated as shown in table (5-1). Mineral fillers have traditionally been used in asphalt mixtures to fill the voids between the larger aggregate particles. Generally, the aggregate material passing the No.200 sieve is referred to be as filler. In this study Kalk was used as filer, which is fine-grained mineral powder with particles diameter smaller than 63 µm. Filler not only affect the handling of the asphalt mix but also improve mechanical properties.

Table (5-1): Physical Properties of Granite

Granite Properties	Specification
Density (Kg/Cm^3)	2,6-2,8
Porosity (V. %)	0,4-1,5
Water absorption (W. %)	0,2-0,5
Compressive strength (N/mm^2)	160-240
Flexural strength (N/mm^2)	10-20

There are a lot of reasons for using filler in asphalt concrete mixtures. Fillers are used to fill voids and to improve the bond between asphalt cement and aggregate which increase the mixtures stability. Labib, M.E. (1992) reported that a significant number of studies have shown that the interaction between asphalt binders and aggregates in pavements directly affect the adhesion and bond strength which consequently affect the level of crack formation upon traffic loading. Fatigue behavior of asphalt concrete mixtures was found to be insensitive to the geometric characteristics of coarse aggregates and their gradation. The air voids content have a significant effect on the fatigue lives of asphalt concrete pavement. To achieve optimum air voids, the aggregate shape, texture, and angularity have to be carefully considered. The amount of asphalt that can be absorbed in a mix is dependent on the surface texture of the mix. Rough surface textured aggregates provide good bonding between the asphalt and the aggregates, and such good adhesion is necessary for fatigue resistance

Table (5-2): Sieve Analysis of Granite under (DIN EN 933-1)

Sieve Size (mm)	Wet sieving **(0/2)** % Passing	**Specification**	Dry Sieving **(2/5)** % Passing	**Specification**	Dry Sieving **(5/8)** % Passing	**Specification**	Dry Sieving **(8/11)** % Passing	**Specification**
22,4							100	**100**
16					100	**100**	100	**98-100**
11,2			100	**100**	100	**98-100**	93,8	**90-99**
8			100	**98-100**	88,5	**90-99**	85	**0-15**
5,6			97	**90-99**	12,9	**0-20**		
4	100	**100**					0,65	**0-5**
2,8	99,9	**98-100**			1,2	**0-5**		
2	97,9	**85-98**	1,1	**0-10**				
1	75,1	**73±10**	0,65	**0-5**				
0,063	6,4	**7±3**						

5.1.2 Bitumen Type

Bitumen and filler acts as an adhesive agent that binds aggregate particles into a cohesive mass. When bound by asphalt cement binder, mineral aggregate acts as a stone framework that provides strength and toughness to the system. Bitumen is a thermoplastic material whose strength and physical behavioural properties are directly related to temperature. Bitumen type and its properties are presented in chapter 3.

5.1.3 Asphalt Concrete Mixtures Design and Calculation

Asphalt concrete mixture was designed based on the European Standard EN 12697 (Bituminous mixtures -Test methods for hot mix asphalt - Part 34: Marshall Test). Once the aggregate and the asphalt cement grade are selected, trial samples are compacted in the lab at various asphalt cement contents above and below the expected optimum. Samples of asphalt mixtures at five different asphalt contents, with three replicates per asphalt content are prepared. The aggregates and the asphalt are heated in the oven at a temperature of 165 °C for about 3 hours. The mixing bucket and the spatulas are also heated to this temperature. After the aggregate and asphalt reached the required mixing temperature directly removed from the oven and mixed in the bucket until a uniform and complete coating of the aggregate is achieved. The amount of asphalt used for each of the five samples relates to the estimated optimum asphalt content, Pb-1.0, Pb-0.5, Pb, Pb+0.5 and Pb+1. The mixtures were placed on scale to weigh out the required amount of asphalt mixtures for one marshal specimen. The asphalt mixture is compacted by a Marshall compaction hammer, which is 4.5 kg (10 pounds) in weight and dropped from a height of 457 mm (18 inches) for a specified number of blows per side of the specimen. The compaction blows depending on the traffic voulme and the number of blows to be applied per side is 35, 50 or 75 for light, medium or heavy designed traffic, respectively.

The bulk specific gravity of the compacted mixtures should be determined before Marshall test. The bulk specific gravity test consists of weighing the specimen in air and in water. In order to determine the volume of the specimen, including open voids in the surface, the mass is also determined after the specimen has been immersed in water and its surface blotted with a damp towel to dry the surface without removing the water in the surface voids. Finally, each cylindrical specimen is subjected to a stability and flow test in a Marshall testing machine. The maximum load resistance (Newton) is recorded as the 'Marshall stability'. The Marshall flow is the total vertical deformation of the specimen, in units of 0.01 inch, when it is loaded to the maximum load in the Marshall Stability test. Figure (5-4) presented Marshall stability and flow measurement relationship.

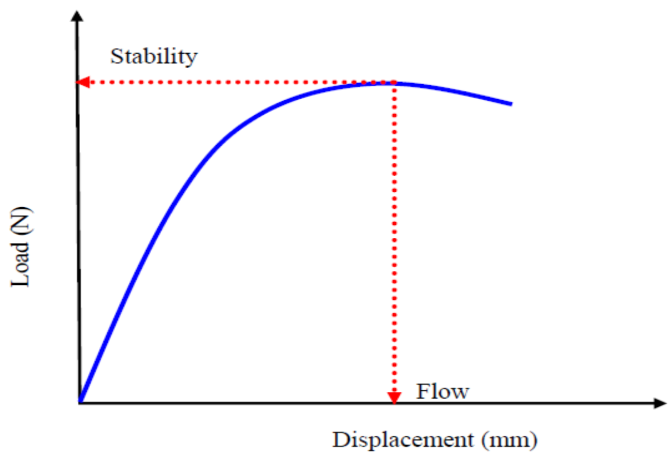


Figure (5-4): Marshall stability and flow measurement relationship.

5.1.4 Computation of volumetric properties of the specimens

The void characteristics of bituminous specimens were determined according to the European standard EN 12697 (Bituminous mixtures -Test methods for hot mix asphalt - Part 8: Determination of void characteristics of bituminous specimens). The volumetric properties associated with the combination of mineral aggregates, asphalt and air are widely used for mix design and production control. The mixtures were then further analyzed to determine the rest of volumetric and physical properties at the design asphalt content.

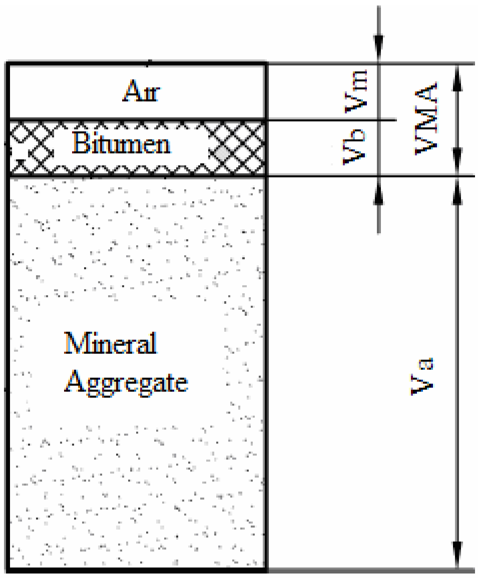


Figure (5-5): Phase diagram of a compacted asphalt mixture (European Standard EN 12697)

Where:

VMA : Volume of voids in mineral aggregate

V_m : Air voids content of the specimen

V_a : Volume of aggregate

V_b : Volume of bitumen

Physical volumetric properties are commonly used when designing asphalt concrete. Appropriate VMA is required to provide space in the mix for enough asphalt cement to achieve proper aggregate coating and bonding, as well as to leave air voids for the thermal expansion of asphalt cement during high in-service temperatures. VMA in a compacted asphalt concrete has two components: the volume of voids that is filled with asphalt cement, and remaining voids filled with air. Definition and calculation of percentage of the voids in the mineral aggregate according to the European standard are in Appendix (A).

5.1.5 Marshall Mix Design Criteria

Marshal mix design criteria are presented in table (5-3).

The Marshall Mix design method as recommended by the Asphalt Institute uses five mix design criteria.

1. Minimum Marshall stability,
2. Range of acceptable Marshall flow,
3. Range of acceptable air voids,
4. Percent voids filled with asphalt (VFA), and
5. Minimum amount of VMA.

5.1.6 Optimum Asphalt Content

The optimum asphalt content according to the Marshall mix design is chosen based on examining volumetric properties of the specimens as well as their stability and flow test results. The optimum asphalt content was determined according to the procedure shown before and all date are presented in table (5-3).

- Average unit weight versus asphalt content
- Average air voids versus asphalt content
- Average Marshall stability versus asphalt content
- Average Marshall flow versus asphalt content
- Average VMA versus asphalt content

The relation between asphalt content and asphalt concrete mixtures properties are presented in figure (5-5).

Table (5-3): Computation of Volumetric Properties of Marshall Specimens

Sp. No.	%Asphalt Cement	Weight of Specimen in Air	Weight of Specimen in Water	Gmb	GMM	% Air voids	%VMA	% VFA	Marshall Stability kN	Flow (mm)
1	5.5	1300,6	770,7	2,447					7.3	4.1
2	5.5	1290,7	767,6	2,459					7.7	3.9
3	5.5	1291,7	764,1	2,439					7.5	4
	Average			**2,448**	**2,588**	**4,9**	**14,6**	**65,1**	**7,5**	**4**
4	6	1289,7	765,0	2,488					8.6	4.8
5	6	1298,3	769,0	2,490					8.4	5.1
6	6	1296,2	766,8	2,489					8.8	5.1
	Average			**2,489**	**2,583**	**3,5**	**14,1**	**76,7**	**8,6**	**5**
7	6.5	1297,0	765,2	2,565					8.9	7.3
8	6.5	1293,3	763,2	2,563					8.9	6.9
9	6.5	1289,0	761,5	2,567					9.2	6.8
	Average			**2,565**	**2,621**	**3,1**	**13,9**	**86,4**	**9**	**7**
10	7	1291,6	759,0	2,526					8.0	10
11	7	1284,3	752,7	2,558					7.9	10.2
12	7	1292,0	761,3	2,538					8.1	9.8
	Average			**2,549**	**2,586**	**1,9**	**14,5**	**89,3**	**8**	**10**
13	7.5	1281,6	750,4	2,498					6.3	10.8
14	7.5	1281,9	747,2	2,488					5.8	10.9
15	7.5	1282,4	751,3	2,546					5.9	11.3
	Average			**2,510**	**2,539**	**1,1**	**15,1**	**91,3**	**6**	**11**

From the test property curves, plotted as described above and general note are outline:

- The satiability value increase with the increase of asphalt content up to a maximum and then start to decrease again.
- The flow value increases with increase of asphalt content.
- The curve of unit weight for total mix is similar to the stability curve.
- The percentage of air voids in total mix decrease with increase of asphalt content
- The percentage of aggregate voids filled with asphalt increase with the increase of asphalt content.

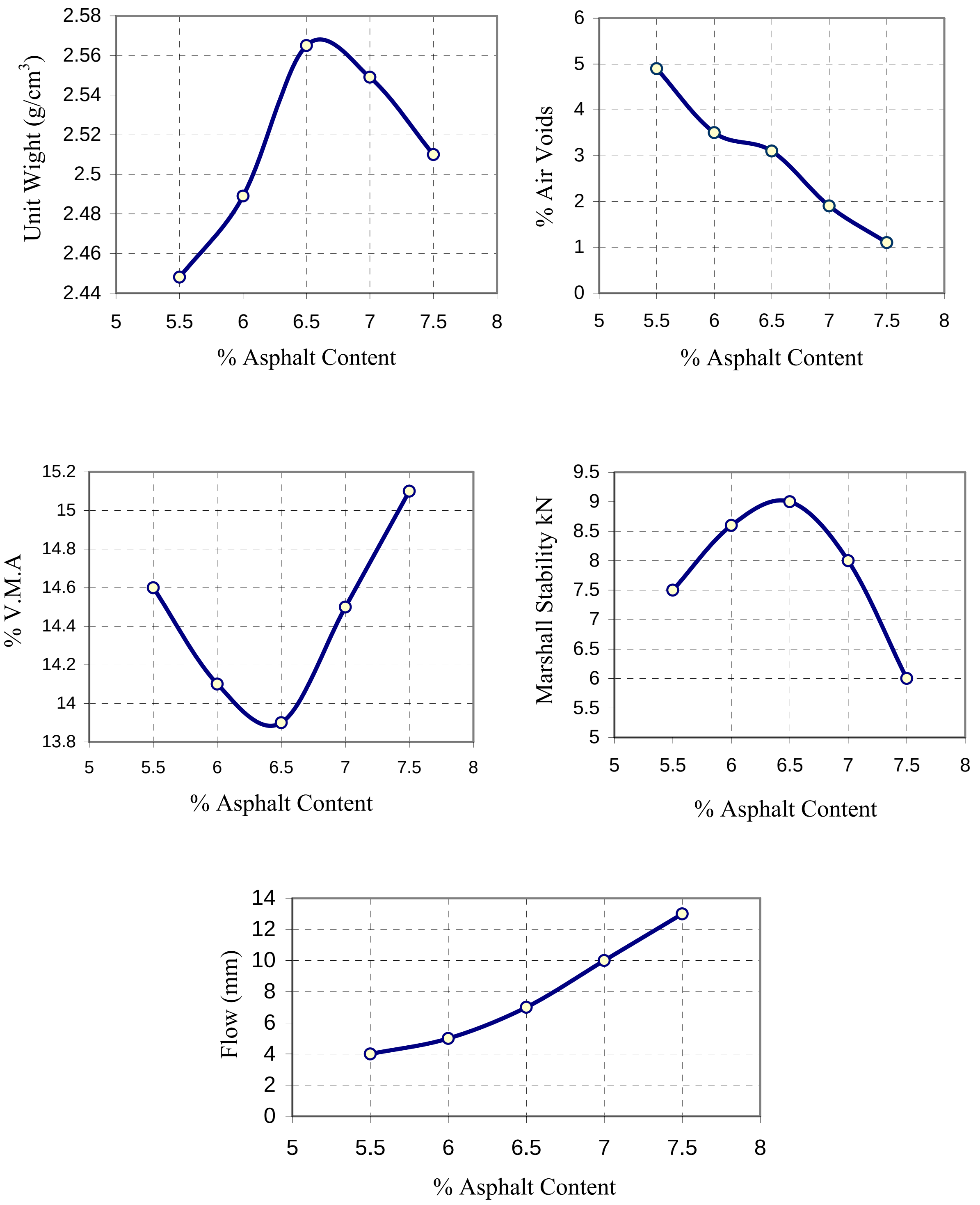


Figure (5-6): Design of Asphalt Concrete Mixtures (Marshall Method)

5.2.0 Laboratory Test Procedures:

The experimental program has been divided into two phases that help to understand and predict asphalt binder and mixture properties. In the first phase, different polymers were blended with asphalt binder and found out the improvement in the bitumen properties, based on the comparison between modified and unmodified bitumen. In the second phase, base asphalt and polymer modified asphalt was mixed with aggregate. A comprehensive laboratory evaluation was conducted on the designed mixtures. Therefore, a suite of mechanistic tests were performed to study the behavior of asphalt mixtures under various loading and environmental conditions. Asphalt concrete mixtures tests consist of three mechanical tests, indirect tensile strength (ITS), indirect tensile resilient modulus (ITMr), and fatigue test.

5.2.1 Indirect Tensile Strength

The indirect tensile test has been used extensively in structural design research for flexible Pavements since the 1960's. A cylindrical specimen is loaded diametrically across the circular cross section. The indirect tensile strength test is performed to determine the tensile strength of the bituminous mixtures. In this study the Indirect tensile test were done according to the European Standard EN 12697-23 (Test Method for hot asphalt mix indirect tensile strength). Figure (5-8) presents indirect tensile test Specimen with load balancing and load stripe pattern.

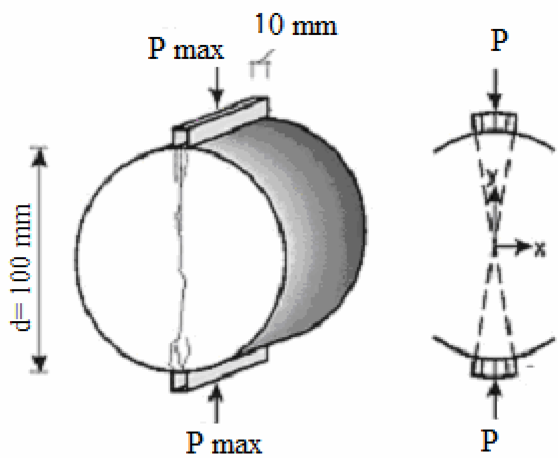


Figure (5-7) Indirect tensile test specimen with load balancing after Grätz (1996)

In the indirect tensile test the load causes a tensile deformation perpendicular to the loading direction, which yields a tensile failure. By registering the ultimate load and by knowing the dimensions of the specimen, the indirect tensile strength of the material can be computed. Witczak, M. W. (2002) reported that the indirect tensile test is one of the most popular tests used for asphalt concrete mixtures characterization in evaluating pavement structures. The primary reason for its popularity is that the tested sample from the field can be tested directly in the lab. The dimension of the sample is 101.6 mm diameter and 63.5 mm height with load to failure along the diametrical plane of the sample. Diametric load is applied continuously at the constant rate of deformation until the peak load is reached, at which point the specimen fractures. Indirect Tensile Strength test is used to determine failure limits as tensile strength and fracture energy. As well as indirect tensile strength (ITS) test may be very useful in understanding the tensile strength characteristics and in predicting the crack appearance in the mixture. The European Standard (EN 12697-23) provides clear description for the test procedure to determine the tensile strength value. In the indirect tensile strength test, a Marshall sample is subjected to compressive loads between two loading strips, which create tensile stress, along the vertical diametric plane causing a splitting failure. Figure (5-8) presented the stress distribution on X-axis and Y-axis for indirect tensile test specimen.

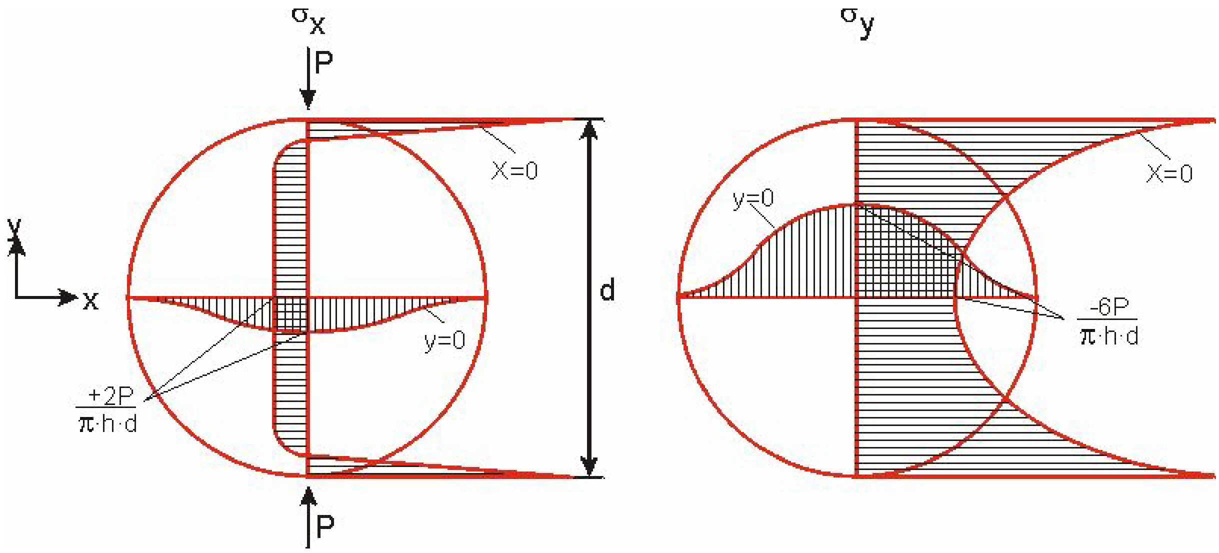


Figure (5-8): Stress distribution in the indirect tensile test specimen after (Hadley, 1970)

The maximum tensile strength calculated from the peak load applied at break and the dimensions of the specimen. The test procedures are presented as shown below:

- The test temperature selected to be 5 °C and 20 °C and the specimen can be loaded diametrically
- Loading strips consisting of 13 x 13 mm (0.5 x 0.5 in.) square steel bars for 102 mm (4 in.) diameter specimens
- The diametrical load is applied continuously with constant speed of deformation of (50 ± 2) mm/min.
- The peak load is reached when the specimen breaks and the type of failure is categorised as :
    a) Clear tensile break where the specimens are clearly broken along a diametrical line, for small triangular sections close to the loading strips.
    b) Deformation where the Specimens are without a clearly visible tensile break line.
    c) Combination where the specimens are with a limited tensile break line and larger deformed areas close to the loading strips.

The type of failure recorded may be help to understand the crack mechanism and to provide real comparison between the testes materials. Failure Type for asphalt concrete specimen from indirect tensile strength presented in figure (5-9).

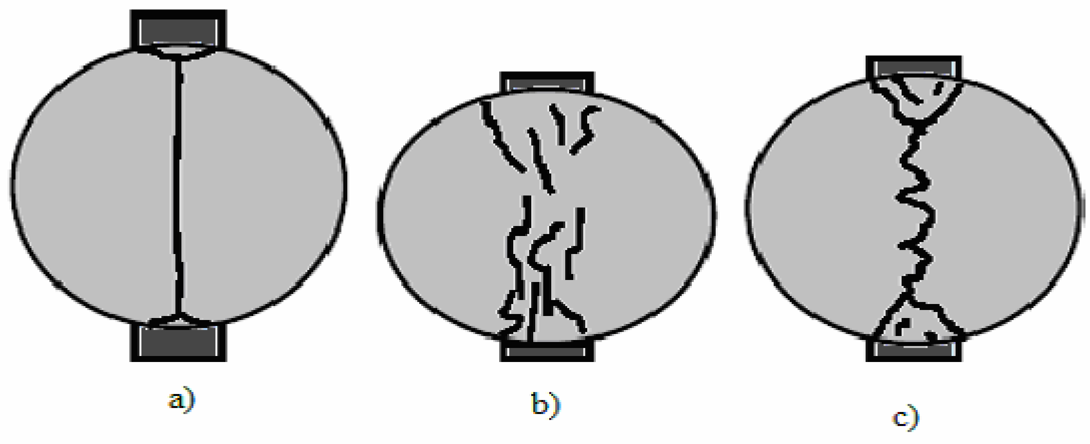


Figure (5-9): Failure type for asphalt concrete specimen from indirect tensile strength European Standard (EN 12697-23)

Indirect tensile method is used to develop tensile stresses along the diametric axis of the test specimen. The horizontal tensile stress at the centre of the test specimen is calculated to determine the indirect tensile strength by doubling the peak load (P) and then dividing it by the diameter (d) of the sample and the thickness (t) of the sample using equation.

$$\sigma = \frac{2P\,\text{max}}{\pi \times h \times d} \qquad (5.9)$$

Where:

σ	:	Indirect tensile strength (Kn/cm^2),
P_{max}	:	Maximum applied (kN),
h	:	Thickness of specimen (cm), and
d	:	Diameter of specimen (cm).

5.2.2 The Resilient Modulus Test

The resilient modulus values can be utilized to analyze the response of the pavement structure due to the application of traffic loads and, also important material property input into the M-E design procedure. Resilient modulus as the ratio of an applied stress to recoverable strain observed when a sample is exposed to cyclic loading and it is a relative measure of mixture stiffness. As well as the resilient modulus is a non-destructive test that can be used to evaluate the relative quality of materials and to generate input for pavement design or pavement evaluation and analysis. The use of the resilient modulus provides a basis for comparison of changes in material stiffness at different polymer levels and temperatures. It is reported that the resilient modulus is an important parameter to predict the pavement performance and to analysis the pavement response to traffic loading. The stiffer pavements had greater resistance to permanent deformation and it is important not to ignore that high stiffness (higher Mr.) at low temperature tend to crack earlier than more flexible mixtures (lower Mr.). Al-Abdul-Wahhab et al. (1991) conducted resilient modulus test on modified and unmodified asphalt concrete mixtures using Marshall specimen. The resilient modulus test procedures are by placing the test specimens in a controlled-temperature cabinet and bring them to the specified test temperature and it is kept in the environmental chamber for a period of minimum 12 hours. After the required test temperature is reached, the specimens were removed from the temperature chamber and place into the loading apparatus positioned and twoo temperatures were used 25 °C and 40 °C. The repeated-load indirect tension test for resilient modulus is conducted by applying a haversine waveform, with a load applied vertically in the vertical diametric plane of a cylindrical specimen. The load application and the horizontal elastic deformation were used to compute the resilient modulus value. As well as the recommended load magnitude should induce an indirect tensile stress 10 to 50% of the indirect tensile strength. A minimum of 50 to 200 load cycles is needed for the specimen precondition period. The software package, which accompanies the test machine, calculates the modulus for each load pulse. As well as the average of the three test results was reported as the resilient modulus of the specimen at that temperature.The actual load, horizontal deformation, and recovered horizontal deformation are determined for each load pulse to calculate the resilient modulus using equation (5.9).

$$M_r = \frac{P(0.27+\upsilon)}{(\Delta h)\,t} \qquad (5.10)$$

Where

P : Dynamic load

t : Specimen thickness

Δh : Recoverable horizontal deformation.

υ : Poisson’s ratio

The resilient modulus test was undertaken based on the European Standard EN 12697- 26 (Test method for hot asphalt mix-stiffness). The used temperatures are presented in table (5-4).

Table (5-4): The resilient modulus test temperature (EN 12697- 26)

Temperature, °C	Poisson’s Ratio, υ
5	0.30
25	0.35
40	0.40

5.2.3 Indirect Tensile Fatigue Test

Indirect tensile fatigue test used to evaluated and calculate fatigue life for asphalt concrete mixtures based on dissipated energy. The procedure for indirect tensile fatigue test were described in details in the European Standard EN (12697- 24). Different stress level was used to evaluate modified and unmodified asphalt concrete mixtures fatigue life. The stress level which used in these tests namely 1500, 1750, 2000 and 2250 N and the test temperature is 20 ^{o}C. Two types of controlled loading can be applied: control stress and control strain. In the control stress test, the stress remains constant but the strain increases with the number of repetitions. In the control stress test, the strain kept constant, and the load or stress is decreased with the number of repetitions. The use of constant stress has the further advantage that failure occurs more quickly and can be more easily defined. Development and accumulation of damage is evaluated in terms of dissipated energy and number of cycles. During the dynamic indirect tensile fatigue test in controlled stress sinusoidal loading force, phase angle and dissipated energy/cycle per volume will change due to the change in the mixtures behavior and damage accumulation. The dissipated energy ratio can be calculated the number of cycles to failure. The dissipated energy versus the number of cycles to failure used to calculate the fatigue life parameter K1 and K2 as described before in chapter 2. Figure (5-10) presented the sample position and deformation strips which are used to for fatigue life calculation.

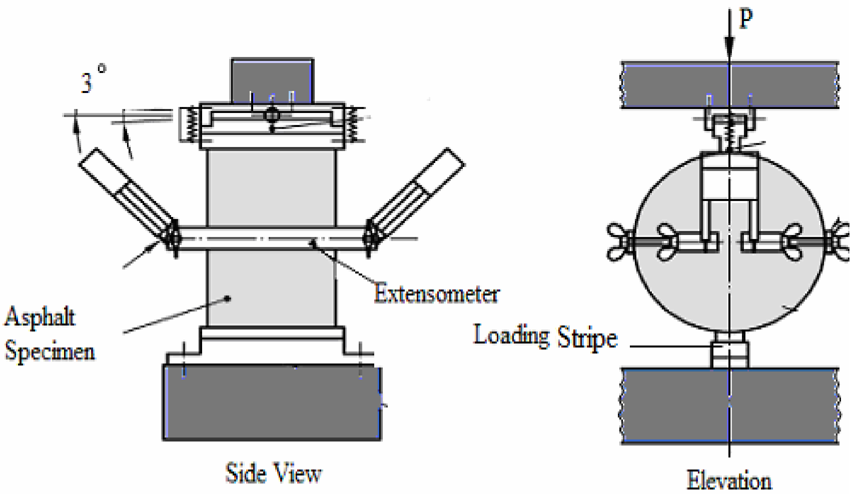


Figure (5-10) Illustration of loading and deformation strips indirect tensile fatigue test European Standard (EN 12697-24).

Chapter 6

Mechanical Properties and Fatigue Analysis for Asphalt concrete Mixture

6.0 Introduction

This chapter presents the results and analysis of modified and unmodified asphalt concrete mixtures. The results are divided into three parts, which is based on indirect tensile test to evaluate mixtures tensile strength, stiffness and fatigue resistance. The first part describes the fracture surfaces of the failed specimens as well as indirect tensile strength and the fracture energy. The second part presents the stiffness of modified and unmodified asphalt concrete mixtures. The third part presents fatigue test results as well as correlation between stiffness and number of cycles to failure for all tested samples. Asphalt concrete mixtures were prepared using different modification level according to the bitumen rheology result in the first phase.

6.1 Indirect Tensile Strength Results

In this section, the indirect tensile strength used to evaluate the tensile strengths of the mixes when it is subjected to constant strain rate. Once the indirect tensile strength test is complete a visual inspection for the fracture surfaces of the failed specimens were performed. Figure (6-1) presents the fracture surfaces of the failed indirect tensile test sample for modified and unmodified asphalt concrete mixtures. Indirect tensile tests were carried out at a temperature of 20ºC and the specimen was loaded to failure along the diametrical plane of the specimen at a deformation rate of 50.8 mm/min. A weak bond was observed between bitumen and aggregate for unmodified mixtures. The loss of bond at the interface between bitumen and aggregate can cause a significant reduction in fatigue life and an increase in rutting. The structure and functional groups of asphalt molecules influences the bonding of the asphalt to the aggregate. Binder-aggregate bond plays an important role in failure and fracture of asphalt concrete mixtures. Crack resistance of asphalt concrete mixtures is function of the bond between binder and aggregate as well as the cohesion of the asphalt binder. The fractured surfaces of indirect tensile test specimen revealed that the failure surfaces exhibited both cohesive and adhesive failures. Adhesion is the bond between asphalt and an aggregate surface, while cohesion is how well bitumen holds together.

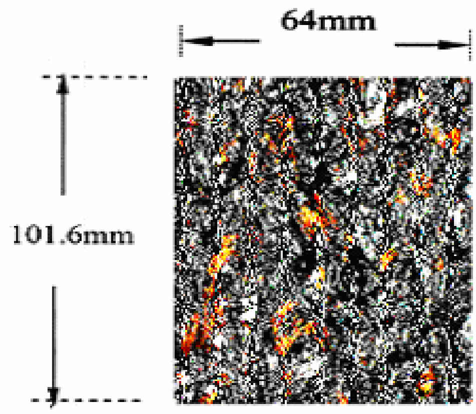
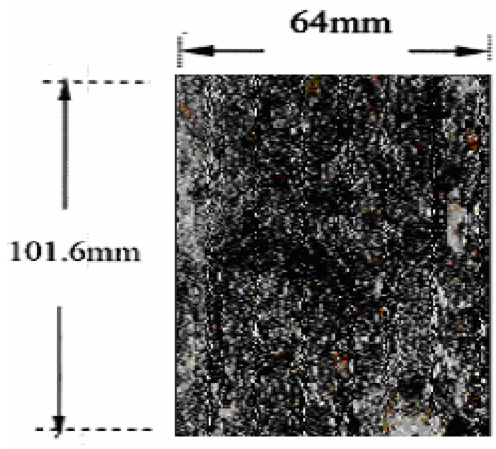
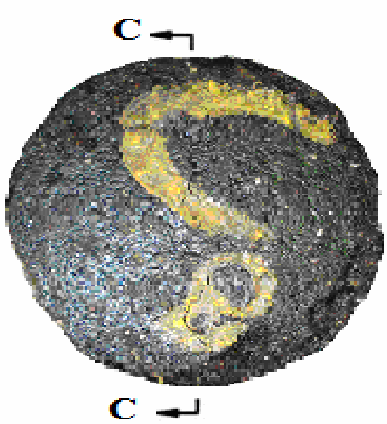
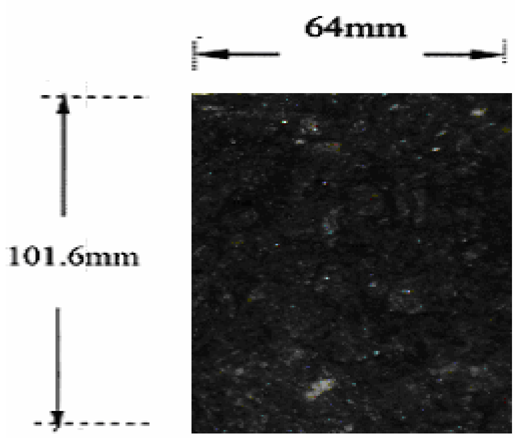
	
Unmodified Mixtures	Section A-A
	
Modified Mixtures with 5% SBS	Section B-B
	
Modified Mixtures with 5% CR	Section C-C

Figure (6-1) typical fracture surfaces of the failed ITS specimens for modified and unmodified asphalt concrete mixtures at 20 °C.

The interfacial bonds between asphalt and aggregate, as well as between asphalt and polymer directly affect the adhesive, cohesive and mechanical strength of asphalt concrete Mixtures. The adhesive and cohesive properties of asphalts determine how well roads behave under traffic and weather. Course aggregate in the fracture surfaces for unmodified failed specimen appeared with visible uncoated aggregate faces but the fracture surfaces for the modified mixtures have thick coating black asphalt cement and no exposed aggregate. The addition of polymer to the bitumen producer either thicker binder film on the aggregate or stronger bonds between binder and aggregate due to the increased viscosity. The thicker bitumen film hold the aggregate together which make the mixture more resistance to crack or more liable to heal under pavement surface condition. The thickness of the binder film affect the on aggregate contact behavior and lead to the improvement in the mechanical properties of the produced mixtures. From fracture surface for the modified mixtures the types of failures that occur have a strong relationship with the bitumen type and the thickness of binder around the stone grains. The maximum load was recorded for each specimen using the hold at break-point mode. The Indirect Tensile Strength of the specimens was calculated using equation (3-1). Load versus deformation from Indirect Tensile Strength test at 20 ^{o}C is presented in Figure (6-2). Three asphalt concrete specimens were tested and the average values from the result were used to for calculation. During the test, the magnitude of the load and the vertical deformations were measured. Figure (6-2) presents load versus deformation for modified and unmodified asphalt concrete mixtures.

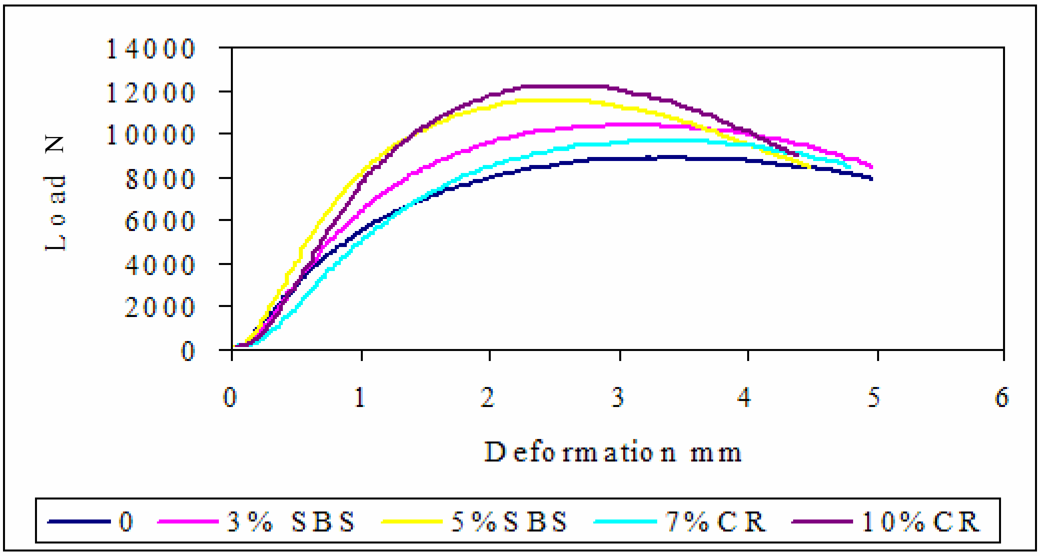


Figure (6-2) Load versus deformation from indirect tensile strength test result at 20 ^{o}C.

From table (6-1) it can be seen that modified asphalt concrete mixtures with CR or SBS have a significant effect on the tensile strength properties. The maximum loads untie failure for all different mixtures were presented and it is clear that the modified mixtures with 10% Crumb Rubber have 1.5 times higher failure load than unmodified mixtures. Load to failure increase by the increase of the polymer content or also changed by the change of polymer type. The modified asphalt concrete mixtures with 10 % crumb rubber have the highest load to failure and low deformation comparing with the rest of mixtures. Indirect tensile strength and fracture energy for all samples are given in table (6-1) at two different temperatures level namely 5 ^{o}C and 20 ^{o}C.

Table (6-2) Indirect tensile test result at 20 ^{o}C

Type of mixtures	T ^{o}C	Maximum load (KN)	ITS (GPa)	Total Fracture Energy KN.mm	Fracture energy to failure KN.mm
Control	20 ^{o}C	8.305	$0.819 *10^{-3}$	45.25	15.40
3 % SBS		10.230	$1.009* 10^{-3}$	56.65	18.98
5% SBS		11.851	$1.17*10^{-3}$	64.61	22.00
7% CR		9.825	$0.969*10^{-3}$	53.30	18.22
10% CR		12.155	$1.200*10^{-3}$	66.29	22.57

Figure (6-3) shows that the addition of styrene-butadiene-styrene (SBS) or (CR) to the tested mixtures led to a strong increase in the tensile strength values compared to unmodified asphalt concrete mixtures. Improvements in tensile strength of the mixtures quantify how much stress that the mixtures will endure before failing. Modified asphalt concrete mixtures with 10 % CR (Crumb rubber) produced the highest tensile strength followed by 5% SBS, 3% SBS and 7 % CR. The strength values range from $0.82x10^{-3}$ to $1.2x10^{-3}$ GPa for all mixtures. Using modified bitumen produce mixtures that can be stretch without crack under tension load. Fatigue is induced by tension, and thus an improvement in the tensile strength property of the mix is seen as improvement in fatigue resistance. The mixtures with high tensile strength are more likely to resist cracking than the mixtures with a low tensile strength.

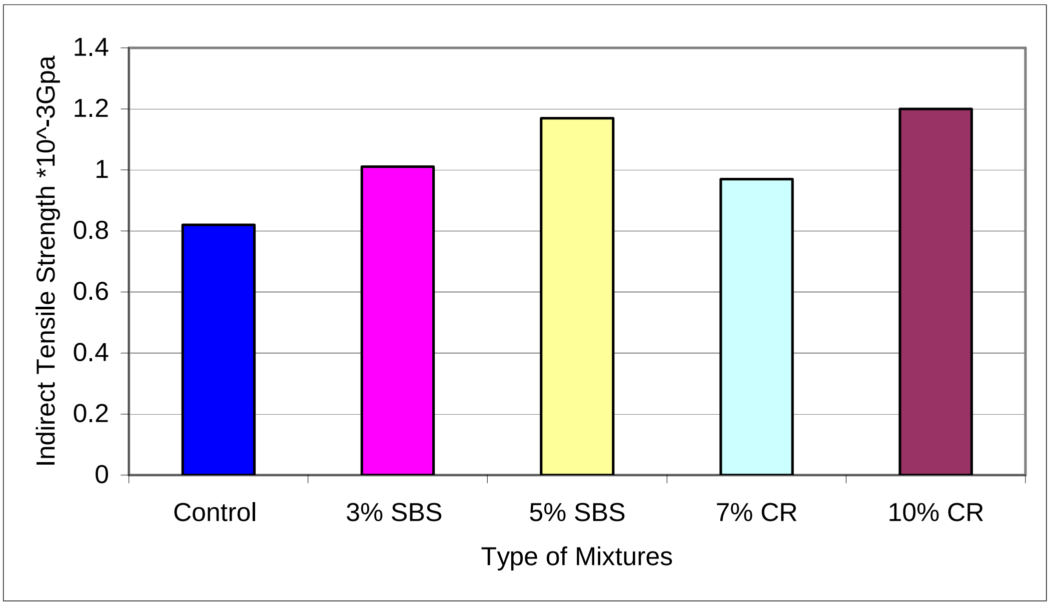


Figure (6-3):Indirect tensile strength for modified and unmodified asphalt concrete mixtures.

The indirect tensile strength values are a relative indicator to the resistance of the asphalt mixtures to fracture whether related to fatigue. Materials with high tensile strength values have high potential to absorb energy without fracture. To evaluate cracking performance for the asphalt concrete mixtures using indirect tensile test a lot of parameters should be considered namely indirect tensile strength, total fracture energy and fracture energy to failure. The fracture energy is defined as the work to be done to fracture the specimen, and is equal to the area under the load-deflection curve up to the maximum failure load. The total fracture energy is calculated as the total area under the load- deformation curve. Figure (6-4) represents the definition of total and failure energy. The fracture energy can be calculated according to the following equation:

$$F_E = \frac{\int_0^{\delta\,max} P(\delta)\,d\delta}{HD} \qquad (6\text{-}1)$$

Where:

F_E = Total Fracture until Failure.

P = Load N

δ. = Deformation mm

H = Thickness of The specimen mm

D = Diameter of the specimen mm.

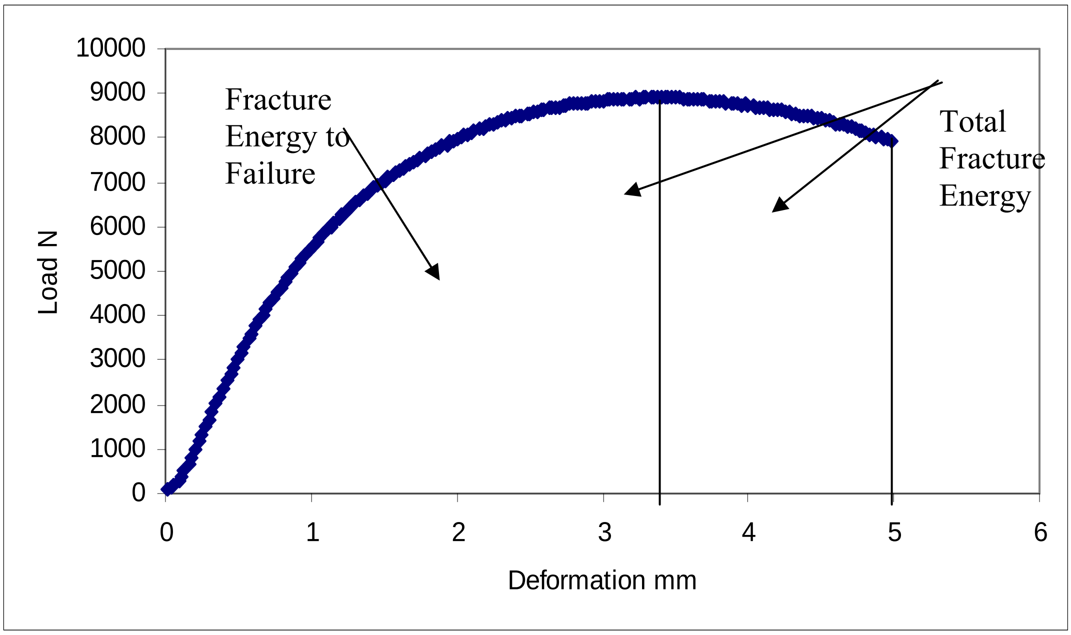


Figure (6-4) Typical example for calculation of total fracture energy and energy until failure.

The fracture energy to failure and total fracture energy are strongly enhanced by the polymer modification (Figure 6-5). This means that both SBS polymers and CR are capable to increase the threshold energy required to crack the mixture. On other hand crumb rubber seems to provide greater benefits in cracking resistance than the SBS.

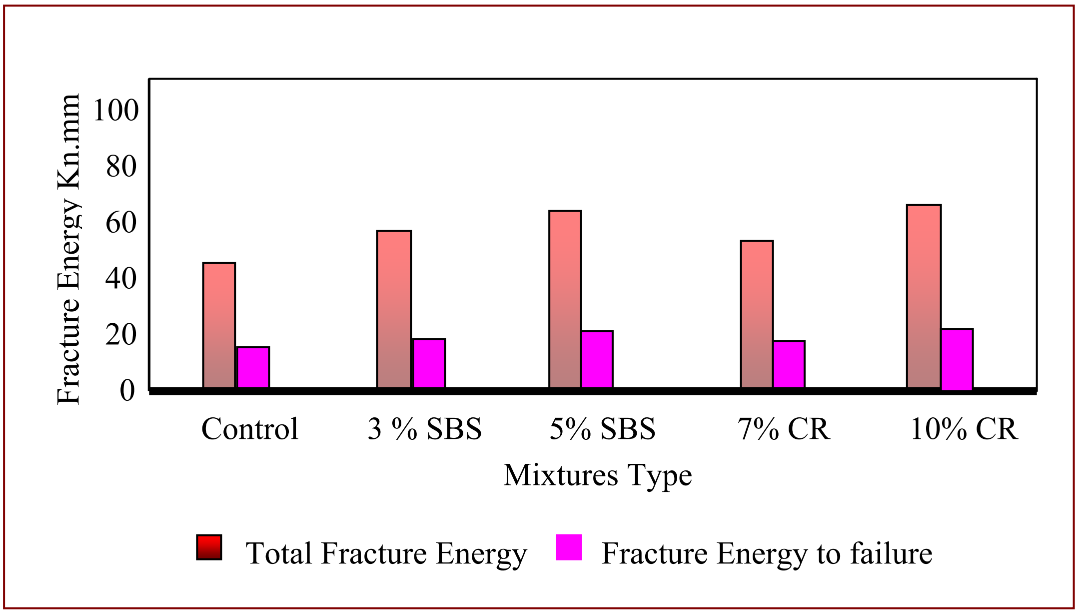


Figure (6-5) Total fracture energy and energy until failure

6.2 Resilient Modulus (Mr) Test Results

In this study, the resilient characteristics of modified and unmodified mixes were determined using the dynamic Apparatus. Three specimens were tested each specimen and height of the specimen was determined. The specimens were conditioned for 12 hours in an environmental chamber at the specified test temperature. Resilient modulus is the most important variable to mechanistic design approaches for pavement structures. It is the measure of pavement response in terms of dynamic stresses and corresponding strains. Repeated load indirect tension resilient modulus tests (DIN 12697-26: 2004(D) part C) were performed on modified and unmodified asphalt concrete mixtures at temperatures of 5°C, 20°C and 40°C, respectively). Constant test temperature was maintained using an environmental air chamber. Each specimen was placed inside the chamber at the set temperature for 3 hours before testing. About 15% of the indirect tensile strength of both mixtures was applied on the vertical diameter for control and modified specimens.

The frequency of load application used was 10 Hz, with haversine compressive load was applied, and the load and corresponding deformation data were recorded continuously. The average resilient modulus values from the three tests were reported as resilient modulus. The average value of those two test results was considered as the final resilient modulus (Mr) of that specimen. The test results are presented in Table (6-3).

Table (6-3): Resilient modulus test result at different temperature.

Type of mixtures	Resilient modulus (MPa) @5°C	Resilient modulus (MPa) @20°C	Resilient modulus (MPa) @40°C
Control	1450	1030	180
3 % SBS	2160	1540	273
5% SBS	3417	2915	451
7% CR	3590	2690	430
10% CR	3941	3060	612

The resilient modulus represents the ratio of an applied stress to the recoverable strain that takes place after the applied stress has been removed. The main purpose of the resilient modulus test result analyses was to determine if the addition of waste materials and commercial polymer brought any significant change in the stiffness properties of modified mixtures. Resilient modulus of the five different types of specimens was plotted as a function of temperature Figure (6-6). It is evident that as the temperature increases, resilient modulus values decrease. The ranges of MR for conventional and 3 % SBS, 5 % SBS 7% CR , 10 % CR modified asphalt concrete samples are 2,200 to 2,040 MPa and 2,720 MPa to 2,630 MPa at 25°C, respectively. The resilient modulus values at 25 ^{o}C and 40 ^{o}C showed on an average a reduction of 52% and 82% respectively when compared to the resilient modulus values at 5^{o}C for unmodified asphalt. The MR value is affected by the modifier type, modifier content and the asphalt temperature.

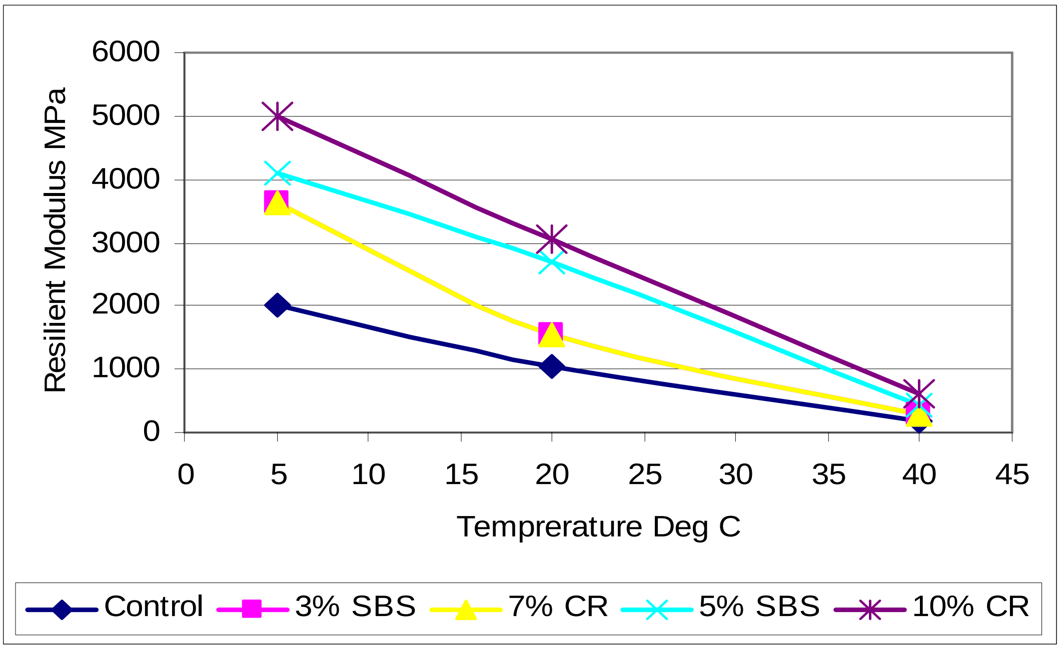


Figure (6-6) Resilient modulus for modified and unmodified asphalt concrete mixtures.

The modified asphalt concrete mixtures consistently exhibited higher resilient modulus values than conventional mixtures. The increase in modifier content produces an improvement in the elastic properties of the studied mixtures. Therefore, modified bitumen has improved the resilient modulus of asphalt mixtures compared to the control mixtures. This might be to the higher viscosity and thick bitumen films which give the mixtures rubber properties that lead to better resilience properties. These findings indicate that using modified bitumen produce asphalt concrete mixtures with greater stiffness and thus higher load bearing capacity. Crumb rubber modified binders showed lower temperature susceptibility. Mixes with modified binders displayed higher flexibility at lower temperatures because of lower resilient modulus and higher stiffness and tensile strength at higher temperatures.

6.3 Fatigue Test Results

Control stress fatigue test was done which used to determine number of cycles to failure for asphalt concrete mixtures. The data acquired during the fatigue test was loaded into a MS Excel worksheet, which used to calculate the stiffness modulus as well as dissipated energy and dissipated energy ratio at all load cycles using the specimen dimensions, load and the deformation data. Number of load cycles to the specimen failure is defined as fatigue life for asphalt concrete mixtures. Test specimen is subjected to a different level of stress namely 1500, 1750, 2000 and 2250 N. The test was done at 20 oC and the test procedure and specification are presented before in chapter 5. A typical fatigue test result for specific applied loads 2250N for modified and unmodified and the relations between deformations (mm) versus number of load cycles to failure are presented figure (6-7).

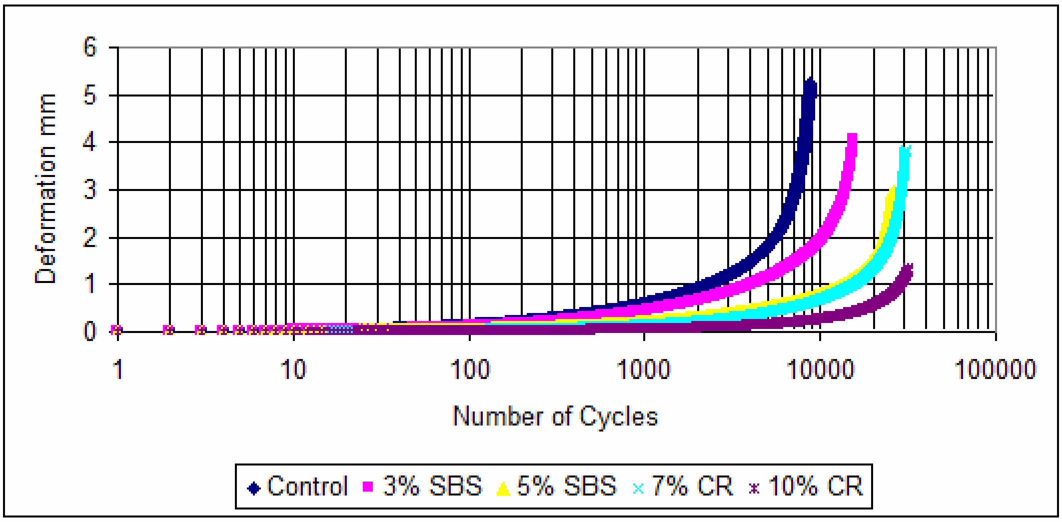


Figure (6-7): A typical fatigue test result for modified and unmodified asphalt concrete mixtures at 2250 N.

From the previous it can be seen clearly, that the use of modified bitumen, improved the mixtures properties. According to these results, by the increase of the modifier percentage the vertical deformation of modified sample decreases significantly. The decreases in deformation for modified asphalt concrete mixtures give an indication for resistance to permanent deformations. The resistance to permanent deformation depends on the type of the used modifier and the modification level. The unmodified asphalt concrete mixtures have high deformation and lower number of load cycles to failure. The CRM mixtures with 10 % have the highest number load cycles to failure and lowest deformation. There are a lot of test used to evaluate the resistance to permanent deformations but the indirect tensile fatigue test result gives only indication about deformation which help which materials is more resistance to rutting.

6.3.1 Dissipated Energy for Modified and Unmodified Asphalt Concrete Mixtures

In asphalt concrete pavement a certain amount of work is done to deform the surface layer during each cycles of traffic loading. Part of this work is recovered while the remaining work is dissipated. The dissipated work is exhibited by one or more damage mechanisms such as fatigue cracking initiation and propagation, permanent deformation and heat. Typical example for dissipated energy ratio versus number of load cycles to failure for unmodified and 10% CR modified asphalt concrete mixtures at stress level 2500 N was presented in figure (6-8) and (6-9) respectively.

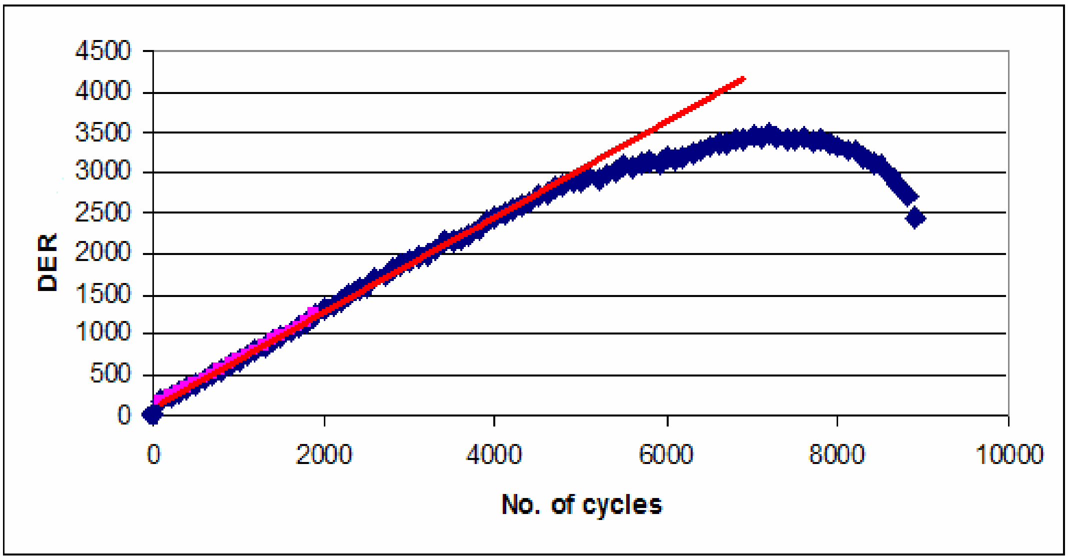


Figure (6-8) Dissipated energy ratio versus number of load cycles to failure for unmodified asphalt concrete mixtures at 2250 N.

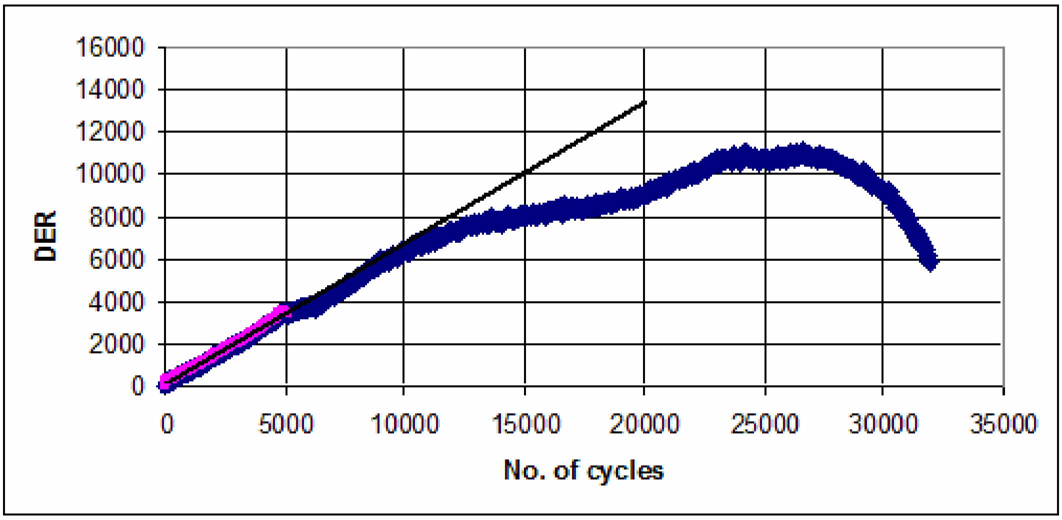


Figure (6-9) Dissipated energy ratio versus number of load cycles to failure for 10% crumb rubber modified mixtures at 2250 N.

The test data presented as a plot of dissipated energy ratio against the number of cycles to failure (N). The relationship between number of load cycles (Nf) and dissipated energy is used to construct classical power model.

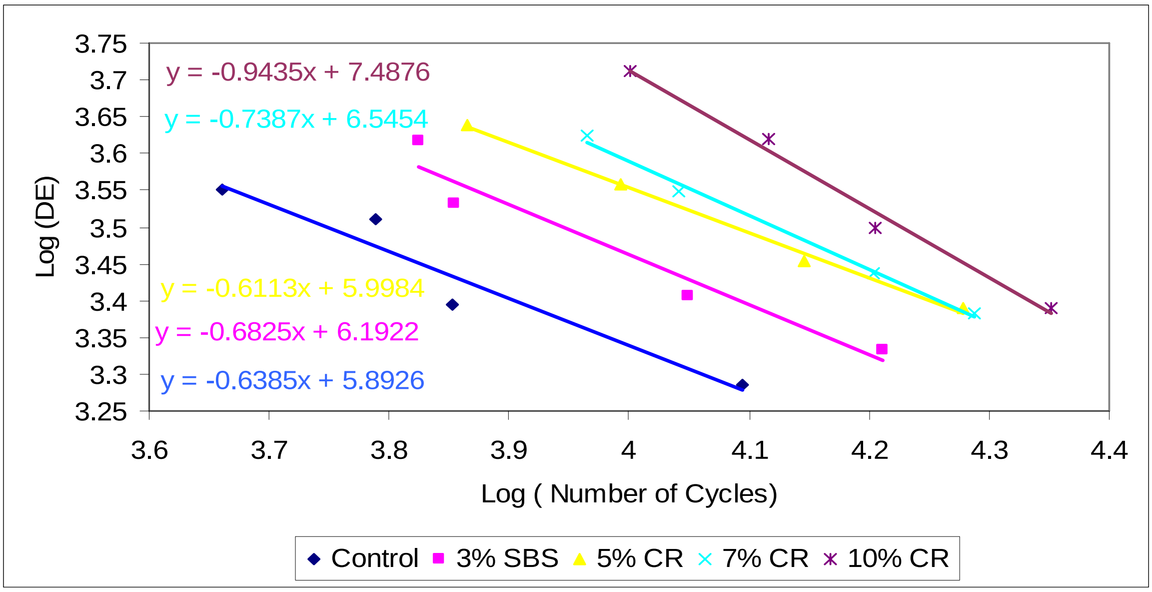


Figure (6-9): Fatigue life curves at 20°C for modified and unmodified asphalt concrete mixtures.

The number of load cycles to failure was calculated from fatigue test result at different stress level. The fatigue lives are plotted against the dissipated energy at failure using a log-log scale and a regression analysis were done on a range of values to allow the development of the fatigue relationship. Results show that fatigue behavior of modified mixes significantly improved compared to unmodified mixtures. Fatigue life increases by about 31%, 53%, 56% and 81% modified asphalt concrete mixtures with modification level namely 3%, 5% (SBS) and 7%, 10% (CR), respectively. The best result for indirect tensile fatigue test were achieved by 10% (CR) modified mixtures and 3 % addition of SBS polymer had the least effect on fatigue life. The increase in fatigue life, as observed from laboratory fatigue test results, is nearly two times. This is probably due to good adhesion between modified bitumen and aggregate that strengthens the interface and helps to prevent aggregate particles from movement. The relationship between fatigue life and dissipated energy approximately follows a simple linear function which can be adequately described by the following simple power law fit as presented in table (6-4). The most important variables from the fatigue test are the intercept and the slope of the fatigue curve, K1 and K2 respectively. It is clear that modified bitumen with crumb rubber has better fatigue life than SBS modified bitumen and unmodified mixtures.

Table (6-4) Relationship between fatigue life and applied maximum stress

Mixtures Type	Load N	Number of cycle (N_f)	Fatigue equation	
Control	1500	12421	Log Nf = -0.9435 log(Wi) + 7.4876	
	1750	7120	$N_f = 0.307 * 10^8 \left(\frac{1}{Wi}\right)^{0.9435}$	
	2000	6148		
	2250	4580	K_1=0.307*10^8	K_2=0.9435
3 % SBS PMB	1500	16272	Log Nf = -0.7387 log(Wi) + 6.5454	
	1750	11205	$N_f = 0.351 * 10^7 \left(\frac{1}{Wi}\right)^{0.7387}$	
	2000	7148		
	2250	6682	K_1=0.351*10^7	K_2=0.7387
5 % SBS PMB	1500	19005	Log Nf = -0.6113 log(Wi) + 5.9984	
	1750	14003	$N_f = 0.996 * 10^6 \left(\frac{1}{Wi}\right)^{0.6113}$	
	2000	9830		
	2250	7342	K_1=0.996*10^6	K_2=0.6113
7 % CRMB	1500	19403	Log Nf = -0.6825 log(Wi) + 6.1922	
	1750	15969	$N_f = 0.155 * 10^7 \left(\frac{1}{Wi}\right)^{0.6825}$	
	2000	11000		
	2250	9234	K_1=0.155*10^7	K_2=0.6825
10 % CRMB	1500	22470	Log Nf = -0.6385 log(Wi) + 5.8926	
	1750	16040	$N_f = 0.780 * 10^6 \left(\frac{1}{Wi}\right)^{0.6385}$	
	2000	13053		
	2250	10021	K_1=0.780*10^6	K_2=0.6385

6.3.2 Relationship between Fatigue Life and Resilient Modulus

Resilient modulus test, indirect tensile test and Fatigue were conducted and the results indicated that the use of both crumb rubber and SBS improved the resilient modulus and fatigue property compared to the unmodified mixtures. Correlation between fatigue life for

SBS modified mixtures and CR modified mixtures as compared to unmodified mixtures are presented in figure (6-10) and (6-11) respectively.

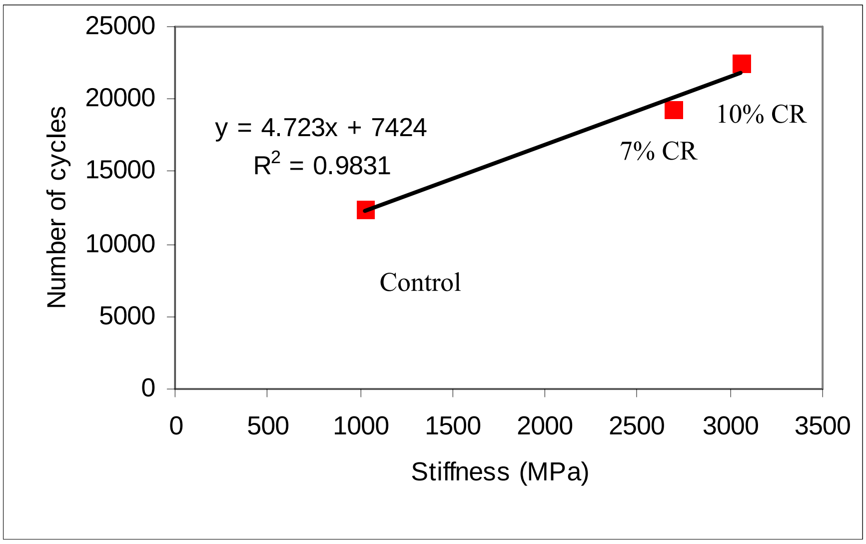


Figure (6-10): Fatigue life versus resilient modulus (stiffness) for control and crumb rubber modified asphalt concrete mixtures.

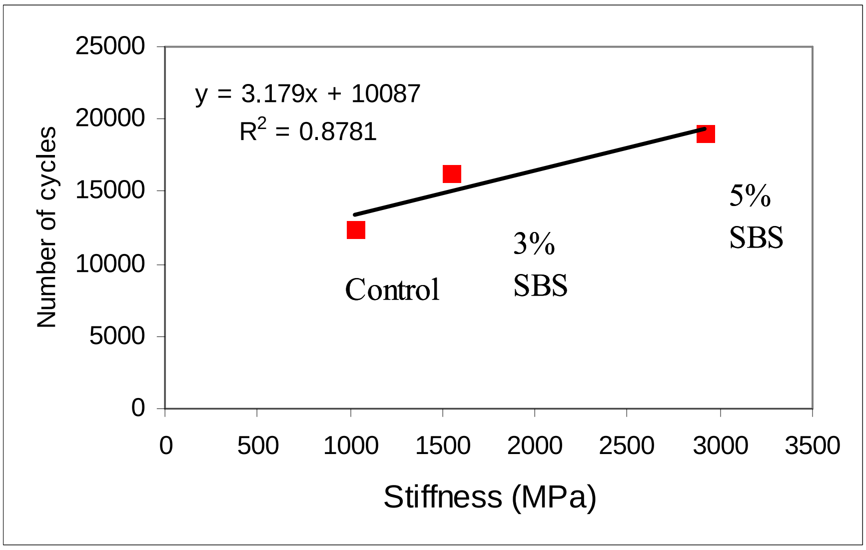


Figure (6-11): Fatigue Life versus resilient modulus (Stiffness) for control and SBS modified asphalt concrete mixtures

The fatigue life for CRM mixtures has good correlation with stiffness than the SBS PM mixtures. The correlation factor between the fatigue lives and stiffness for CRM mixtures and SBSPM mixtures are $R^2 = 0.9831$ and $R^2 = 0.8781$, respectively. The increase in stiffness properties of the specimens is followed by an increase in fatigue life. The relationship between fatigue life and stiffness for CRM mixtures and SBSPM mixtures follows a simple linear function as shown in equation (6-1) and (6-2), respectively.

Fatigue life = 4.723 (stiffness) + 7424 (6-1)

Fatigue life = 3.179 (stiffness) + 10087 (6-2)

Nearly 2.2×10^4 cycles load to failure make the stiffness of the mixtures appear to have good fatigue resistance.

6.3.3 Relationship between Fatigue Life and Deformation

From fatigue test result correlation between deformation at failure and fatigue life for SBS modified mixtures and CR modified mixtures as compared to unmodified mixtures are presented in figure (6-12) and (6-13) respectively. The fatigue life of the specimens showed a proportional correlation with the stiffness properties of the mixtures, while there was an inverse correlation between the fatigue life and deformations.

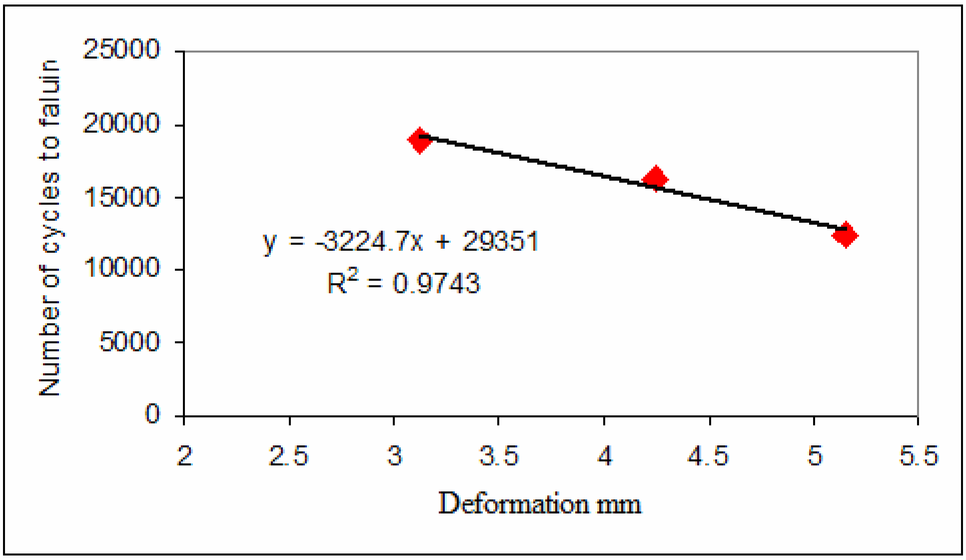


Figure (6-12) Fatigue life versus deformation (mm) for SBS modified mixtures

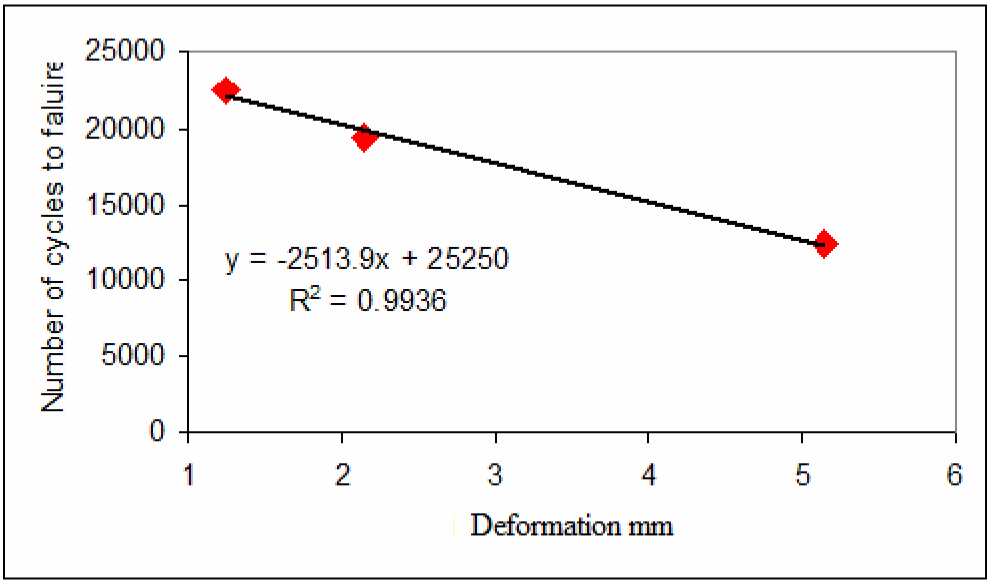


Figure (6-13) Fatigue Life versus deformation (mm) for Crumb rubber modified mixtures.

The correlation factor between the fatigue life and deformation for SBS modified mixtures was R^2 = 0.97, while a slightly higher correlation factor is obtained between the fatigue life and deformation for crumb rubber modified asphalt concrete with R^2 = 0.99. The modified bitumen with 10% crumb rubber has the higher stiffness and lower deformation as compare to unmodified asphalt concrete mixtures in additional to higher fatigue life. The relationship between fatigue life and deformation for CRM mixtures and SBSPM mixtures follows a simple linear function as shown in equation (6-1) and (6-2), respectively.

$$\text{Fatigue life} = -3224\,(\text{deformation}) + 29351 \qquad (6\text{-}3)$$

$$\text{Fatigue life} = -2513\,(\text{deformation}) + 25250 \qquad (6\text{-}4)$$

In conclusion, the increase in stiffness properties of the specimens is followed by an increase in fatigue life of the specimens Figure(6-10, 6-11) and an increase in deformation properties of the specimens is followed by a decrease in fatigue life of the specimens Figure(6-12, 6-13). Fatigue life of crumb rubber modified mixtures was found to be significantly improved compared to unmodified mixtures. The increase in fatigue life, as observed from laboratory fatigue test results, is nearly two times more than unmodified mixtures.

6.4 Dissipated Energy and Number of Cycles for Asphalt Binder and Mixture

The amount of energy dissipated per volume from the bitumen specimen in dynamic shear rheometer fatigue test can be correlated with the amount of energy dissipated per volume from asphalt concrete specimen in indirect tensile fatigue test. The amount of damage per unit volume of a material is logically a better parameter to compare different geometries. The energy relationship obtained from bitumen and mixtures can be correlated with dissipated energy that occurs in a pavement structure under traffic load. To normalize the results for volume the dissipated energy values were multiplied by the sample volume in mm^3. The dissipated energy per volume was calculated for dynamic shear rheometer specimen and for indirect tensile fatigue specimen. The relation between dissipated energy for modified and unmodified bitumen and mixtures are presented in figure (6-14).

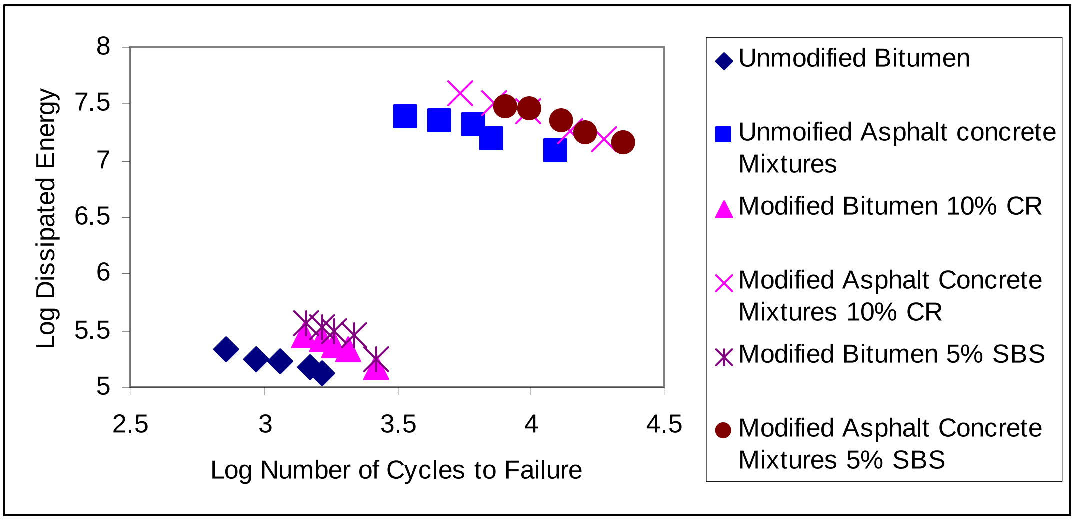


Figure (6-14): Relationship between dissipated energy and number of cycles to failure for asphalt binder and mixtures.

From figure (6-14) there are shift factor between the dissipated energy for bitumen and asphalt concrete mixtures. The difference in dissipated energy for the same volume of bitumen may be due to aggregate and sand in asphalt concrete mixtures or stress strain distribution in bitumen and asphalt concrete mixtures specimens. The shift factor can be used to predict fatigue life for asphalt mixture from the fatigue life for asphalt binder.

Chapter 7
Conclusion and Recommendations

7.0 Conclusion

The main objectives of this research were to characterize the rheoloical and mechanical properties of modified asphalt binder and mixtures. The additions of modifier to the pure bitumen improve the viscoelastic behavior of the bitumen and change its rheologicl properties. Two type of modifiers were used, which include crumb rubber (CR) and styrene butadiene styrene (SBS) have different amount of influence, decreasing or increasing, in the rheological properties of asphalt binder from the same source. After conducting laboratory tests on asphalt binder and mixtures with different polymer content and after analyzing the data and comparing the results, the following conclusions have been drawn:

- Polymer-modified bitumen typically is more viscous (thicker) than unmodified binders, and tend to show improved adhesive bonding to aggregate particles.
- Thicker binder coatings usually take longer to become brittle, so the durability of the pavement can be improved.
- At higher SBS polymer content (7%) and (10%), the behaviour of the modified binders remains close to that of the modified binder with 5% SBS.
- At lower rubber content (3%) and (5%), the behaviour of the modified binders remains close to that of the base bitumen.
- Modified bitumen exhibited better rutting and fatigue resistance as compared to the unmodified binder mixes.
- The result showed significant improvement in fatigue behavior of all modifier types used when compared with the control mixtures.
- Among the two types of mixtures prepared by 3% and 5% of SBS polymer, the mixtures with 5% SBS had the most improved mechanical behavior and fatigue resistance.
- Among the two types of mixtures prepared by 7% and 10% of crumb rubber, the mixtures with 10% crumb rubber had the most improved mechanical behavior and fatigue resistance.

- Excellent fit were found between dynamic shear rheometer experimental results and finite element model.
- Using both commercial polymer and crumb rubber improve the mechanical properties of asphalt concrete mixture nearly by the same amount of increment.

7.1 Impact of Using Asphalt Additives:

Using asphalt additives for highway construction project depends on many factors such as cost, construction ability, availability, and expected performance. Asphalt additives have been used to improve asphalt pavement performance as well as to reduce asphalt pavement distresses such as moisture damage, permanent deformation, and thermal fatigue cracking. The performance of modified asphalt concrete pavement is expected to be more stable at warmer temperatures and more flexible at colder temperatures. Polymer becomes a permanent part of the highway construction and the degree of modification depends on the polymer property, polymer content and nature of the bitumen.

7.1.1 Advantages of Using Asphalt Additives

1. Asphalt additives improve durability, crack resistance, asphalt binder elasticity as well as increase the pavement life.
2. Adding rubber to asphalt increases its flexibility and reduces cracking and it gives some benefits for both the economy and the environment more than commercial polymer.
3. Blending bitumen with additives expands the useful temperature range of the modified bitumen and increases the temperature susceptibility.
4. Using both SBS and CR improve the rheological properties and fatigue resistance for modified asphalt binder and mixtures.

7.1.2 Disadvantages of Using Asphalt Additives

1. Using commercial polymer is significantly more expensive than conventional asphalt because of the higher temperature needed, additional equipment and other special handling requirements.

2. The disadvantage of rubber modified asphalt is associated with difficulties the preparation process during heated storage.
3. Crumb rubber modified asphalt separate into two or more phases, because the weak interaction between the rubber particle surface and the asphalt.
4. Modified asphalt concrete mixtures are hard on equipment and it has some difficulties during the manufacturing, transporting, placement and compaction processes.

7.2 Recommendations

The following recommendation briefly describes the area in which further research work valuable.

- A life cycle cost analysis of pavement constructed using various polymer-modified asphalt binder in comparison to those constructed using conventional binder needs to be performed.
- Use scanning electron microscope images to evaluate binder-aggregate adhesion.
- Evaluate compatibility, chemical reaction, storage stability and effectiveness of local available Egyptian modifier to modify the local Egyptian bitumen.
- Conduct more studies for fatigue damage, including more mix variables and different rubber size to evaluate the effect of the particle size and texture of rubber.
- Blend several different polymer types with same base asphalt, for example by blending base bitumen with thermoplastic polymer, thermoelastic polymer and high boiling point petroleum oil in order to improve rutting and fatigue resistances as well as low temperature cracking.
- Trail section has to be constructed in order to validate the results obtained in this research.
- New part should be added to Egyptian code for specification under title using of polymer technology in highway construction.
- Calculate the dissipated energy that occurs in a pavement structure under traffic load using Finite Element Method (FEM) and correlate it with dissipated energy calculated from asphalt concrete mixtures specimen in order to predict fatigue life for asphalt pavement.

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Appendix (A)

Volumetric Characteristics of Asphalt Concrete Mixtures.

This European Standard describes a procedure for calculating two volumetric characteristics of a compacted bituminous specimen: the air voids content (Vm) and the voids content in the mineral aggregate filed with binder (VFB). The method is suitable for specimens which are laboratory compacted or specimens from cores cut from the pavement after placement and compacting. These volumetric characteristics can be used as mix design criteria or as parameters for evaluating the mixture after placing and compacting in the road. For air voids calculation according to the European Standard the following terms and definitions can be used:

- Air void:
  Pocket of air between the bitumen-coated aggregate particles in a compacted bituminous specimen
- Air voids content (Vm):
  Volume of the air voids in a bituminous specimen, expressed as a percentage of the total volume of that specimen
- Maximum density:
  Mass per unit volume without air voids of a bituminous material at known test temperature
- Bulk density:
  Mass per unit volume, including the air voids, of a specimen at known test temperature

1- Determination of the air voids content (Vm):

The air voids content of the bitumen specimen is calculated using the maximum density of the mixture and the bulk density of the specimen. The air voids should be calculated to the nearest 0.1% (V/V) as follows:

$$V_m = \frac{\rho_m - \rho_b}{\rho_m} \times 100\ \% \qquad \text{(A-1)}$$

Where

V_m = Air void content of the mixture, in the nearest 0.1% (V/V)

ρ_m = Maximum density of the mixture, in kilogram per cubic meter (kg/m^3),

ρ_b = Bulk density of the specimen, in kilogram per cubic meter (kg/m^3)

2- Determination of the percentage of voids in mineral aggregate filled with bitumen (VFB). The percentage of voids in the mineral aggregate for asphalt concrete specimen filled with bitumen can be calculated using bitumen content, voids in mineral aggregate, bulk density of specimen and density of bitumen.

$$VFB = ((B \times \rho_b / \rho_B) / VMA) \times 100\% \qquad \text{(A-2)}$$

Where

VFB = the percentage of the voids in the mineral aggregate filled with bitumen,

B = Percentage of the bitumen in the specimen,

ρ_b = Bulk density of the specimen, in kilogram per cubic meter (kg/m^3),

ρ_B = density of the bitumen, in kilogram per cubic meter (kg/m^3),

VMA = Void content in the mineral aggregate.

3- Void in the mineral aggregate (VMA):

Volume of inter-granular void space between the aggregate particles of a compacted bituminous mixture that includes the air voids and the volume of the bituminous binder in the specimen, expressed as a percentage of the total volume of the specimen

$$VMA = Vm + \frac{B \times \rho_b}{\rho_B} \ \% \qquad \text{(A-3)}$$

Where

VMA = Voids content in mineral aggaragte,

Vm = Air voids content of the specimen

B = Percentage of the bitumen in the specimen,

ρ_b = Bulk density of the specimen, in kilogram per cubic meter (kg/m^3),

ρ_B = density of the bitumen, in kilogram per cubic meter (kg/m^3),

Appendix (B)

Amplitude Stress Sweep Test

- Stress τ , in Pa = 100 - 15000 pa,
- Temperatures (T) = 35 ^{o}C
- Frequency (f) = 1 Hz,
- Materials: crumb rubber 0 %, 3%, 5%, 7%, and 10%

Table (B-1): Amplitude Stress Sweep Test result for Conventional Bitumen

Amplitude Stress Sweep Test Conventional Bitumen						
time in s	τ, in Pa	|G*| in Pa	G' in Pa	G' in Pa	δ in °	γ in -
1,255	100	41000	14000	13700	74,78	0,002435
7,740	1000	42000	14000	13890	74,69	0,02384
14,57	2000	41000	14000	13610	74,77	0,04841
21,05	3000	40000	13000	13170	74,78	0,07445
27,82	4000	39000	13000	12630	74,94	0,103
34,28	5000	37000	12000	12020	75,04	0,134
41,21	6000	36000	11000	11440	75,14	0,168
48,20	7000	34000	11000	10820	75,25	0,207
55,19	8000	32000	10000	10170	75,32	0,252
62,19	9000	30000	9500	9528	75,38	0,303
69,16	10000	27000	8800	8840	75,51	0,365
76,70	11000	25000	8100	8118	75,63	0,440
84,15	12000	24000	7700	7651	75,72	0,509
91,02	13000	22000	7100	7055	75,74	0,587
97,83	14000	21000	6600	6569	75,89	0,660
104,3	15000	20000	6000	5952	75,99	0,759

Table (B-2): Amplitude Stress Sweep Test result for 3% Crumb Rubber modified bitumen

Amplitude Stress Sweep Test Crumb Rubber modified bitumen 3%						
time in s	τ, in Pa	|G*| in Pa	G' in Pa	G' in Pa	δ in °	γ in -
1,306	100	78000	21000	20570	73,69	0,001276
7,930	1000	78000	21000	20550	73,62	0,01285
14,49	2000	78000	21000	20550	73,66	0,02557
21,78	3000	78000	20000	20350	73,69	0,03870
29,74	4000	76000	20000	19820	73,85	0,05246
37,68	5000	75000	19000	19360	74,01	0,06668
45,58	6000	74000	19000	18900	74,16	0,08141
53,59	7000	72000	18000	18430	74,31	0,09668
60,95	8000	71000	18000	18100	74,44	0,112
67,86	9000	71000	18000	17810	74,49	0,128
74,30	10000	70000	17000	17430	74,63	0,144
81,77	11000	69000	17000	17020	74,74	0,160
89,19	12000	68000	17000	16710	74,76	0,177
95,93	13000	67000	16000	16480	74,89	0,194
102,5	14000	66000	16000	16070	74,93	0,212
109,8	15000	65000	16000	15620	75,04	0,233

Table (B-3): Amplitude Stress Sweep Test result for 5% Crumb Rubber modified bitumen

Amplitude Stress Sweep Test Crumb Rubber modified bitumen 5%						
time in s	τ, in Pa	|G*| in Pa	G' in Pa	G' in Pa	δ in °	γ in -
1,388	100	61000	17000	17190	73,62	0,001640
7,985	1000	66000	19000	18550	73,61	0,01521
14,53	2000	65000	18000	18340	73,66	0,03068
21,84	3000	64000	18000	17750	73,81	0,04712
29,13	4000	62000	17000	17090	73,98	0,06460
35,66	5000	60000	16000	16370	74,20	0,08314
42,55	6000	58000	16000	15700	74,39	0,103
49,96	7000	57000	15000	15030	74,58	0,124
57,32	8000	55000	15000	14570	74,69	0,145
63,80	9000	54000	14000	14060	74,89	0,167
70,67	10000	53000	14000	13620	75,01	0,190
77,57	11000	51000	13000	13160	75,17	0,214
85,10	12000	50000	13000	12580	75,40	0,241
92,91	13000	48000	12000	12090	75,55	0,268
100,7	14000	47000	12000	11690	75,67	0,296
107,2	15000	46000	11000	11280	75,83	0,325

Table (B-4): Amplitude Stress Sweep Test result for 7% Crumb Rubber modified bitumen

Amplitude Stress Sweep Test Crumb Rubber modified bitumen 7%						
time in s	τ, in Pa	|G*| in Pa	G' in Pa	G' in Pa	δ in °	γ in -
1,553	100	110000	34000	34050	71,40	0,0009366
8,911	1000	100000	33000	32580	71,72	0,009625
15,48	2000	100000	31000	31340	71,94	0,01979
22,28	3000	97000	30000	29810	72,16	0,03084
28,86	4000	93000	28000	28160	72,43	0,04288
35,64	5000	89000	27000	26560	72,67	0,05607
43,30	6000	85000	25000	24720	73,00	0,07096
50,72	7000	81000	23000	23490	73,21	0,08612
57,20	8000	78000	22000	22300	73,44	0,102
64,00	9000	75000	21000	21260	73,62	0,119
70,57	10000	73000	20000	20250	73,84	0,137
77,37	11000	70000	19000	19310	74,02	0,157
84,09	12000	68000	18000	18480	74,17	0,177
90,67	13000	65000	18000	17680	74,31	0,199
97,45	14000	63000	17000	16840	74,50	0,222
104,3	15000	61000	16000	16030	74,65	0,248

Table (B-5): Amplitude Stress Sweep Test result for 10% Crumb Rubber modified bitumen.

Amplitude Stress Sweep Test Crumb Rubber modified bitumen 10%						
time in s	τ, in Pa	|G*| in Pa	G' in Pa	G' in Pa	δ in °	γ in -
1,648	100	120000	33000	33160	70,52	0,0008467
9,468	1000	110000	32000	31790	70,66	0,008869
17,32	2000	110000	32000	31600	70,76	0,01780
24,66	3000	110000	31000	31110	70,93	0,02708
31,23	4000	110000	30000	30180	71,05	0,03686
38,54	5000	110000	29000	29000	71,21	0,04750
46,43	6000	100000	28000	27890	71,26	0,05870
54,30	7000	100000	27000	26930	71,31	0,07029
62,28	8000	97000	26000	26150	71,30	0,08208
69,60	9000	96000	26000	25650	71,27	0,09386
76,11	10000	95000	25000	25050	71,20	0,106
83,44	11000	93000	24000	24490	71,08	0,118
90,81	12000	92000	24000	24180	71,06	0,130
97,31	13000	91000	24000	23740	71,42	0,143
104,1	14000	90000	23000	23450	71,95	0,155
110,6	15000	89000	23000	23060	72,47	0,168

Appendix (c)
Frequency Sweep Test at Constant Stress

- Stress (τ) = 1000 (Pa)
- Temperatures (T) = 20 ^{o}C
- Frequencies (F) = 0.1 to 50 (Hz)
- Bitumen thickness (t) =1 (mm)
- Spindle diameter (D) = 25 (mm)
- Materials: crumb rubber 0 %, 3%, 5%, 7%, and 10%

Table (C-1): Frequency Sweep Test result at Constant Stress for Conventional Bitumen.

Frequency Sweep Test at Constant Stress Conventional Bitumen								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,369	1010	6700000	6580000	1463000	0,0001494	12,38	50,00
1|2	5,420	1000	6300000	6124000	1522000	0,0001592	13,40	40,00
1|3	9,181	1000	5900000	5643000	1603000	0,0001709	15,88	30,00
1|4	13,06	1000	5300000	5038000	1719000	0,0001881	18,33	20,00
1|5	17,66	1000	4100000	3733000	1802000	0,0002414	25,29	10,00
1|6	22,61	1000	4000000	3539000	1787000	0,0002523	26,26	9,000
1|7	26,97	1000	3700000	3286000	1793000	0,0002672	28,05	8,000
1|8	32,36	1000	3500000	3030000	1771000	0,0002849	29,80	7,000
1|9	38,11	1000	3200000	2737000	1734000	0,0003087	31,87	6,000
1|10	44,74	1000	2900000	2402000	1675000	0,0003415	34,21	5,000
1|11	52,14	1000	2600000	2034000	1576000	0,0003887	37,26	4,000
1|12	59,16	1000	2200000	1614000	1434000	0,0004632	40,97	3,000
1|13	66,04	1000	1700000	1146000	1209000	0,0006002	45,97	2,000
1|14	72,46	1000	1000000	611300	840900	0,0009619	53,48	1,000

Table (C-2): Frequency Sweep Test at Constant Stress for 3% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 3% Crumb Rubber								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,332	1010	6500000	6331000	1390000	0,0001554	12,53	50,00
1|2	5,519	1000	6500000	6364000	1516000	0,0001535	13,96	40,00
1|3	9,364	1000	6100000	5824000	1657000	0,0001656	15,86	30,00
1|4	13,55	1000	5500000	5183000	1717000	0,0001834	18,84	20,00
1|5	18,01	1000	4300000	3872000	1830000	0,0002336	25,77	10,00
1|6	22,91	1000	4100000	3662000	1807000	0,0002450	26,79	9,000
1|7	27,50	1000	3800000	3395000	1809000	0,0002600	28,62	8,000
1|8	31,75	1000	3600000	3123000	1788000	0,0002779	30,31	7,000
1|9	37,68	1000	3300000	2811000	1748000	0,0003021	32,36	6,000
1|10	44,02	1000	3000000	2489000	1692000	0,0003323	34,88	5,000
1|11	49,81	1000	2600000	2102000	1599000	0,0003787	37,77	4,000
1|12	56,88	1000	2200000	1682000	1460000	0,0004490	41,61	3,000
1|13	63,76	1000	1700000	1195000	1237000	0,0005814	46,53	2,000
1|14	70,28	1000	1100000	640500	865100	0,0009290	53,98	1,000

Table (C-3): Frequency Sweep Test at Constant Stress for 5% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 5% Crumb Rubber								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,347	1010	6900000	6756000	1485000	0,0001456	12,40	50,00
1|2	5,294	1000	6800000	6612000	1435000	0,0001485	12,25	40,00
1|3	9,286	1000	6200000	6023000	1500000	0,0001615	13,98	30,00
1|4	13,40	1000	5800000	5526000	1629000	0,0001738	16,43	20,00
1|5	17,95	1000	4600000	4240000	1793000	0,0002173	22,92	10,00
1|6	22,91	1000	4400000	4051000	1793000	0,0002258	23,87	9,000
1|7	27,32	1000	4200000	3773000	1797000	0,0002393	25,47	8,000
1|8	32,81	1000	3900000	3489000	1786000	0,0002552	27,10	7,000
1|9	39,93	1000	3600000	3161000	1765000	0,0002762	29,17	6,000
1|10	47,77	1000	3300000	2818000	1727000	0,0003026	31,51	5,000
1|11	55,27	1000	2900000	2413000	1649000	0,0003423	34,35	4,000
1|12	62,20	1000	2500000	1945000	1529000	0,0004042	38,18	3,000
1|13	69,21	1000	1900000	1404000	1317000	0,0005195	43,17	2,000
1|14	75,74	1000	1200000	754000	931500	0,0008344	51,01	1,000

.

Table (C-4): Frequency Sweep Test at Constant Stress for 7% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 7% Crumb Rubber								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,327	1010	7700000	7508000	1729000	0,0001306	12,97	50,00
1|2	5,426	1000	7700000	7544000	1676000	0,0001299	12,53	40,00
1|3	9,331	1000	7300000	7024000	1822000	0,0001381	14,55	30,00
1|4	13,41	1000	6600000	6289000	1877000	0,0001525	16,62	20,00
1|5	17,96	1000	5200000	4779000	2014000	0,0001929	22,85	10,00
1|6	22,95	1000	5000000	4588000	1978000	0,0002002	23,32	9,000
1|7	27,31	1000	4700000	4240000	2008000	0,0002132	25,34	8,000
1|8	31,39	1000	4400000	3926000	2001000	0,0002270	27,00	7,000
1|9	37,14	1000	4100000	3572000	1972000	0,0002451	28,90	6,000
1|10	45,20	1000	3700000	3177000	1927000	0,0002691	31,23	5,000
1|11	52,48	1000	3300000	2710000	1832000	0,0003057	34,06	4,000
1|12	59,48	1000	2800000	2180000	1685000	0,0003630	37,70	3,000
1|13	66,32	1000	2100000	1570000	1447000	0,0004684	42,68	2,000
1|14	72,71	1000	1300000	843800	1007000	0,0007611	50,04	1,000

Table (C-5): Frequency Sweep Test at Constant Stress for 10% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 10% Crumb Rubber								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,359	1010	8000000	7907000	1486000	0,0001251	10,65	50,00
1|2	5,387	1000	7900000	7761000	1561000	0,0001268	11,37	40,00
1|3	9,449	1000	7500000	7282000	1660000	0,0001342	12,84	30,00
1|4	13,52	1000	6900000	6619000	1769000	0,0001461	14,97	20,00
1|5	19,61	1000	5500000	5188000	1923000	0,0001808	20,34	10,00
1|6	26,25	1000	5400000	4992000	1935000	0,0001868	21,19	9,000
1|7	30,74	1000	5100000	4698000	1949000	0,0001966	22,53	8,000
1|8	34,99	1000	4800000	4389000	1954000	0,0002082	24,00	7,000
1|9	40,90	1000	4500000	4041000	1964000	0,0002226	25,92	6,000
1|10	48,91	1000	4100000	3625000	1931000	0,0002435	28,05	5,000
1|11	56,54	1000	3700000	3163000	1884000	0,0002716	30,77	4,000
1|12	63,69	1000	3100000	2593000	1763000	0,0003189	34,21	3,000
1|13	70,80	1000	2500000	1922000	1565000	0,0004034	39,16	2,000
1|14	77,34	1000	1600000	1078000	1154000	0,0006333	46,94	1,000

Appendix (D)

Frequency Sweep Test at Constant Stress

- Stress (τ) = 1000 (Pa)
- Temperatures (T) = 27 ^{o}C,
- Frequencies (F) = 0.1 to 50 (Hz)
- Bitumen thickness (t) =1 (mm)
- Spindle diameter (D) = 25 (mm)
- Materials: crumb rubber 0 %, 3%, 5%, 7%, and 10%

Table (D-1): Frequency Sweep Test result at Constant Stress for Conventional Bitumen

Frequency Sweep Test at Constant Stress Conventional Bitumen								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,341	1010	4000000	3600000	1815000	0,0002523	26,98	50,00
1|2	5,476	1010	3700000	3200000	1731000	0,0002745	28,18	40,00
1|3	9,441	1000	3200000	2700000	1675000	0,0003130	31,50	30,00
1|4	13,47	1000	2700000	2100000	1581000	0,0003766	36,46	20,00
1|5	18,03	1000	1800000	1300000	1264000	0,0005618	45,24	10,00
1|6	22,96	1000	1700000	1100000	1210000	0,0005997	46,48	9,000
1|7	27,35	1000	1500000	1000000	1148000	0,0006489	48,11	8,000
1|8	32,79	1000	1400000	900000	1070000	0,0007139	49,81	7,000
1|9	39,97	1000	1200000	770000	979300	0,0008015	51,69	6,000
1|10	46,24	1000	1100000	650000	881200	0,0009145	53,68	5,000
1|11	52,23	1000	930000	520000	767000	0,001081	56,00	4,000
1|12	59,04	1000	750000	390000	636800	0,001341	58,65	3,000
1|13	65,97	1000	550000	260000	484400	0,001819	61,79	2,000
1|14	72,83	1000	310000	130000	282500	0,003237	66,13	1,000

Table (D-2): Frequency Sweep Test at Constant Stress for 3% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 3% Crumb Rubber Modified Bitumen .								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,605	1010	4500000	4100000	1900000	0,0002220	24,73	50,00
1|2	6,029	1010	4200000	3800000	1878000	0,0002370	26,28	40,00
1|3	10,30	1000	3700000	3200000	1822000	0,0002703	29,40	30,00
1|4	14,55	1000	3100000	2600000	1747000	0,0003231	34,29	20,00
1|5	19,61	1000	2100000	1500000	1449000	0,0004740	43,35	10,00
1|6	24,83	1000	2000000	1400000	1389000	0,0005062	44,65	9,000
1|7	29,43	1000	1800000	1300000	1317000	0,0005471	46,09	8,000
1|8	33,96	1000	1700000	1100000	1240000	0,0005964	47,67	7,000
1|9	38,79	1000	1500000	980000	1151000	0,0006603	49,45	6,000
1|10	44,12	1000	1300000	830000	1048000	0,0007466	51,47	5,000
1|11	49,24	1000	1200000	680000	927900	0,0008691	53,74	4,000
1|12	54,34	1000	940000	520000	787900	0,001059	56,56	3,000
1|13	59,36	1000	710000	350000	611300	0,001416	59,97	2,000
1|14	64,52	1000	420000	180000	375900	0,002403	64,60	1,000

-Table (D-3): Frequency Sweep Test at Constant Stress for 5% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 5% Crumb Rubber Modified Bitumen .								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,625	1010	4300000	3900000	1807000	0,0002339	24,78	50,00
1|2	5,962	1010	3900000	3500000	1751000	0,0002559	26,47	40,00
1|3	9,964	1000	3500000	3000000	1715000	0,0002896	29,66	30,00
1|4	14,36	1000	2900000	2400000	1640000	0,0003467	34,60	20,00
1|5	19,24	1000	2000000	1500000	1380000	0,0004993	43,55	10,00
1|6	24,49	1000	1900000	1300000	1333000	0,0005283	44,76	9,000
1|7	29,11	1000	1800000	1200000	1270000	0,0005686	46,22	8,000
1|8	33,71	1000	1600000	1100000	1201000	0,0006159	47,69	7,000
1|9	38,60	1000	1500000	960000	1122000	0,0006769	49,39	6,000
1|10	43,76	1000	1300000	820000	1025000	0,0007615	51,29	5,000
1|11	48,75	1000	1100000	670000	909500	0,0008841	53,51	4,000
1|12	53,86	1000	930000	520000	770600	0,001078	56,16	3,000
1|13	58,73	1000	700000	350000	598600	0,001437	59,36	2,000
1|14	63,96	1000	420000	180000	374400	0,002396	63,77	1,000

Table (D-4): Frequency Sweep Test at Constant Stress for 7% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 7% Crumb Rubber Modified Bitumen								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,791	1010	5100000	4700000	2004000	0,0001986	23,27	50,00
1|2	6,112	1010	4700000	4300000	2064000	0,0002118	25,78	40,00
1|3	10,28	1000	4200000	3700000	2051000	0,0002373	29,03	30,00
1|4	14,54	1000	3400000	2900000	1908000	0,0002910	33,67	20,00
1|5	19,53	1000	2300000	1700000	1587000	0,0004265	42,59	10,00
1|6	24,88	1000	2200000	1600000	1521000	0,0004556	43,83	9,000
1|7	29,67	1000	2000000	1400000	1437000	0,0004944	45,28	8,000
1|8	34,22	1000	1800000	1300000	1343000	0,0005416	46,67	7,000
1|9	38,99	1000	1700000	1100000	1245000	0,0006005	48,39	6,000
1|10	46,90	1000	1500000	940000	1131000	0,0006804	50,30	5,000
1|11	52,04	1000	1300000	760000	994800	0,0007981	52,55	4,000
1|12	56,94	1000	1000000	590000	830700	0,0009800	54,50	3,000
1|13	62,13	1000	750000	390000	642000	0,001328	58,50	2,000
1|14	67,64	1000	430000	200000	383500	0,002319	62,81	1,000

Table (D-5): Frequency Sweep Test at Constant Stress for 10% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 10% Crumb Rubber Modified Bitumen .								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,824	1010	4500000	4200000	1698000	0,0002244	22,20	50,00
1|2	6,224	1010	4200000	3800000	1733000	0,0002396	24,39	40,00
1|3	10,34	1000	3800000	3300000	1730000	0,0002665	27,36	30,00
1|4	14,63	1000	3200000	2700000	1661000	0,0003179	31,81	20,00
1|5	19,80	1000	2200000	1700000	1457000	0,0004477	40,68	10,00
1|6	25,10	1000	2100000	1600000	1412000	0,0004731	41,91	9,000
1|7	29,79	1000	2000000	1400000	1354000	0,0005069	43,35	8,000
1|8	34,27	1000	1800000	1300000	1292000	0,0005469	44,93	7,000
1|9	39,20	1000	1700000	1100000	1219000	0,0005980	46,77	6,000
1|10	44,36	1000	1500000	990000	1125000	0,0006682	48,71	5,000
1|11	49,39	1000	1300000	810000	1002000	0,0007769	51,08	4,000
1|12	54,25	1000	1100000	620000	848800	0,0009520	53,90	3,000
1|13	60,15	1000	770000	420000	652400	0,001291	57,39	2,000
1|14	65,88	1000	450000	210000	394200	0,002245	62,25	1,000

Appendix (E)
Frequency Sweep Test at Constant Stress

- Stress (τ) = 1000 (Pa)
- Temperatures (T) = 35 ^{o}C,
- Frequencies (F) = 0.1 to 50 (Hz)
- Bitumen thickness (t) =1 (mm)
- Spindle diameter (D) = 25 (mm)
- Materials: crumb rubber 0 %, 3%, 5%, 7%, and 10%

Table (E-1): Frequency Sweep Test at Constant Stress for Conventional Bitumen.

Frequency Sweep Test at Constant Stress Conventional Bitumen								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,704	1010	1800000	1200000	1344000	0,0005538	47,34	50
1|2	6,101	1010	1600000	1000000	1207000	0,0006400	49,99	40
1|3	10,33	1010	1300000	760000	1023000	0,0007871	53,25	30
1|4	14,50	1000	940000	510000	792100	0,001064	57,22	20
1|5	19,42	1000	560000	250000	499600	0,001789	63,26	10
1|6	24,60	1000	520000	230000	462800	0,001942	63,91	9
1|7	29,37	1000	470000	200000	423000	0,002139	64,71	8
1|8	33,93	1000	420000	170000	380400	0,002393	65,51	7
1|9	38,73	1000	370000	150000	339800	0,002698	66,40	6
1|10	43,90	1000	320000	120000	295700	0,003123	67,41	5
1|11	48,88	1000	270000	98000	247800	0,003755	68,51	4
1|12	53,78	1000	210000	72000	196000	0,004790	69,82	3
1|13	58,83	1000	150000	47000	140900	0,006726	71,44	2
1|14	64,17	1000	83000	23000	79490	0,01208	73,85	1

Table (E-2): Frequency Sweep Test at Constant Stress for 3% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 3% Crumb Rubber Modified Bitumen .								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,839	1010	2000000	1400000	1403000	0,0005102	45,05	50
1|2	6,238	1010	1700000	1200000	1288000	0,0005775	47,57	40
1|3	10,47	1000	1500000	930000	1134000	0,0006866	50,78	30
1|4	14,79	1000	1100000	630000	900600	0,0009126	55,07	20
1|5	19,96	1000	670000	320000	585600	0,001504	61,62	10
1|6	25,25	1000	610000	280000	542900	0,001633	62,36	9
1|7	29,91	1000	560000	250000	501700	0,001780	63,21	8
1|8	34,34	1000	510000	220000	459600	0,001958	64,11	7
1|9	39,02	1000	460000	190000	415100	0,002186	65,06	6
1|10	44,20	1000	400000	160000	362100	0,002527	66,18	5
1|11	49,34	1000	330000	130000	302100	0,003057	67,39	4
1|12	54,32	1000	250000	90000	233300	0,003999	68,84	3
1|13	59,75	1000	180000	59000	166500	0,005667	70,62	2
1|14	65,09	1000	100000	29000	95440	0,01003	73,13	1

Table (E-3): Frequency Sweep Test at Constant Stress for 5% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 5% Crumb Rubber Modified Bitumen .								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,578	1010	2100000	1500000	1439000	0,0004925	44,48	50
1|2	5,936	1010	1800000	1300000	1333000	0,0005493	46,59	40
1|3	10,18	1000	1500000	960000	1148000	0,0006707	50,01	30
1|4	14,56	1000	1100000	660000	912600	0,0008902	54,13	20
1|5	19,53	1000	680000	340000	589000	0,001476	60,33	10
1|6	24,72	1000	620000	300000	544000	0,001610	61,09	9
1|7	29,46	1000	560000	270000	496900	0,001775	61,84	8
1|8	33,93	1000	510000	230000	449700	0,001976	62,66	7
1|9	38,79	1000	450000	200000	400900	0,002235	63,56	6
1|10	43,97	1000	390000	170000	348300	0,002594	64,59	5
1|11	48,99	1000	320000	130000	294100	0,003100	65,72	4
1|12	54,07	1000	260000	100000	238900	0,003855	67,05	3
1|13	59,66	1000	190000	68000	175500	0,005310	68,74	2
1|14	66,01	1000	110000	34000	100700	0,009401	71,16	1

Table (E-4): Frequency Sweep Test at Constant Stress for 7% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 7% Crumb Rubber Modified Bitumen								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,630	1010	2300000	1600000	1588000	0,0004490	44,87	50
1|2	6,063	1010	1900000	1300000	1408000	0,0005187	46,46	40
1|3	10,17	1000	1600000	1000000	1216000	0,0006327	49,97	30
1|4	14,51	1000	1200000	690000	956800	0,0008484	54,08	20
1|5	19,35	1000	690000	340000	602800	0,001443	60,37	10
1|6	24,59	1000	630000	310000	556200	0,001576	61,17	9
1|7	29,14	1000	570000	270000	505000	0,001749	62,01	8
1|8	33,71	1000	510000	230000	453000	0,001965	62,84	7
1|9	38,71	1000	450000	200000	400200	0,002242	63,75	6
1|10	43,93	1000	380000	160000	346800	0,002610	64,80	5
1|11	49,01	1000	320000	130000	293200	0,003117	66,02	4
1|12	54,25	1000	260000	99000	238400	0,003872	67,38	3
1|13	59,55	1000	190000	67000	175200	0,005334	69,12	2
1|14	65,08	1000	100000	33000	98910	0,009595	71,63	1

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Table (E-5): Frequency Sweep Test at Constant Stress for 10% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 10% Crumb Rubber								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,887	1010	2300000	1600000	1603000	0,0004430	44,65	50
1|2	6,078	1010	2000000	1400000	1454000	0,0005008	46,29	40
1|3	10,21	1000	1700000	1100000	1263000	0,0006071	49,74	30
1|4	14,44	1000	1200000	730000	1002000	0,0008081	53,91	20
1|5	19,40	1000	750000	370000	648300	0,001341	60,30	10
1|6	24,65	1000	680000	330000	596900	0,001467	61,03	9
1|7	29,25	1000	620000	290000	544400	0,001621	61,88	8
1|8	33,80	1000	550000	250000	488800	0,001819	62,70	7
1|9	38,75	1000	480000	210000	430000	0,002085	63,68	6
1|10	43,91	1000	410000	180000	371400	0,002435	64,72	5
1|11	48,84	1000	340000	140000	312000	0,002926	65,90	4
1|12	53,76	1000	270000	110000	251300	0,003672	67,29	3
1|13	58,89	1000	200000	70000	182200	0,005126	69,02	2
1|14	64,13	1000	110000	35000	103600	0,009160	71,54	1

Appendix (F)
Frequency Sweep Test at Constant Stress

- Stress (τ) = 1000 (Pa)
- Temperatures (T) = 45 ^{o}C,
- Frequencies (F) = 0.1 to 50 (Hz)
- Bitumen thickness (t) =1 (mm)
- Spindle diameter (D) = 25 (mm)
- Materials: crumb rubber 0 %, 3%, 5%, 7%, and 10%

Table (F-1): Frequency Sweep Test at Constant Stress for Conventional Bitumen

Frequency Sweep Test at Constant Stress Conventional Bitumen								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,561	1020	520000	200000	482000	0,001949	67,03	50,00
1|2	5,914	1010	440000	160000	406100	0,002316	67,91	40,00
1|3	10,06	1010	340000	120000	321300	0,002940	69,28	30,00
1|4	14,44	1010	250000	81000	235800	0,004035	71,12	20,00
1|5	19,27	1000	140000	38000	133200	0,007238	74,13	10,00
1|6	24,74	1000	130000	34000	121200	0,007964	74,45	9,000
1|7	29,40	1000	110000	30000	109400	0,008836	74,79	8,000
1|8	33,91	1000	100000	26000	98290	0,009850	75,20	7,000
1|9	38,64	1000	89000	22000	86360	0,01123	75,63	6,000
1|10	43,94	1000	76000	18000	73530	0,01322	76,18	5,000
1|11	49,00	1000	62000	14000	60740	0,01604	76,81	4,000
1|12	54,09	1000	48000	10000	47210	0,02070	77,62	3,000
1|13	59,52	1000	34000	6700	33320	0,02944	78,69	2,000
1|14	64,97	1000	18000	3100	18070	0,05458	80,42	1,000

Table (F-2): Frequency Sweep Test at Constant Stress for 3% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 3% Crumb Rubber								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,664	1020	560000	230000	508300	0,001831	65,85	50,00
1|2	5,956	1010	480000	190000	445800	0,002092	66,89	40,00
1|3	9,964	1010	370000	140000	347700	0,002700	68,40	30,00
1|4	14,32	1010	260000	87000	243400	0,003889	70,27	20,00
1|5	19,14	1000	150000	42000	141800	0,006769	73,36	10,00
1|6	24,39	1000	130000	38000	129600	0,007421	73,72	9,000
1|7	28,98	1000	120000	34000	118300	0,008142	74,11	8,000
1|8	33,46	1000	110000	29000	106000	0,009100	74,45	7,000
1|9	38,44	1000	97000	25000	93370	0,01035	74,95	6,000
1|10	43,67	1000	82000	21000	79800	0,01214	75,52	5,000
1|11	48,81	1000	68000	16000	66280	0,01466	76,17	4,000
1|12	53,68	1000	53000	12000	51680	0,01886	76,98	3,000
1|13	58,63	1000	37000	7700	36430	0,02686	78,11	2,000
1|14	64,08	1000	20000	3600	19930	0,04941	79,89	1,000

Table (F-3): Frequency Sweep Test at Constant Stress for 5% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 5% Crumb Rubber								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,606	1020	600000	260000	538700	0,001710	64,54	50,00
1|2	6,167	1010	490000	200000	447700	0,002062	65,47	40,00
1|3	10,37	1010	390000	150000	354300	0,002623	66,94	30,00
1|4	14,76	1010	280000	100000	260400	0,003601	68,81	20,00
1|5	19,53	1000	160000	50000	153900	0,006188	71,89	10,00
1|6	24,81	1000	150000	45000	140400	0,006796	72,21	9,000
1|7	29,55	1000	130000	40000	128400	0,007439	72,53	8,000
1|8	34,08	1000	120000	35000	115000	0,008325	72,98	7,000
1|9	38,79	1000	110000	30000	101600	0,009447	73,44	6,000
1|10	44,01	1000	91000	25000	87770	0,01096	74,02	5,000
1|11	48,99	1000	75000	20000	72650	0,01328	74,69	4,000
1|12	53,82	1000	59000	15000	56900	0,01702	75,53	3,000
1|13	59,01	1000	41000	9500	40330	0,02414	76,72	2,000
1|14	64,43	1000	23000	4500	22190	0,04418	78,61	1,000

Table (F-4): Frequency Sweep Test at Constant Stress for 7% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 7% **Crumb Rubber**								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,566	1020	580000	260000	515900	0,001766	63,12	50,00
1|2	5,999	1010	540000	230000	483200	0,001893	64,38	40,00
1|3	10,10	1010	450000	180000	412400	0,002237	66,12	30,00
1|4	14,62	1010	320000	120000	294700	0,003165	68,12	20,00
1|5	19,47	1000	180000	58000	171900	0,005522	71,30	10,00
1|6	24,63	1000	160000	52000	156000	0,006096	71,66	9,000
1|7	29,19	1000	150000	46000	141700	0,006724	72,01	8,000
1|8	33,66	1000	130000	40000	126500	0,007550	72,48	7,000
1|9	38,32	1000	120000	34000	110900	0,008632	72,95	6,000
1|10	43,42	1000	100000	28000	96020	0,009995	73,53	5,000
1|11	48,46	1000	83000	23000	79780	0,01207	74,23	4,000
1|12	53,33	1000	65000	17000	62460	0,01543	74,42	3,000
1|13	58,62	1000	45000	11000	44210	0,02198	76,34	2,000
1|1	1,566	1020	580000	260000	515900	0,001766	63,12	50,00

Table (F-5): Frequency Sweep Test at Constant Stress for 10% Crumb Rubber Modified Bitumen .

Frequency Sweep Test at Constant Stress 10% **Crumb Rubber**								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	1,575	1020	680000	310000	602900	0,001507	63,09	50,00
1|2	5,928	1010	610000	270000	553300	0,001651	64,41	40,00
1|3	10,01	1010	480000	200000	441400	0,002089	66,09	30,00
1|4	14,30	1000	330000	120000	306700	0,003038	68,00	20,00
1|5	19,17	1000	180000	59000	173700	0,005456	71,13	10,00
1|6	24,50	1000	160000	52000	156400	0,006074	71,51	9,000
1|7	29,35	1000	150000	46000	140900	0,006755	71,86	8,000
1|8	33,90	1000	130000	40000	126200	0,007558	72,33	7,000
1|9	38,94	1000	120000	35000	111700	0,008561	72,77	6,000
1|10	44,28	1000	100000	29000	95930	0,009995	73,37	5,000
1|11	49,38	1000	83000	23000	79410	0,01212	74,09	4,000
1|12	54,62	1000	65000	17000	62790	0,01539	74,96	3,000
1|13	59,91	1000	46000	11000	44710	0,02173	76,19	2,000
1|14	65,48	1000	25000	5200	24780	0,03950	78,10	1,000

Appendix (G)

Frequency Sweep Test at Constant Stress

- Stress (τ) = 1000 (Pa)
- Temperatures (T) = 20 °C,
- Frequencies (F) = 0.1 to 30 (Hz)
- Bitumen thickness (t) =1 (mm)
- Spindle diameter (D) = 25 (mm)
- Materials: SBS 3%, 5%, 7%, and 10%

Table (G-1): Frequency Sweep Test at Constant Stress for 3% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 3% **SBS.**								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	2.825	1000	6000000	5700000	1854000	0.0001675	18.05	30.00
1|2	7.514	1000	5200000	4900000	1849000	0.0001912	20.67	20.00
1|3	12.42	1000	4100000	3700000	1839000	0.0002425	26.48	10.00
1|4	17.70	1000	4000000	3500000	1805000	0.0002528	27.15	9.000
1|5	22.05	1000	3700000	3300000	1777000	0.0002674	28.37	8.000
1|6	26.59	1000	3500000	3100000	1733000	0.0002850	29.59	7.000
1|7	31.27	1000	3200000	2800000	1670000	0.0003091	31.08	6.000
1|8	36.44	1000	2900000	2500000	1603000	0.0003391	32.93	5.000
1|9	40.82	1000	2600000	2100000	1497000	0.0003848	35.16	4.000
1|10	45.58	1000	2200000	1700000	1359000	0.0004530	37.99	3.000
1|11	50.05	1000	1700000	1300000	1155000	0.0005769	41.78	2.000
1|12	55.34	1000	1100000	750000	818600	0.0009003	47.48	1.000

Table (G-2): Frequency Sweep Test at Constant Stress for 5% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 5% **SBS**								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	0.02246	1000	7100000	7000000	1106000	0.0001421	9.016	30.00
1|2	1.437	1000	6400000	6200000	1647000	0.0001556	14.82	20.00
1|3	2.923	1000	5200000	4900000	1682000	0.0001922	18.85	10.00
1|4	5.096	1000	5100000	4800000	1743000	0.0001950	19.87	9.000
1|5	6.863	1000	4900000	4500000	1787000	0.0002053	21.52	8.000
1|6	9.560	1000	4700000	4300000	1755000	0.0002146	22.12	7.000
1|7	12.13	1000	4400000	4000000	1789000	0.0002267	23.92	6.000
1|8	14.66	1000	4100000	3700000	1776000	0.0002423	25.49	5.000
1|9	17.56	1000	3800000	3300000	1774000	0.0002660	28.15	4.000
1|10	20.79	1000	3300000	2800000	1703000	0.0003035	31.11	3.000
1|11	24.83	1000	2700000	2100000	1560000	0.0003766	35.99	2.000
1|12	30.68	1000	1700000	1200000	1215000	0.0005747	44.29	1.000

Table (F-5): Frequency Sweep Test at Constant Stress for 7% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 7% SBS								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	3.036	1000	7200000	7000000	1670000	0.0001388	13.36	30.00
1|2	7.610	1000	6600000	6400000	1694000	0.0001510	14.80	20.00
1|3	12.47	1000	5600000	5300000	1873000	0.0001778	19.44	10.00
1|4	17.71	1000	5400000	5100000	1855000	0.0001847	20.03	9.000
1|5	21.90	1000	5200000	4800000	1853000	0.0001939	21.05	8.000
1|6	26.44	1000	4900000	4500000	1846000	0.0002048	22.22	7.000
1|7	31.11	1000	4600000	4200000	1837000	0.0002176	23.56	6.000
1|8	36.34	1000	4300000	3800000	1821000	0.0002350	25.34	5.000
1|9	40.77	1000	3900000	3400000	1761000	0.0002585	27.08	4.000
1|10	45.55	1000	3400000	2900000	1698000	0.0002943	29.97	3.000
1|11	50.12	1000	2800000	2300000	1551000	0.0003554	33.44	2.000
1|12	55.37	1000	2000000	1600000	1245000	0.0005010	38.59	1.000

Table (F-5): Frequency Sweep Test at Constant Stress for 10% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 10% **SBS**								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	2.926	1000	7100000	7000000	1348000	0.0001407	10.91	30.00
1|2	7.631	1000	6500000	6400000	1434000	0.0001529	12.64	20.00
1|3	12.44	1000	5700000	5400000	1734000	0.0001760	17.76	10.00
1|4	17.62	1000	5500000	5300000	1744000	0.0001806	18.36	9.000
1|5	21.73	1000	5300000	5000000	1780000	0.0001896	19.72	8.000
1|6	26.19	1000	5000000	4700000	1797000	0.0001999	21.05	7.000
1|7	30.97	1000	4700000	4300000	1819000	0.0002123	22.71	6.000
1|8	36.33	1000	4400000	4000000	1818000	0.0002296	24.67	5.000
1|9	40.78	1000	3900000	3500000	1810000	0.0002537	27.33	4.000
1|10	45.73	1000	3400000	2900000	1752000	0.0002915	30.71	3.000
1|11	50.57	1000	2800000	2300000	1614000	0.0003600	35.52	2.000
1|12	55.68	1000	1800000	1300000	1274000	0.0005407	43.52	1.000

Appendix (H)
Frequency Sweep Test at Constant Stress

- Stress (τ) = 1000 (Pa)
- Temperatures (T) = 27 °C,
- Frequencies (F) = 0.1 to 30 (Hz)
- Bitumen thickness (t) =1 (mm)
- Spindle diameter (D) = 25 (mm)
- Materials: SBS 3%, 5%, 7%, and 10%

Table (H-1): Frequency Sweep Test at Constant Stress for 3% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 3% **SBS**								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	2.843	1010	1400000	1100000	944100	0.0007110	41.84	30.00
1|2	7.365	1000	1200000	840000	844200	0.0008401	44.99	20.00
1|3	12.25	1000	820000	520000	626800	0.001226	50.16	10.00
1|4	17.43	1000	770000	490000	593600	0.001305	50.70	9.000
1|5	21.66	1000	710000	440000	553600	0.001412	51.39	8.000
1|6	26.03	1000	640000	390000	509100	0.001554	52.27	7.000
1|7	30.83	1000	570000	340000	459400	0.001743	53.14	6.000
1|8	36.06	1000	500000	290000	405300	0.002003	54.26	5.000
1|9	40.75	1000	420000	240000	347300	0.002372	55.45	4.000
1|10	45.65	1000	340000	190000	283800	0.002944	56.65	3.000
1|11	50.28	1000	250000	130000	214300	0.003964	58.15	2.000
1|12	55.57	1000	150000	77000	130100	0.006621	59.45	1.000

Table (H-2): Frequency Sweep Test at Constant Stress for 5% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 5% SBS								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	3.205	1000	4400000	3800000	2072000	0.0002297	28.33	30.00
1|2	7.606	1000	3600000	3000000	1946000	0.0002791	32.84	20.00
1|3	12.73	1000	2500000	1800000	1640000	0.0004060	41.73	10.00
1|4	18.06	1000	2300000	1700000	1572000	0.0004315	42.68	9.000
1|5	22.16	1000	2100000	1500000	1475000	0.0004725	44.15	8.000
1|6	26.69	1000	1900000	1300000	1375000	0.0005220	45.85	7.000
1|7	31.38	1000	1700000	1100000	1252000	0.0005901	47.62	6.000
1|8	36.61	1000	1500000	950000	1115000	0.0006836	49.65	5.000
1|9	41.20	1000	1200000	750000	962400	0.0008191	52.02	4.000
1|10	46.34	1000	940000	540000	770900	0.001059	54.75	3.000
1|11	51.12	1000	660000	350000	560700	0.001515	58.17	2.000
1|12	56.27	1000	370000	170000	327500	0.002712	62.67	1.000

Table (H-3): Frequency Sweep Test at Constant Stress for 7% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 7% **SBS**								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	3.010	1010	3400000	3100000	1539000	0.0002925	26.60	30.00
1|2	7.611	1000	3000000	2600000	1456000	0.0003377	29.38	20.00
1|3	12.74	1000	2300000	1800000	1291000	0.0004443	34.98	10.00
1|4	17.96	1000	2100000	1700000	1247000	0.0004686	35.74	9.000
1|5	22.28	1000	2000000	1600000	1190000	0.0005020	36.66	8.000
1|6	26.70	1000	1900000	1500000	1133000	0.0005397	37.67	7.000
1|7	31.48	1000	1700000	1300000	1070000	0.0005870	38.88	6.000
1|8	36.95	1000	1500000	1200000	997200	0.0006466	40.14	5.000
1|9	41.44	1000	1400000	1000000	914400	0.0007257	41.57	4.000
1|10	46.39	1000	1200000	870000	817700	0.0008390	43.32	3.000
1|11	50.98	1000	970000	680000	689000	0.001031	45.27	2.000
1|12	56.47	1000	680000	450000	500300	0.001480	47.79	1.000

Table (H-4): Frequency Sweep Test at Constant Stress for 10% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 10% SBS								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	2.918	1000	5300000	4900000	2007000	0.0001903	22.37	30.00
1|2	7.429	1000	4500000	4000000	1958000	0.0002244	26.02	20.00
1|3	12.42	1000	3200000	2700000	1793000	0.0003084	33.56	10.00
1|4	17.74	1000	3000000	2500000	1718000	0.0003337	34.96	9.000
1|5	21.92	1000	2800000	2200000	1656000	0.0003609	36.68	8.000
1|6	26.31	1000	2500000	2000000	1580000	0.0003935	38.42	7.000
1|7	31.01	1000	2300000	1700000	1493000	0.0004356	40.56	6.000
1|8	36.37	1000	2000000	1500000	1383000	0.0004897	42.62	5.000
1|9	40.62	1000	1800000	1300000	1259000	0.0005633	45.17	4.000
1|10	45.48	1000	1500000	980000	1094000	0.0006796	48.00	3.000
1|11	50.19	1000	1100000	700000	879400	0.0008901	51.51	2.000
1|12	55.48	1000	690000	390000	573500	0.001446	56.02	1.000

Appendix (I)

Frequency Sweep Test at Constant Stress

- Stress (τ) = 1000 (Pa)
- Temperatures (T) = 35 ^oC,
- Frequencies (F) = 0.1 to 30 (Hz)
- Bitumen thickness (t) =1 (mm)
- Spindle diameter (D) = 25 (mm)
- Materials: SBS 3%, 5%, 7%, and 10%

Table (I-1): Frequency Sweep Test at Constant Stress for 3% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 3% SBS								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	3.195	1010	1600000	970000	1312000	0.0006155	53.39	30.00
1|2	7.628	1000	1200000	670000	1020000	0.0008230	56.85	20.00
1|3	12.48	1000	670000	340000	639900	0.001383	62.12	10.00
1|4	17.51	1000	660000	300000	589000	0.001510	62.70	9.000
1|5	21.97	1000	590000	270000	530100	0.001687	63.35	8.000
1|6	26.53	1000	520000	230000	471400	0.001911	64.17	7.000
1|7	31.28	1000	460000	190000	414700	0.002186	64.98	6.000
1|8	36.67	1000	390000	160000	355400	0.002569	65.85	5.000
1|9	41.08	1000	330000	130000	302300	0.003043	66.88	4.000
1|10	46.06	1000	260000	99000	243800	0.003804	67.98	3.000
1|11	50.59	1000	190000	67000	176300	0.005306	69.25	2.000
1|12	55.82	1000	110000	35000	102400	0.009234	71.04	1.000

Table (I-2): Frequency Sweep Test at Constant Stress for 5% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 5% SBS								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	3.002	1010	1400000	880000	1093000	0.0007165	51.10	30.00
1|2	7.489	1000	1100000	640000	887800	0.0009172	54.24	20.00
1|3	12.48	1000	680000	350000	583800	0.001470	59.01	10.00
1|4	17.64	1000	630000	320000	539300	0.001601	59.60	9.000
1|5	21.79	1000	570000	280000	495000	0.001756	60.26	8.000
1|6	26.22	1000	510000	250000	445500	0.001963	60.91	7.000
1|7	30.91	1000	450000	210000	394200	0.002234	61.68	6.000
1|8	36.26	1000	380000	180000	338900	0.002617	62.41	5.000
1|9	40.85	1000	320000	140000	283100	0.003155	63.24	4.000
1|10	45.78	1000	250000	110000	224100	0.004018	64.21	3.000
1|11	50.22	1000	170000	75000	158200	0.005717	64.74	2.000
1|12	55.42	1000	100000	41000	93010	0.009849	66.36	1.000

Table (I-3): Frequency Sweep Test at Constant Stress for 7% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 7% SBS								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	3.205	1010	1400000	1000000	990200	0.0007159	44.77	30.00
1|2	7.719	1000	1100000	740000	799000	0.0009236	47.34	20.00
1|3	12.73	1000	700000	440000	546800	0.001430	51.35	10.00
1|4	17.86	1000	650000	400000	510600	0.001540	51.77	9.000
1|5	22.08	1000	600000	370000	473100	0.001673	52.27	8.000
1|6	26.42	1000	550000	330000	434900	0.001830	52.68	7.000
1|7	31.15	1000	500000	300000	396300	0.002021	53.19	6.000
1|8	36.52	1000	440000	260000	355800	0.002267	53.74	5.000
1|9	41.05	1000	380000	220000	310700	0.002615	54.30	4.000
1|10	45.99	1000	320000	180000	258100	0.003168	54.86	3.000
1|11	50.38	1000	240000	140000	197100	0.004176	55.40	2.000
1|12	55.68	1000	160000	89000	131300	0.006309	55.90	1.000

Table (I-4): Frequency Sweep Test at Constant Stress for 10% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 10% SBS.								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	2.839	1010	1900000	1400000	1241000	0.0005334	41.11	30.00
1|2	7.548	1000	1600000	1100000	1088000	0.0006383	43.81	20.00
1|3	12.32	1000	1100000	720000	803800	0.0009278	48.15	10.00
1|4	17.59	1000	1000000	680000	771300	0.0009739	48.63	9.000
1|5	21.73	1000	970000	630000	732300	0.001033	49.10	8.000
1|6	26.16	1000	900000	580000	687300	0.001113	49.84	7.000
1|7	30.84	1000	810000	520000	624400	0.001235	50.42	6.000
1|8	36.25	1000	670000	420000	523900	0.001489	51.25	5.000
1|9	40.63	1000	560000	350000	443500	0.001773	51.83	4.000
1|10	45.71	1000	460000	280000	366300	0.002172	52.70	3.000
1|11	50.45	1000	360000	210000	290300	0.002782	53.85	2.000
1|12	55.55	1000	230000	130000	191700	0.004284	55.23	1.000

Appendix (J)
Frequency Sweep Test at Constant Stress

- Stress (τ) = 1000 (Pa)
- Temperatures (T) = 45 °C,
- Frequencies (F) = 0.1 to 30 (Hz)
- Bitumen thickness (t) =1 (mm)
- Spindle diameter (D) = 25 (mm)
- Materials: SBS 3%, 5%, 7%, and 10%

Table (J-1): Frequency Sweep Test at Constant Stress for 3% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 3% SBS								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	3.254	1010	410000	170000	371900	0.002469	65.34	30.00
1|2	7.792	1010	310000	120000	281300	0.003290	66.95	20.00
1|3	12.62	1000	170000	62000	162700	0.005757	69.12	10.00
1|4	17.86	1000	160000	56000	147600	0.006353	69.33	9.000
1|5	22.01	1000	140000	49000	132500	0.007084	69.58	8.000
1|6	26.48	1000	130000	44000	118700	0.007921	69.78	7.000
1|7	31.30	1000	110000	38000	103900	0.009060	70.05	6.000
1|8	36.49	1000	94000	32000	88470	0.01066	70.38	5.000
1|9	41.07	1000	78000	26000	73670	0.01282	70.72	4.000
1|10	45.99	1000	62000	20000	58300	0.01624	71.17	3.000
1|11	50.61	1000	44000	14000	41940	0.02264	71.69	2.000
1|12	55.83	1000	25000	7600	23920	0.03985	72.38	1.000

Table (J-2): Frequency Sweep Test at Constant Stress for 5% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 5% SBS								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	2.891	1010	480000	240000	415900	0.002115	60.34	30.00
1|2	7.336	1010	350000	170000	307900	0.002872	61.41	20.00
1|3	12.17	1000	220000	99000	194600	0.004589	62.95	10.00
1|4	17.39	1000	200000	91000	179700	0.004972	63.01	9.000
1|5	21.66	1000	190000	84000	165500	0.005402	63.13	8.000
1|6	26.09	1000	170000	75000	149500	0.005986	63.27	7.000
1|7	30.82	1000	150000	66000	131800	0.006787	63.28	6.000
1|8	36.21	1000	130000	57000	114600	0.007812	63.45	5.000
1|9	40.78	1000	110000	49000	97450	0.009188	63.46	4.000
1|10	45.60	1000	88000	39000	78680	0.01140	63.67	3.000
1|11	50.27	1000	65000	28000	58300	0.01542	63.96	2.000
1|12	55.45	1000	39000	17000	35630	0.02532	64.42	1.000

Table (J-3): Frequency Sweep Test at Constant Stress for 7% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 7% SBS								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	2.823	1020	470000	290000	372900	0.002151	52.19	30.00
1|2	7.343	1010	370000	230000	295100	0.002710	52.45	20.00
1|3	12.19	1000	240000	150000	192500	0.004161	52.98	10.00
1|4	17.32	1000	220000	140000	178400	0.004480	52.84	9.000
1|5	21.65	1000	200000	120000	162400	0.004909	52.70	8.000
1|6	26.18	1000	190000	110000	149300	0.005335	52.66	7.000
1|7	31.04	1000	170000	100000	133900	0.005941	52.60	6.000
1|8	36.36	1000	150000	92000	119600	0.006637	52.41	5.000
1|9	40.74	1000	130000	80000	104600	0.007592	52.51	4.000
1|10	45.56	1000	110000	67000	86730	0.009121	52.24	3.000
1|11	50.22	1000	86000	53000	67840	0.01163	52.05	2.000
1|12	55.40	1000	58000	36000	45510	0.01732	52.02	1.000

Table (J-4): Frequency Sweep Test at Constant Stress for 10% SBS Modified bitumen

Frequency Sweep Test at Constant Stress 10% SBS								
Nr-Seg	t_seg in s	τ in Pa	|G*| in Pa	G' in Pa	G" in Pa	γ in -	δ in °	F (Hz)
1|1	2.802	1020	570000	380000	424700	0.001786	48.26	30.00
1|2	7.314	1010	460000	300000	343200	0.002212	48.79	20.00
1|3	11.98	1000	310000	200000	233000	0.003263	49.29	10.00
1|4	17.37	1000	280000	190000	213600	0.003545	49.04	9.000
1|5	21.51	1000	260000	170000	196900	0.003855	49.21	8.000
1|6	25.93	1000	230000	150000	176000	0.004301	49.05	7.000
1|7	30.72	1000	210000	140000	161100	0.004699	49.11	6.000
1|8	36.06	1000	190000	130000	143000	0.005268	48.80	5.000
1|9	40.43	1000	170000	110000	126400	0.005951	48.72	4.000
1|10	45.23	1000	140000	95000	108100	0.006967	48.84	3.000
1|11	49.76	1000	110000	75000	86100	0.008778	49.07	2.000
1|12	55.11	1000	78000	50000	59330	0.01286	49.71	1.000

Appendix K
Dissipated Energy Calculation

Typical Example for Dissipated energy Calculation for unmodified bitumen at 20 ^{o}C, the initial input energy is 100 Kpa., and the frequency 10 Hz.

- Number for Cycles = =B2*C2
- Delta Number for cycles = G3-G2
- Dissipated Energy =PI()*(A3^2)/(D3/SIN(E3*PI()/180))
- Dissipated energy/ cycle = =I3*H3
- Σ Dissipated Energy =K2+J3
- Dissipated Energy Ratio = Σ (K2)/ (I25)

All calculation were done as presented before chapter 3.

Table (K-1): Dissipated Energy for unmodified bitumen

No.,	A	B	C	D	E	F
1	Stress (Pa)	Frequency (Hz)	Time (S)	Complex Shear Modulus (Pa)	Phase angle in ^{o}	Strain (-)
2	100000	10	25.2	13546891	29.9	0.0295
3	100000	10	37.5	12972280	30.7	0.0308
4	100000	10	49.9	12593559	31.2	0.0317
5	100000	10	62.2	12297547	31.7	0.0325
6	100000	10	74.6	12058126	32.0	0.0331
7	100000	10	87.0	11844824	32.3	0.0337
8	100000	10	99.3	11657640	32.6	0.0343
9	100000	10	111.6	11487868	32.9	0.0348
10	100000	10	124.0	11331156	33.1	0.0353
11	100000	10	136.3	11183150	33.3	0.0357
12	100000	10	148.7	11030791	33.5	0.0362
13	100000	10	161.2	10882785	33.7	0.0367
14	100000	10	173.4	10739133	34.0	0.0372
15	100000	10	198.1	10443121	34.4	0.0383
16	100000	10	210.4	10286409	34.6	0.0389
17	100000	10	222.7	10116637	34.9	0.0395
18	100000	10	247.5	9750976	35.4	0.0410
19	100000	10	259.8	9546379	35.7	0.0419
20	100000	10	272.1	9315664	36.0	0.0429
21	100000	10	284.5	9071890	36.4	0.0441
22	100000	10	309.2	8501632	37.3	0.0471
23	100000	10	321.5	8179501	37.8	0.0490
24	100000	10	333.9	7826899	38.3	0.0512
25	100000	10	358.6	7030279	39.6	0.0570
26	100000	10	370.9	6586262	40.4	0.0609
27	100000	10	383.3	6103066	41.2	0.0658
28	100000	10	407.8	5023494	43.2	0.0801

	G	H	I	J	K	L
	Number for Cycles	Delta Number for cycles	Dissipated Energy	Dissipated energy/ cycle	Σ Dissipated Energy	Dissipated Energy Ratio
1	0	0	0	0	0	0
2	252	252	1157.3	291209	291209	252
3	375	123	1236.8	152355	443564	359
4	499	124	1294.0	160759	604322	467
5	622	123	1341.3	165329	769651	574
6	746	124	1381.8	170768	940419	681
7	870	124	1418.7	175907	1116326	787
8	993	123	1452.5	179158	1295484	892
9	1116	123	1484.0	181834	1477317	996
10	1240	124	1514.0	187971	1665288	1100
11	1363	123	1543.1	190325	1855613	1203
12	1487	124	1573.5	195354	2050967	1303
13	1612	124	1603.8	199117	2250084	1403
14	1734	123	1634.6	200284	2450368	1499
15	1981	247	1699.6	419277	2869645	1688
16	2104	123	1735.5	214062	3083707	1777
17	2227	123	1775.4	218981	3302689	1860
18	2475	247	1866.5	461963	3764652	2017
19	2598	123	1920.6	235330	3999982	2083
20	2721	123	1984.2	244736	4244718	2139
21	2845	124	2055.4	255182	4499901	2189
22	3092	247	2237.1	553668	5053568	2259
23	3215	123	2351.6	288150	5341718	2271
24	3339	124	2488.3	308931	5650650	2271
25	3586	247	2848.4	702656	6353306	2230
26	3709	123	3088.5	380946	6734252	2180
27	3833	123	3389.8	418111	7152363	2110
28	4078	246	4277.6	1051756	8204119	1918

Appendix (L)
Calculation for dissipated Energy for asphalt concrete mixtures

Typical example for calculation for dissipated energy for asphalt concrete mixtures using indirect tensile fatigue test result. All calculation were done as presented before chapter 5 using equation (2-4) and (2-5)

Table (L-1): Dissipated Energy for asphalt concrete mixtures.

Number for Cycles	Load N	Deformation (γ)	Phase angle δ	Dissipated Energy	Σ Dissipated Energy	DER
1	2000	0	33.7	1.82186E-05	1.82186E-05	1
2	2000	0.0315	34.5	3.10063E-06	2.13192E-05	7
3	2000	0.063	35.1	6.03674E-06	2.73559E-05	5
4	2000	0.063	35.6	5.87447E-06	3.32304E-05	6
5	2000	0.0945	36.0	8.55649E-06	4.17869E-05	5
6	2000	0.126	36.4	1.08325E-05	5.26194E-05	5
7	2000	0.1575	36.7	1.26043E-05	6.52237E-05	5
8	2000	0.189	37.0	1.42005E-05	7.94243E-05	6
9	2000	0.2205	37.2	1.53904E-05	9.48146E-05	6
10	2000	0.2205	37.5	1.41223E-05	0.000108937	8
11	2000	0.2835	37.7	1.70856E-05	0.000126023	7
12	2000	0.315	38.0	1.72394E-05	0.000143262	8
13	2000	0.3465	38.2	1.75432E-05	0.000160805	9
14	2000	0.4095	38.7	1.93493E-05	0.000180154	9
15	2000	0.441	38.9	2.00843E-05	0.000200239	10
16	2000	0.504	39.2	2.16463E-05	0.000221885	10
17	2000	0.5355	39.8	2.25319E-05	0.000244417	11
18	2000	0.567	40.2	2.33604E-05	0.000267777	11
19	2000	0.5985	40.5	2.2528E-05	0.000290305	13
20	2000	0.6615	40.9	2.60867E-05	0.000316392	12
21	2000	0.7245	41.9	2.85712E-05	0.000344963	12
22	2000	0.7875	42.5	3.10556E-05	0.000376019	12
23	2000	0.819	43.1	3.37428E-05	0.000409762	12
24	2000	0.882	44.5	3.63384E-05	0.0004461	12
25	2000	0.9135	45.4	3.92339E-05	0.000485334	12
26	2000	0.9765	46.3	4.36322E-05	0.000528966	12
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Appendix (M)

Indirect tensile test result

Indirect tensile test result for modified asphalt concrete mixtures, 7% Crumb Rubber three specimens.

Table (M-1): Indirect tensile test result for7% Crumb Rubber

ITS for Marshall specimen (1)			ITS for Marshall specimen (2)			ITS for Marshall specimen (3)		
Number	Load (N)	Deformation (1/1000) mm	Number	Load (N)	Deformation (1/1000) mm	Number	Load (N)	Deformation (1/1000) mm
1	50	20	1	37	30	1	25	10
2	75	40	2	50	60	2	37	40
3	112	70	3	62	90	3	37	60
4	149	90	4	87	110	4	37	90
5	199	120	5	112	130	5	50	110
6	261	140	6	162	150	6	50	140
7	324	170	7	211	180	7	50	160
8	398	190	8	286	200	8	62	180
9	473	210	9	336	230	9	75	210
10	572	240	10	435	250	10	112	230
11	684	260	11	522	270	11	112	260
12	796	290	12	609	300	12	137	280
13	921	310	13	684	330	13	162	310
14	1045	340	14	783	350	14	199	330
15	1182	370	15	870	380	15	236	360
16	1319	390	16	982	400	16	286	380
17	1468	410	17	1069	430	17	336	400
18	1618	440	18	1181	450	18	398	430
19	1754	460	19	1281	480	19	485	450
20	1916	490	20	1393	500	20	572	480
21	2065	510	21	1505	530	21	672	500
22	2240	540	22	1616	550	22	784	530
23	2401	560	23	1728	580	23	896	550
24	2576	590	24	1865	600	24	1020	580
25	2737	610	25	1965	630	25	1144	600
26	2899	630	26	2089	650	26	1269	630
27	3061	660	27	2213	670	27	1418	650
28	3235	680	28	2350	700	28	1567	680
29	3372	710	29	2474	720	29	1729	700
30	3534	730	30	2599	750	30	1903	730
31	3671	750	31	2723	770	31	2077	750
32	3820	780	32	2860	800	32	2264	780
33	3957	800	33	2972	820	33	2438	800
34	4118	830	34	3109	850	34	2637	830
35	4255	850	35	3208	870	35	2811	850
36	4380	880	36	3332	900	36	3023	880
37	4517	900	37	3432	920	37	3222	900
38	4653	930	38	3556	950	38	3421	930

39	4778	950	39	3656	970	39	3620	950
40	4902	980	40	3768	1000	40	3831	980
41	5014	1000	41	3880	1020	41	4030	1000
42	5139	1030	42	3979	1050	42	4241	1020
43	5263	1050	43	4079	1070	43	4428	1050
44	5375	1070	44	4190	1090	44	4640	1070
45	5500	1100	45	4265	1120	45	4826	1100
46	5624	1120	46	4377	1150	46	5038	1120
47	5748	1150	47	4452	1170	47	5212	1150
48	5860	1170	48	4563	1190	48	5398	1170
49	5972	1200	49	4638	1220	49	5585	1200
50	6084	1220	50	4738	1240	50	5784	1220
51	6184	1250	51	4825	1270	51	5958	1250
52	6308	1270	52	4912	1290	52	6070	1270
53	6383	1300	53	4999	1320	53	6306	1300
54	6507	1320	54	5098	1340	54	6493	1320
55	6594	1350	55	5173	1370	55	6655	1340
56	6706	1370	56	5260	1390	56	6829	1370
57	6794	1400	57	5322	1420	57	6978	1400
58	6906	1420	58	5421	1440	58	7140	1420
59	6980	1450	59	5471	1470	59	7276	1440
60	7067	1470	60	5571	1490	60	7438	1470
61	7154	1490	61	5633	1520	61	7575	1490
62	7241	1520	62	5720	1540	62	7724	1520
63	7329	1550	63	5770	1570	63	7861	1540
64	7403	1570	64	5857	1590	64	7998	1570
65	7478	1590	65	5906	1620	65	8135	1590
66	7565	1620	66	5981	1640	66	8259	1620
67	7640	1640	67	6031	1670	67	8371	1640
68	7689	1670	68	6118	1690	68	8508	1670
69	7752	1690	69	6180	1720	69	8620	1690
70	7839	1720	70	6242	1740	70	8744	1720
71	7901	1740	71	6317	1770	71	8856	1740
72	7963	1770	72	6354	1790	72	8968	1770
73	8025	1790	73	6404	1820	73	9068	1790
74	8100	1820	74	6478	1840	74	9180	1820
75	8150	1840	75	6528	1870	75	9267	1840
76	8212	1870	76	6590	1890	76	9366	1860
77	8262	1890	77	6652	1910	77	9466	1890
78	8324	1920	78	6715	1940	78	9553	1920
79	8386	1940	79	6752	1970	79	9640	1940
80	8436	1970	80	6814	1990	80	9714	1960
81	8473	1990	81	6851	2020	81	9801	1990
82	8548	2020	82	6914	2040	82	9889	2010
83	8585	2040	83	6951	2070	83	9951	2040
84	8635	2070	84	7001	2090	84	10038	2060
85	8660	2090	85	7050	2110	85	10112	2090
86	8710	2120	86	7100	2140	86	10175	2110
87	8747	2140	87	7137	2170	87	10249	2140
88	8797	2170	88	7187	2190	88	10311	2160
89	8834	2190	89	7224	2220	89	10374	2190

90	8871	2220	90	7274	2240	90	10448	2210
91	8909	2240	91	7311	2270	91	10486	2240
92	8946	2270	92	7361	2290	92	10560	2260
93	8983	2290	93	7386	2310	93	10598	2290
94	9008	2310	94	7448	2340	94	10672	2310
95	9046	2340	95	7473	2370	95	10722	2340
96	9071	2370	96	7510	2390	96	10784	2360
97	9108	2390	97	7535	2420	97	10809	2390
98	9133	2410	98	7585	2440	98	10871	2410
99	9158	2440	99	7610	2470	99	10908	2440
100	9195	2460	100	7647	2490	100	10958	2460
101	9220	2490	101	7660	2510	101	10996	2490
102	9257	2510	102	7697	2540	102	11045	2510
103	9282	2540	103	7722	2570	103	11083	2540
104	9307	2570	104	7772	2590	104	11120	2560
105	9319	2590	105	7796	2620	105	11145	2590
106	9357	2610	106	7821	2640	106	11195	2610
107	9382	2640	107	7846	2670	107	11219	2640
108	9406	2670	108	7883	2690	108	11269	2660
109	9419	2690	109	7908	2720	109	11269	2690
110	9444	2710	110	7933	2740	110	11319	2710
111	9456	2740	111	7946	2770	111	11344	2740
112	9469	2770	112	7983	2790	112	11369	2760
113	9469	2790	113	7995	2820	113	11381	2790
114	9481	2810	114	8020	2840	114	11418	2810
115	9494	2840	115	8033	2870	115	11431	2840
116	9518	2860	116	8058	2890	116	11456	2860
117	9518	2890	117	8070	2920	117	11468	2890
118	9531	2910	118	8095	2940	118	11481	2910
119	9556	2940	119	8107	2970	119	11481	2940
120	9568	2960	120	8132	2990	120	11493	2960
121	9581	2990	121	8145	3020	121	11493	2990
122	9581	3010	122	8169	3040	122	11518	3010
123	9593	3040	123	8194	3070	123	11530	3040
124	9606	3060	124	8207	3090	124	11543	3060
125	9618	3090	125	8219	3120	125	11530	3090
126	9630	3110	126	8244	3140	126	11530	3110
127	9630	3140	127	8244	3170	127	11530	3140
128	9630	3160	128	8269	3190	128	11530	3160
129	9643	3190	129	8269	3220	129	11518	3190
130	9643	3210	130	8281	3240	130	11530	3210
131	9643	3240	131	8294	3270	131	11530	3240
132	9643	3260	132	8319	3290	132	11530	3260
133	9655	3290	133	8331	3320	133	11506	3290
134	9643	3310	134	8344	3340	134	11506	3310
135	9655	3340	135	8344	3370	135	11493	3340
136	9668	3360	136	8368	3390	136	11493	3360
137	9668	3390	137	8368	3420	137	11468	3380
138	9668	3410	138	8393	3440	138	11456	3410
139	9655	3440	139	8393	3470	139	11431	3440
140	9655	3460	140	8406	3490	140	11418	3460

141	9668	3490	141	8418	3520	141	11406	3490
142	9668	3510	142	8431	3540	142	11369	3510
143	9643	3540	143	8431	3570	143	11344	3540
144	9643	3560	144	8443	3590	144	11331	3560
145	9643	3590	145	8443	3620	145	11294	3580
146	9643	3610	146	8455	3640	146	11269	3610
147	9630	3640	147	8468	3670	147	11232	3630
148	9618	3660	148	8493	3690	148	11207	3660
149	9618	3690	149	8480	3720	149	11170	3680
150	9606	3710	150	8505	3740	150	11132	3710
151	9581	3740	151	8493	3770	151	11107	3730
152	9568	3760	152	8518	3790	152	11070	3760
153	9581	3790	153	8493	3820	153	11008	3780
154	9556	3810	154	8518	3840	154	10983	3810
155	9543	3830	155	8518	3870	155	10921	3830
156	9531	3860	156	8518	3890	156	10896	3860
157	9518	3890	157	8518	3920	157	10821	3880
158	9506	3910	158	8543	3940	158	10784	3910
159	9481	3940	159	8530	3970	159	10734	3930
160	9469	3960	160	8543	3990	160	10685	3960
161	9456	3990	161	8530	4020	161	10622	3980
162	9431	4010	162	8543	4040	162	10573	4000
163	9419	4040	163	8530	4070	163	10510	4030
164	9382	4070	164	8543	4090	164	10473	4060
165	9369	4090	165	8530	4120	165	10411	4080
166	9357	4110	166	8518	4140	166	10361	4110
167	9319	4140	167	8518	4170	167	10287	4130
168	9307	4160	168	8518	4190	168	10237	4160
169	9282	4190	169	8493	4220	169	10175	4180
170	9245	4210	170	8518	4240	170	10125	4210
171	9220	4240	171	8493	4270	171	10050	4230
172	9182	4260	172	8518	4290	172	10000	4260
173	9170	4290	173	8493	4320	173	9926	4280
174	9158	4310	174	8493	4340	174	9864	4310
175	9120	4340	175	8493	4370	175	9789	4330
176	9095	4370	176	8480	4400	176	9739	4360
177	9058	4390	177	8468	4420	177	9652	4380
178	9046	4410	178	8455	4450	178	9590	4410
179	8996	4440	179	8455	4470	179	9515	4430
180	8983	4470	180	8455	4500	180	9466	4460
181	8934	4490	181	8443	4520	181	9391	4480
182	8909	4520	182	8443	4550	182	9329	4510
183	8884	4540	183	8418	4570	183	9242	4530

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Education

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Civil Engineering -Road and Pavement Engineering, Technische Universität Darmstadt, Germany (2010).
M.Sc.: Civil Engineering – Highway Engineering (South valley University, 2005, Distinction 90.75%).
B.Sc.: Civil Engineering (south valley University, Aswan, Egypt, June 2000), " 79.95% Very Good with honour degree ".

Computer Skills

SAP, Autocad, Matlab, ABAQUS and Microsoft Offic.

Work Experience

2006/11 – 2010/03:
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2005/12 – 2006/11:
- Assistant Lecturer, Aswan Faculty of Engineering, South Valley University.

2002/04 – 2005/12:
- Demonstrator in the Faculty of Enigineering -Aswan –Egypt.
- Team leader assistant in the U.S.-Egypt Science and Technology Joint fund in Cooperation with U.S National Science Foundation under project MAN7-001-001 (Manufacturing of Asphaltic super pavement Employing polymer technology).

2001/12- 2002/04:
- Site Engineer: Department of Irrigation , Ministry of water resources and irrigation, Qena, Egypt.

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Conference And Workshop

- Sep.14-15 /2009 Darmstädter Ingenierkongress – Bau und Umwelt; The 1st Darmstadt Engineering Congress – Civil Engineering and Environment, Darmstadt, Germany
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