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MECHANICAL PERFORMANCE AND FATIGUE CRACK GROWTH BEHAVIOR
OF POLYMER MODIFIED ASPHALT CONCRETE MIXTURES

by

AYMAN MAHMoud OTHMAN

Submitted in partial fulfillment of the requirements
for the Degree of Doctor of Philosophy

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May, 1995
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GRADUATE STUDIES

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MECHANICAL PERFORMANCE AND FATIGUE CRACK GROWTH BEHAVIOR
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ABSTRACT

by

AYMAN MAHMOUD OTHMAN

The mechanical performance and fatigue crack growth behavior of SBS modified asphalt concrete mixtures subjected to varying loading and environmental conditions were studied. Paris’ and the Modified Crack Layer (MCL) models have been used to characterize the resistance of asphalt concrete mixtures to fatigue crack propagation. Both models successfully discriminate the effect of processing conditions on the fracture resistance of the AC-20 asphalt concrete mixtures as well as the effect of SBS content on the fracture resistance of the modified AC-5 mixture. SEM examination of the
fracture surface of the SBS modified mixtures revealed ridge formation in binder rich areas which increases in intensity with the SBS percent in the mixture.

Using AC-20 asphalt concrete mixture, the current study demonstrates the dependency of the Paris' and the MCL models parameters on the level of stress during fatigue loading. Fatigue crack growth analysis of SBS modified mixtures at various temperatures also revealed that Paris’ model parameters ($C^*$ and $m^*$) are both independent of temperature. However, the MCL model indicates only the consistency of the specific energy of damage $\gamma'$ with temperature, while the dissipative coefficient $\beta'$ showed dependency on the temperature. The accumulated dissipated energy through fatigue failure which is introduced in this research as a fracture criteria, was correlated to fatigue life through temperature independent parameters.

A cyclic thermal aging program was performed in an environmental chamber between 0° F and 70° F for five different numbers of cycles. The current investigation revealed that the SBS modifier helps maintain a constant fatigue resistance of the mixture over a moderate number of low temperature thermal cycles. A decrease in the fatigue resistance was observed at higher number of thermal cycles. The fatigue performance of the mixtures as predicted from Paris’ and the MCL models was found to be consistent with what was predicted from the conventional indirect tensile strength and the critical energy release rate ($J_{ic}$).
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Chapter 1

Introduction

The durability of asphalt concrete mixtures is greatly influenced by the environmental changes during the year especially between summer and winter and between day and night, when the daily average temperature changes can be considerably large. During the summer, high temperature can soften the asphalt binder and consequently reduce the stiffness of the paving mixture. On the other hand, in the winter low temperatures can stiffen the asphalt binder and reduce the flexibility of the paving mixture. As a result, thermal cracking of the pavement surface may develop which adversely affects the performance of the paving mixture. Thus, high temperature stiffness and low temperature flexibility are important properties which increase the life of asphalt concrete pavements.

Various elastomer and plastomer modifiers have been sought in an attempt to address this problem. Polymer modifiers vary in function and effectiveness. Elastomers, which are at least to some extent derived from a diene chemical structure, will toughen asphalt and improve low temperature properties. Plastomers, which come from non-diene
chemicals, improve the high temperature viscoelastic properties of softer asphalt, and have good intrinsic low temperature properties. The properties of asphalt mixtures can be improved by selecting modifiers in the proper molecular weight range and by combining the modifiers with asphalt mixtures in an appropriate manner. In addition, these modifiers must have solubility properties close to those of the asphalt mixtures.

In the current study, evaluation of the mechanical properties and fatigue characteristics of SBS (Styrene-Butadiene-Styrene) modified AC-5 asphalt concrete mixtures subjected to a changing loading and environmental conditions is undertaken. The choice of SBS was influenced by the fact that it displayed superior fatigue resistance among three modifiers tested previously. SBS consists of two different polymer blocks: hard polystyrene endblocks chemically crosslinked to soft rubbery polybutadiene midblocks in a three-dimensional rubber network. Therefore, SBS is expected to improve temperature susceptibility.

The AAMAS procedure developed in 1991 (Von Quintus et al., 1991) requires the indirect tensile strength and the resilient modulus tests to simulate the characteristics of asphalt mixtures placed in the field. Therefore, these two tests will be used in this research to study the mechanical performance of asphalt concrete mixtures. Also, the concept of the critical energy release rate \((J_{\text{c}})\) as a fracture criteria for asphalt concrete mixtures has been adopted in this research.

Asphalt concrete pavements are continuously subjected to dynamic loads by moving vehicles. The repetitive nature of traffic loading on pavements has led to laboratory
investigations to study their performance under cyclic loading. Extensive research has been carried out in this field using two main approaches; a phenomenological approach and a fracture mechanics approach. The phenomenological approach is based on the endurance concept using the S-N curve (stress range versus the number of cycles to final failure) to predict the fatigue life of the asphalt concrete pavement. The fracture mechanics approach is based on the equation derived by Paris et al. [Paris et al., 1963] to correlate the stress intensity factor K with the crack speed da/dN.

In this research, Paris’ and the Modified Crack Layer models (through a fracture mechanics approach) will be used to identify and determine material parameters responsible for the asphalt concrete resistance to crack propagation, i.e. fracture toughness. Among these parameters $C^*$ and $m^*$ for Paris’ model are considered crack growth characteristics parameters. Similarly for the Modified Crack Layer model, $\gamma'$ (the specific energy of damage, in-lb/in$^3$) which can be defined as the energy required to cause a unit volume of the material to change from undamaged to damaged, and $\beta'$ (the dissipation coefficient) which reflects the percentage of energy expended on dissipative processes and damage growth within the active zone, are the corresponding crack growth characteristics parameters. $C^*$, $m^*$, $\gamma'$ and $\beta'$ are proposed as material parameters to characterize the material’s resistance to fatigue crack propagation.

Chapter 2 reviews the extent of the use of both fracture mechanics and phenomenological approaches in evaluating pavement systems. Also, a brief review in the topics of asphalt additives and modified asphalt concrete mixtures performance is
presented, as well as an introduction to the indirect tensile strength, resilient modulus and critical energy release rate tests.

The laboratory testing used to evaluate the mechanical performance and the resistance of asphalt concrete mixtures to crack propagation is described in Chapter 3. The study has been focused in determining the effect of SBS when added to asphalt cement in improving the resistance of asphalt concrete mixtures to fatigue crack propagation. Also included in this chapter are descriptions of the procedures used to prepare specimens for indirect tensile strength, resilient modulus, critical energy release rate and fatigue crack propagation testing of asphalt concrete, as well as the testing machine developed to conduct the fatigue crack propagation tests. The last section of this chapter describes the experimental procedure followed for Scanning Electron Microscopy (SEM) analysis.

Paris’ and the Modified Crack Layer models, which are used in the crack propagation studies, are introduced and discussed in detail in Chapter 4. The objective of the crack propagation studies is to identify and determine parameters responsible for a mixture’s resistance to crack propagation, i.e. fracture toughness.

Although extensive research has been done on various aspects of asphalt pavements to reduce their susceptibility to cracking, studies on the effect of compaction on pavement performance have not been given as much attention. Therefore, the effect of processing conditions on both mechanical and fatigue performance of AC-20 mixtures will be studied in Chapter 5. Also, the dependency of Paris’ and the Modified Crack Layer
models parameters on the level of stress during fatigue loading will be examined in Chapter 5.

The effect of SBS content on the mechanical and fatigue behavior of AC-5 asphalt concrete mixtures is studied in Chapter 6. Scanning Electron Microscopy (SEM) analysis is performed on asphalt concrete samples modified with different contents of SBS to be compared with unmodified samples.

In Chapter 7, the effect of temperature on the mechanical and fatigue behavior of both AC-5 and SBS modified AC-5 mixtures is examined. Also, the total dissipated energy to failure in fatigue testing is proposed as a fracture criteria which accounts for the effect of both temperature and stress level. Focus is placed on the evaluation of AC-5 mixtures and SBS modified AC-5 mixtures under thermal cycling aging at a low temperature regime. The performance of both mixtures is evaluated based on their resistance to fatigue crack propagation as well as their mechanical properties. Fatigue crack propagation analysis is performed in view of both Paris' and the Modified Crack Layer models.

Finally, conclusions and recommendations for future work have been included in Chapter 8.

In summary specific objectives of this research include:

2. Evaluation of the mechanical performance of asphalt concrete mixtures as indicated by the indirect tensile strength, the resilient modulus and the critical energy release rate ($J_{r}$).

3. Evaluation of the effect of the stress level and processing conditions on the fatigue behavior of AC-20 mixtures.


5. Verification of the dependency of Paris’ model parameters ($C^*$ and $m^*$) and MCL parameters ($\gamma'$ and $\beta'$) on the stress level and temperature.

6. Introduction to the total dissipated energy in fatigue failure as a new fracture criteria which counts for loading and temperature changes.

Chapter 2

Literature Review

In this Chapter, a brief literature review in the topics of asphalt additives and modified asphalt concrete mixtures performance, mechanical performance of asphalt concrete mixtures, fatigue behavior of asphalt concrete mixtures and aging of asphalt concrete mixtures is conducted. The investigation of asphalt additives includes the evaluation of both modified asphalt blends and modified asphalt concrete mixtures. The mechanical properties of asphalt concrete mixtures include the indirect tensile strength, resilient modulus and critical energy release rate. Study of the fatigue behavior of asphalt concrete mixtures usually include both the phenomenological approach which can normally consider a load control mode and a deflection control mode, and the fracture mechanics approach which will be used in this research for evaluating the fatigue fracture behavior of modified asphalt concrete mixtures. The topic of aging of asphalt concrete mixtures includes extended heating aging, pressure
oxidation aging, ultraviolet treatment aging and thermal cycling aging. The latter simulates the most severe form of environmental aging asphalt concrete mixtures are normally exposed to, and will be studied in detail in this research.

2.1 Investigation of Asphalt Additives

An asphalt cement additive can be best defined as a material which would normally be added to the asphalt cement before or during mix production, to improve the properties and performance of the resulting binder. A perfect design of an asphalt additive is a difficult task, since an additive which tends to increase mixture stability may decrease mixture flexibility or increase the probability of cracking. A perfect additive should be capable of lowering the temperature susceptibility, control age hardening and be compatible with all asphalt types.

Studying the effect of different kinds of additives on improving asphalt concrete mixture properties is a field of interest for many researchers and has led to laboratory investigations. These investigations have been carried out through two main methods. One is testing asphalt cement with and without additives to determine chemical, rheological, elastic, fracture and thermal properties as well as sensitivity to heat and oxidation and compatibility between asphalt and additives. The other method is to test asphalt concrete mixtures with and without additives to determine stability, compatibility and water
susceptibility as well as stiffness, tensile strength, fatigue resistance and creep deformation. A review of these two previous approaches follows.

2.1.1 Evaluation of Asphalt-Additive Blends

The properties of asphaltic materials for use in paving has been classified into four headings as follows; consistency, durability or resistance to weathering, rate of curing and resistance to water action. The quality of asphalt and its suitability for practical use are defined by these four properties. On this basis, the study of the effect of additives in improving these four properties is of great interest to many researchers.

In 1986, Little et al. (Little et al., 1986) conducted a study on the effect of various additives on the performance of asphalt cement. Five additives were used, namely Latex (Styrene-Butadiene Rubber), Kraton (Styrene-Butadiene-Styrene), Elvax (Ethylene Vinylacetate), Polyethylene and Carbon Black. They found that each additive demonstrated the ability to substantially alter the temperature susceptibility of asphalt cement, with the degree of alteration depending on the chemical composition of the asphalt cement. They also viewed that each additive tested showed a potential to reduce temperature susceptibility of the base asphalt. In general, they proved that each additive was successful to some degree in improving the four mentioned properties. Another research has been
conducted by the polymer products department at DuPont (DuPont, 1981) to study the possible improvement in blend properties attained by adding Elvax resin to AC-20 asphalt. Samples of AC-20 asphalt with and without Elvax resin were subjected to penetration, softening point, tensile strength, toughness and viscosity tests. They viewed the remarkable improvement attained in blend properties by adding 5% Elvax to AC-20 asphalt.

An investigation on the effect of rubber concentration and rubber particle size on the properties of asphalt cement was performed by Khedaywi et al. (Khedaywi et al., 1993). Four different rubber concentrations and three different rubber particle sizes were used. It was concluded that, penetration, ductility, flash point and specific gravity decreased as the rubber concentration in the binder increased. Also, it was found that the ductility and specific gravity of the asphalt-rubber binder decreased as the size of the rubber particles increased.

2.1.2 Evaluation of Asphalt Concrete Mixtures

In 1986, Little et al. (Little et al., 1986) also conducted a study to evaluate asphalt concrete mixtures containing additives. Beams 3" square and 15" long were prepared using modified and unmodified asphalt concrete mixtures and tested to evaluate fatigue cracking
potential using controlled stress and controlled displacement fatigue testing. In addition a healing and thermal cracking potential evaluation was conducted.

The same five additives used in the asphalt cement study, namely Latex (styrene-butadiene rubber), Kraton (styrene-butadiene-styrene), Elvax (ethylene vinylacetate), Polyethylene and Carbon Black were selected for evaluation. Based on the phenomenological regression approach for controlled stress fatigue tests mixtures containing AC-5 blends with additive exhibited superior fatigue performance as compared to mixtures which contained AC-20 with no additive. In controlled strain fatigue testing, it was found that mixtures containing AC-5 plus an additive gave better resistance to crack propagation than mixtures containing AC-20 with no additives. In general, they found that the most effective additives in reducing rutting were Elvax and Kraton for AC-5 asphalt, whereas Elvax, Kraton, Latex and Polyethylene where the most successful additives in reducing flexural fatigue cracking.

In a limited study of crack healing, they found that mixtures containing AC-5 plus an additive gave better response than those containing AC-20 with no additives. They also determined through creep testing that all additives except Latex produced equal or better performance than AC-20 mixtures at high temperature.
In 1989, Magawer et al. (Magawer et al., 1989) performed an experimental study to evaluate the effect of two deicing additives (Verglimit and PlusRid) on the resistance of asphalt concrete mixture to low-temperature cracking. It was found that the addition of Verglimit (which consists mainly of calcium chloride and a small amount of sodium hydroxide) reduced stiffness at low temperature and increased stiffness at high temperatures. However, during sample curing moisture absorption caused them to swell and crack. Therefore, pavements with Verglimit were expected to have cracking problems. On the other hand, the addition of PlusRide (which is derived from granulating whole tires and tire buffing) caused a slight decrease in stiffness at 40° F, 77° F and 104° F. However, in evaluating its effect on the resistance to permanent deformation, it was observed that, PlusRide would cause a greater rutting problem than the control mixture at high temperatures.

In 1991, Khosla (Khosla, 1991) studied the effect of two commercially available asphalt modifiers (Carbon Black and Polymerized asphalt (Styrelf)) in improving the mechanical properties as well as the fatigue life of asphalt concrete mixtures. The mechanical properties determined were creep and resilient modulus at temperatures of 0°, 40°, 70°, 100° and 140° F. Khosla concluded that using Carbon Black and Styrelf reduced the temperature susceptibility and improved the resistance to low-temperature cracking.
Also, they reduced the permanent deformation at high temperatures which reduced the potential for rutting. He also found that Styrelf was superior to Carbon Black in improving the fatigue life of the mixtures.

The effect of three different kinds of additives (Chromium Trioxide, Maleican Hydride and Furfural) when added to AC-20, was studied by Stuart (Stuart, 1993). Mechanical properties such as resistance to rutting, creep, resilient modulus, indirect tensile strength and resistance to damage were tested. On the basis of the results of that study, it was found that, the penetration and viscosity data for the three chemically modified binders indicated their higher stiffness at high temperature and lower stiffness at low temperatures, as compared with the AC-20 control asphalt. Also, it was indicated that all three chemically modified mixtures had improved low-temperature properties down to approximately 39°F. The four mixtures had equivalent test results below this temperature.

Similarly, an investigation on the effect of rubber concentration and rubber particle size on the stability of asphalt concrete mixtures was performed by Khedaywi et al. (Khedaywi et al., 1993). Four different rubber concentrations and three different rubber particle sizes were used. It was concluded that, the stability of the asphalt concrete mixtures decreased as the rubber concentration and particle size increased.
Although research mentioned above viewed the remarkable improvement in asphalt concrete mixtures when using additives, more detailed studies dealing with the effect of additive to asphalt ratio are required. Comparisons can be conducted using the more rigorous fracture mechanics approach to describe the fatigue crack propagation of unmodified and modified asphalt concrete mixtures.

Studies on the effect of Elvax and Kraton modifiers on the flexural strength and fatigue fracture behavior of AC-5 asphalt concrete mixtures were conducted by Othman (Othman, 1992). It was found that 6% of Elvax content had a higher flexural strength and a higher fracture toughness than AC-20 asphalt concrete mixtures (which is a harder asphalt than AC-5). Also, it was found that a 10% of Elvax content had the maximum ultimate strength and maximum fracture toughness compared with 6 and 15% Elvax content. Similar results were obtained for Kraton modified mixtures except that a 15% of Kraton mixture had a higher ultimate strength and fracture toughness than mixtures with 6 and 10% Kraton. An optimum Kraton content may be found beyond 15%.

2.2 Mechanical Performance of Asphalt Concrete Mixtures

Laboratory investigations on the mechanical performance of asphalt concrete mixtures have been conducted by several researchers using varying specimen geometry and
loading configurations. The Marshall stability test, which is one of the most common tests, is used in highway engineering for both mix design and evaluation. However, the Marshall method is essentially empirical and is only useful in comparing mixtures under specific conditions. Therefore a more rational procedure for asphalt concrete characterization is required to provide material properties needed for the fundamental structural analysis and the eventual design of pavements.

Because of the importance of linking mixture design to structural design and pavement performance variables, the AAMAS (asphalt-aggregate mixture analysis system) was developed by Von Quintus et al. (Von Quintus et al., 1991). According to the AAMAS procedure, severe pavement failures can be reduced by selecting asphalt, aggregate and additives consistent with the structural design and using performance-related specification for mixture control. The AAMAS procedure requires six types of tests to simulate the characteristics of mixtures placed in the field. Table 2.1 shows the laboratory tests recommended by the AAMAS study.

In the research described herein, two out of the six tests recommended by the AAMAS study, namely the indirect tensile strength and resilient modulus test, will be used in the evaluation of asphalt concrete mixtures under different loading and environmental conditions. The critical energy release rate ($J_{ic}$), which was recently adapted by many
researchers, will also be used in the evaluation of the fracture toughness of asphalt concrete mixtures. Following is a brief review of the fundamentals of each test.

Table 2.1: Tests Recommended by AAMAS Study
(Von Quintus et al., 1991)

<table>
<thead>
<tr>
<th>Test</th>
<th>Loading Procedure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Indirect Tensile Strength</td>
<td>Diametral</td>
</tr>
<tr>
<td>Resilient Modulus</td>
<td>Diametral</td>
</tr>
<tr>
<td>Indirect Tensile Creep</td>
<td>Diametral</td>
</tr>
<tr>
<td>Uniaxial Compression Resilient Modulus</td>
<td>Axial</td>
</tr>
<tr>
<td>Unconfined Compressive Strength</td>
<td>Axial</td>
</tr>
<tr>
<td>Uniaxial Creep</td>
<td>Axial</td>
</tr>
</tbody>
</table>

2.2.1 Indirect Tensile Strength

The indirect tensile test was developed for use in testing cylindrical concrete specimens through the application of a compression load along a diametrical plane through two opposite loading heads. As it was shown by Hudson et al. (Hudson et al.,
1968) this type of loading produces a relatively uniform stress acting perpendicular to the applied load plane, causing the specimen to fail by splitting along the loaded plane.

Timoshenko and Goodier (Timoshenko and Goodier, 1951) developed an expression for the stress in a circular disk with a unit thickness subjected to two equal and opposite forces acting along the diametrical plane. Frocht (Frocht, 1948) developed an expression for the stresses in a circular plate when its thickness is different from one. Both Timoshenko’s and Frocht’s expressions are based on the theory of elasticity and a point loading is assumed. However, in testing asphalt concrete mixtures theoretical conditions will never be attained. The heterogeneity of asphaltic materials affects the stress distribution. Hudson et al. (Hudson et al., 1968) showed through testing that the influence of heterogeneity of asphaltic materials was small and Frocht’s results have been considered satisfactory for use. The theory of elasticity does not hold for asphaltic materials tested at slow loading rates because of their viscoelastic behavior, however as the rate of loading increases these materials become more elastic and at very fast loading rates they can be considered completely elastic.

Based on Frocht’s expression, the stress distributions along principal axes for cylindrical specimens is shown in Figure (2.1). The stress distribution on a vertical
Figure 2.1.a: Stress Distributions on Y-Axis for Indirect Tensile Strength Test Assuming Point Loading (Hudson et al., 1968).

Figure 2.1.b: Stress Distributions on X-Axis for Indirect Tensile Strength Test Assuming Point Loading (Hudson et al., 1968).
diametrical plane is shown on Figure (2.1.a). It can be seen that, the tensile strength is constant along the y-axis. However the compressive strength goes to infinity at the top and bottom ends of the specimen due to the point loading assumption in Frocht’s theory, which is not the case for the indirect tensile strength test, in which the load is spread evenly over a 0.5 inch wide loading strip. The stress distribution on a horizontal diametrical plane is shown on Figure (2.1.b), with the stress in the horizontal direction (x-axis) being a tensile stress and the stress in the vertical direction (y-axis) being compressive. Also, it is noticed that the maximum compressive strength is three times higher than the maximum tensile strength.

Hondros (Hondros, 1971) found that the stress distribution when loading strips were used did vary from that caused by a point loading, however the stresses were the same at he center of the disk. He also found that, the compressive stress along the y-axis is approximately twice that of the tensile stress and since asphalt concrete can withstand more compressive stress than tensile stress, the specimen will fail in tension. Considering all the mentioned variations from the theory of elasticity, the expression for the maximum tensile strength can be stated as:

\[ \sigma_t = \frac{2P_{\text{max}}}{\pi DH} \]  

(2.1)
Where:

\[ \sigma_t = \text{Indirect Tensile Strength} \]
\[ P_{\text{max}} = \text{Maximum Load} \]
\[ H = \text{Thickness of Specimen} \]
\[ D = \text{Diameter of Specimen} \]

The fracture energy, which is defined as the work to be done to fracture the specimen, is equal to the area under the complete load-deflection curve, and can be expressed as,

\[
G_f = \frac{\int_0^{\delta_{\text{max}}} P(\delta) \, d\delta}{H \, D} \quad (2.2)
\]

where:

\[ G_f = \text{Fracture Energy} \]
\[ P(\delta) = \text{Load} \]
\[ \delta = \text{Vertical Deflection} \]
2.2.2 Resilient Modulus

The resilient modulus test (ASTM D4123-82) developed by Schmidt (Schmidt, 1972) has gained popularity as a means of evaluating the response of asphalt concrete mixtures to loads. It is a nondestructive test which uses a loading apparatus capable of applying a light pulsating load across the vertical diameter of a Marshall-sized specimen, whereas the deformation across the horizontal diameter caused by the load is measured by LVDT’s.

Schmidt used the same theory as for the standard indirect tensile strength test conducted under a gradually increasing load. The equation proposed by Schmidt to determine the resilient modulus is as follows:

\[
M_r = \frac{P(\nu+0.2734)}{H \delta_v} 
\]  

(2.3)

Where:

- \( M_r \) = Resilient Modulus (psi)
- \( P \) = Dynamic Load (lbs)
- \( \nu \) = Poisson’s Ratio
\[ H \quad = \text{Thickness of Specimen (in.)} \]
\[ \delta_h \quad = \text{Total Horizontal Deformation (in.)} \]

The resilient modulus can be used to evaluate the relative quality of materials as well as an input for pavement design or pavement evaluation and analysis. Because of the fact that it is a dynamic loading test, the resilient modulus test may offer a better correlation to fatigue life than the standard indirect tensile strength.

2.2.3 Critical Energy Release Rate (\(J_{1,c}\))

The \(J\)-integral proposed by Rice (Rice, 1968) is a path-independent contour integral representing a nonlinear elastic energy release rate. Rice found that the \(J\)-integral is more appropriate to evaluate the crack tip elastic-plastic field from a thermodynamics viewpoint. He mathematically formulated the energy release rate for materials displaying elastic-plastic response. Rice has also shown that the \(J\)-integral around a crack tip is the change in potential energy for a crack increment \(\Delta a\). The path independent \(J\)-integral proposed by Rice was viewed by Begley and Landes (Begley and Landes, 1972) as a measure of the crack tip elastic-plastic field and can be evaluated from the load-displacement curve associated with crack extension during monotonic loading. The critical value of the \(J\)-integral is known as the critical energy...
release rate \( (J_{ic}) \) and can be considered the limiting value of the energy release rate \( (J) \) just as the yield stress might be considered the limiting value of the applied stress. Two different methods are used to evaluate the \( J_{ic} \) (Dongre et al., 1989). In the first one, the energy release rate \( (J) \) is computed as the change in total strain energy per unit thickness as follows,

\[
J = -\frac{1}{B} \left( \frac{dU}{da} \right)
\]  

(2.4)

Where:

\( U \) = Total absorbed energy (area under the load/load point deflection curve)

\( B \) = Specimen thickness

\( a \) = Crack length

The value of the critical energy release rate \( (J_{ic}) \) is obtained by plotting the calculated values of \( J \) against the corresponding values of \( a \). This method requires testing multiple specimens with different crack lengths. Rice et al. (Rice et al., 1973) proposed a second method to evaluate \( J_{ic} \) with the advantage of requiring only one specimen with one crack length. This method has been explained in detail by Sumpter and Turner (Sumpter and Turner, 1976), where \( J_{ic} \) is calculated by;
\[ J_{te} = \frac{\mu U}{B(D-a)} \]  \hspace{1cm} (2.5)

Where:

\[ \mu \] = Constant

\[ D \] = Beam depth.

For single edge notched bending specimens, \( \mu \) is equal to 2 (Sumpter and Turner, 1976) for notch to depth ratios between 0.5 and 0.7. It has been shown by Dongre et al (Dongre et al, 1989) that the second method described above correlates very well with the values obtained from method one and gives more consistent results than method one due to the fact that it eliminates the statistical variation of different specimens. These workers also showed that \( J_{te} \) is sensitive to asphalt concrete properties especially at low temperatures (below 60° F), while \( K_{te} \), the classical stress intensity factor, is not. The concept of \( J_{te} \) as a fracture criteria for pavements has been recommended by other researchers (Little et al., 1985).
2.3 Fatigue of Asphalt Concrete Mixtures

The available literature on the topic of fatigue of asphalt concrete mixtures can be best classified into two categories; one endorsing the phenomenological approach and the other which follows the fracture mechanics approach. The largest body of literature on the subject deals with the phenomenological approach or the S-N approach which correlates the number of cycles to failure $N_f$ to the applied stress ($\sigma$) or the strain ($\varepsilon$) through empirical constants. Both approaches will be presented next.

2.3.1 Phenomenological Approach

Extensive laboratory fatigue studies on asphalt concrete mixtures have been carried out throughout the world. The phenomenological approach to fatigue life of pavements is based on the endurance concept using Wöhler's technique (Wöhler, 1871). A test specimen is said to be subjected to a simple loading if the loading condition remains unchanged during the fracture life. If the loading condition changes during the fracture life it is considered to be subjected to compound loading. These two kinds of loading may involve changes in the environment, such as temperature.

The term used to describe how stress and strain levels are permitted to vary during fatigue loading is called mode of loading. Usually there are two different modes of loading,
the controlled load mode and the controlled deflection mode. Both modes of loading will be discussed next.

2.3.1.1 Controlled Load Mode

In the controlled load mode the nominal stress level or load is maintained constant during the fracture life. The load control mode is shown in Figure 2.2 for simple loading. It can be seen that, the strain, \( \varepsilon \) increases with increasing number of repetitions until failure occurs.

The familiar fatigue diagram can be obtained by performing simple repeated loading tests in the controlled stress mode at different stress levels and determining the corresponding fracture lives. This is shown in Figure 2.3 as a straight line on a log-log graph. The relationship between stress \( \sigma \) and fracture life \( N_f \) is expressed as:

\[
N_f = K \left( \frac{1}{\sigma} \right)^n
\]  

(2.6)

Where:

\( N_f \) = Number of load applications to fracture
Figure 2.2: Fatigue Behavior of Asphalt Paving Materials for Controlled load Mode.

Figure 2.3: Fatigue Diagram for Controlled Load Mode.
$K_I = \text{Constant depending on the mixture characteristics}$

$\sigma = \text{Applied stress}$

$n_I = \text{Regression constant (slope of the curve)}$

2.3.1.2 Controlled Deflection Mode

In the controlled deflection mode the nominal strain level is maintained constant through fracture life. The controlled deflection mode is shown in Figure 2.4 for simple loading. In the controlled strain mode the stress or load will gradually decrease with increasing number of repetitions since the specimen is gradually damaged, with less load required to produce the same deformation.

As in the controlled load mode, the familiar fatigue diagram can be obtained by performing simple loading tests in the controlled strain mode and determining the corresponding lives. This is shown in Figure 2.5 as a straight line on a log-log plot. Thus the relationship between strain $\varepsilon$ and fracture life $N_f$ is expressed as:

$$N_f = K_2 \left( \frac{1}{\varepsilon} \right)^{n_I} \quad (2.7)$$
Figure 2.4: Fatigue Behavior of Asphalt Paving Materials for Controlled Deflection Mode.

Figure 2.5: Fatigue Diagram for Controlled Deflection Mode.
where:

\( N_f \) = Number of load repetitions to fracture

\( K_2 \) = Constant which depends on the mixture characteristics

\( \varepsilon \) = Applied strain

\( n_2 \) = Regression constant (slope of the curve)

A comparison can be obtained between both modes of loading by examining the stress versus service life relationships determined in both types of tests. The relationships shown in Figures 2.3 and 2.5 can be assumed to represent the results of tests on identical specimens and that the conditions of stress and strain are chosen so that identical initial values (termed \( \sigma_0 \) and \( \varepsilon_0 \)) are obtained. The initial stress versus service life for the controlled stress tests can be plotted to get a comparison with the controlled strain tests, as shown in Figure 2.6.

Due to its simplicity, this approach has been widely adopted. However, it carries severe limitations. It does not account for the number of cycles to failure for both crack initiation and propagation, which is important from a structural design viewpoint. This view has been shared by Majidzadeh et al. (Majidzadeh et al., 1976). Moreover, the constants \( K_1, K_2, n_1 \) and \( n_2 \) are indeed regression constants and are influenced by many variables such as the type and rate of loading. Therefore, they are not material constants. This view is also
Figure 2.6: Fatigue Diagram Comparison for Controlled Load and Controlled Deflection Tests.
shared by Ramsamooj (Ramsamooj, 1970), who indicated that since these constants are dependent on the boundary conditions and the type of tests performed, they can not be viewed as material parameters.

2.3.2 Fracture Mechanics Approach

The fracture mechanics approach adopted by many researches considers fatigue as a process of damage and utilizes fracture mechanics principles to investigate cracking of paving mixtures. The stress intensity factor, $K$, defines the magnitude of the local stresses around the crack tip. Fatigue crack propagation experiments have been conducted by various laboratories to correlate $K$ with the crack speed $da/dN$ in the equation proposed by Paris et al. (Paris et al., 1963). In this equation;

$$\frac{da}{dN} = C (\Delta K)^m$$  \hspace{1cm} (2.8)

Where:

$\frac{da}{dN}$ = Crack Growth Rate

$\Delta K$ = Stress Intensity Range ($K_{\text{max}} - K_{\text{min}}$)

$C, m$ = Material Constants.
One of the objectives of this research is to study Paris' equation and verify if $C$ and $m$ are in effect material constants. Therefore Paris' equation will be discussed in detail in Chapter 4. Using Equation 2.8, Majidzadeh et al. (Majidzadeh et al., 1972) stated that at low temperature for sand-asphalt and asphalt concrete beams $C$ in Paris' equation becomes a material constant. However, at high temperature $C$ and $m$ can no longer be considered as material constants.

Schapery (Schapery, 1975) developed a theoretical analysis to link material properties such as the creep compliance, tensile strength and fracture energy to determine $C$ and $m$ in Paris' equation for viscoelastic materials. This theoretical relationship has been examined using asphalt concrete mixtures by several researchers including Little et al. (Little et al., 1986) and Germann and Lytton (Germann and Lytton, 1979) Little found that the crack speeds of various asphalt concrete specimens at two different crack lengths (one inch and two inches) are identical when they were calculated from either the viscoelastic analysis or the linear elastic approach. He concluded that both analyses yielded the same results when using $C$ and $m$ from either approach to determine the crack speed. Germann and Lytton (Germann and Lytton, 1979) found that the calculated values of $C$ and $m$ agree fairly well with those determined experimentally for high asphalt content samples. At lower asphalt content, the theoretical and experimental values differ significantly. It has been
stated (Aglan and Figueroa, 1991) that although Schapery's analysis may be applicable in some instances, it has not been widely accepted. Even if the constants \( C \) and \( m \) can be related to some material properties as Schapery suggested they are still used in a power law relationship which at best can only describe a linear region of fatigue crack propagation, i.e. it will not describe fatigue crack propagation over the entire range of the crack driving force.

From the previous discussion, it can be seen that both the phenomenological and the fracture mechanics based approaches have been treated in a rather empirical manner using regression constants which cannot in most cases be considered as material constants. One of the objectives of the present work is to identify, verify and determine parameters responsible for a mixture's resistance to crack propagation using the modified Paris' equation with the energy release rate (\( J \)) instead of the stress intensity factor (\( K \)) and the Modified Crack Layer model. Both Paris' and the Modified Crack Layer models will be discussed in detail in Chapter 4.

2.4 Thermal Cycling Aging of Asphalt Concrete Mixtures

Asphalt concrete by far is the most common paving material in use for public roads. Although it is a versatile material, the physical properties of asphalt may limit its
usefulness and performance. Among these properties is the durability which is greatly influenced by the environmental changes during the year especially between summer and winter and between day and night, when the daily average temperature change can be considerably large. During the summer, high temperature can soften the asphalt binder and consequently reduce the stiffness of the paving mixture. On the other hand, in the winter low temperature can stiffen the asphalt binder and reduce the flexibility of the paving mixture. As a result, thermal cracking of the pavement surface may develop which adversely affects the performance of the paving mixture. Thus, high temperature stiffness and low temperature flexibility are important properties which increase the life of asphalt concrete pavements. Polymer modified asphalt is proposed in an attempt to improve the low as well as the temperature properties of asphalt concrete mixtures.

One of the critical factors that should be considered for better polymer modified asphalt is the air void percentage in the total mix. The performance of a polymer modified asphalt mixture will be improved as this percentage is reduced (Narusch, 1982), (Little, 1991). In general, the air void percentage depends on the load capacity of the transportation facility being designed. Lower air void percentage can be obtained by increasing both the modifier and the asphalt binder content until the required value is reached (Narusch, 1982). Investigation into the effect of asphalt additives on pavement
performance (Little et al., 1986) has revealed that in general all additives improved their temperature susceptibility. Under stress control fatigue, using the phenomenological approach, these workers concluded that Kraton was one of the top additives among five tested at 0° and 68°F. This has also been confirmed using the Modified Crack Layer model (Agam and Figueroa, 1991). However when the mixtures containing additives were aged at 140°F for 7 days, their fatigue life decreased considerably in comparison to their unaged counterparts (Little et al., 1986). Under controlled displacement fatigue, using Paris’ model, Little et al. also concluded that Kraton additives were considerably superior among those additives tested at 33°F. At 77°F crack branching was observed, which tends to redistribute the stress causing main crack growth retardation.

The phenomenon of aging of asphalt cement has been acknowledged for a long time and extensive research has been conducted on the effect of long term aging on asphalt cement. However, little work has been done on asphalt mixtures and so far no standard test methods exist (Bell, 1989). In general, studies of aging of pavements are performed through three main methods. These are, extended heating, pressure oxidation and ultraviolet light treatment. Aging by extended heating (Hugo et al., 1985), (Kemp et al., 1981), (Plancher et al., 1976) involves an oven aging program by which pavement samples are heated to a specific temperature for a specific period of time. Then standard tests such
as the resilient modulus and indirect tensile tests are performed to evaluate the effect of the laboratory aging. Pressure oxidation aging, studied by several workers (Von Quintus et al., 1991), (Kumar et al., 1977), (Kim et al., 1986) involves exposing heated samples of asphalt concrete to air or oxygen under pressure for a certain period of time. At the conclusion of the experiments, standard tests such as creep, resilient modulus, fatigue etc. are used to evaluate the effect of pressure oxidation aging on pavement. Aging by ultraviolet treatment (Hveem et al., 1963), (Traxler, 1961) involves subjecting samples of pavement to ultraviolet radiation for a specific period of time and then performing standard tests similar to those for the extended heating and oxidative aging. Combined aging with UV and forced draft oven heating has been suggested by Tia et al. (Tia et al., 1988), noting that the effect of UV aging occurs at the surface although it does affect a significant depth of the asphalt concrete.

As it can be seen from the literature, all aging methods which have so far been used involve mainly heating the asphalt concrete. This is done to harden them via volatilization and oxidation. The main drawback of these methods is the softening of the pavement samples during aging. This can lead to a loss in their mechanical stability. This view is supported by Kim et al. (Kim et al., 1986) who suggested that maintaining the integrity of a sample subjected to these aging methods presents a problem. Bell (Bell, 1989) has stated
that in some aging programs specimen confinement may be necessary to prevent collapse. It has been concluded by the same researcher that none of the aging methods used so far are clearly superior and the possibility exists for new methods.

As far as it is known, very little work has been done on aging using thermal cycling. This is of critical importance since it simulates the actual environmental aging cycles which asphalt mixtures are normally exposed to. Thermal cycling is known to cause transverse cracking in asphalt pavements which considerably shortens their life, particularly at low temperature. This is caused by stresses due to shrinkage which become greater than the tensile strength of the material (Dongre et al., 1989). Thus it appears to be logical that the environmental aging of the pavements by low temperature thermal cycling can be used to test the assumption that polymer modifiers improve the performance of asphalt concrete pavements.

In the current study focus is placed on the evaluation of AC-20 and Styrene-Butadiene-Styrene (SBS) modified AC-5 mixtures. This evaluation will involve the effect of stress level and processing conditions in the fatigue behavior of the AC-20 mixture. Also, the effect of SBS content, temperature and thermal cycling aging in a low temperature regime on the mechanical and fatigue performance of Kraton modified AC-5 mixtures will be studied. The performance of the asphalt concrete mixture will be evaluated
based on its resistance to fatigue crack propagation in view of Paris’ as well as the Modified Crack Layer models, and the comparison between the fatigue performance of the mixtures as it relates to conventional properties such as indirect tensile strength and resilient modulus.
Chapter 3

Laboratory Testing

The conducted experimental work is presented in this chapter, beginning with a brief description of the testing machine which was developed in order to perform fatigue crack propagation tests, continuing through the sample fabrication process for indirect tensile strength test, resilient modulus, critical energy release rate \( (J_{lc}) \) and fatigue crack propagation tests, ending with a brief description for each test. Finally, a description of the Scanning Electron Microscopy analysis is presented.

3.1 Fatigue Crack Propagation Testing Machine

A repeated load pneumatic flexure testing machine was developed to perform controlled stress flexure fatigue tests on asphalt concrete beams. Symmetrical four point loading produces a constant bending moment over the middle third of a 15" long beam specimen with 3.5" in depth and 2" in width. The repeated flexure apparatus allows the rotation and translation of the beam ends.
The electro-pneumatic system applied repeated loads in the form of haversine waves. The load duration was 0.2 sec. followed by a 2 sec. rest period between repeated loads. This loading condition was selected to agree closely with those used for previous flexure fatigue studies. A limiting switch on the machine automatically stopped the loading and the system load counter after beam failure.

The machine is provided with a 20 gallon air tank to maintain a constant pressure through the piston operating system, which is connected to a 1000 lb. capacity load cell attached to the 2 point loading moving head. The dynamic beam deflection was measured with a linear variable differential transducer (LVDT). The hysteresis loop resulting from the loading and unloading of the beam was recorded at 0.25" intervals of crack growth by means of an X-Y plotter.

3.2 Sample Fabrication

Two types of samples were prepared: Marshall-sized samples (cylindrical in shape used to determine the indirect tensile strength and resilient modulus for each mixture) and rectangular cross section beams to be tested for critical energy release rate and fatigue crack propagation tests.

3.2.1 Preparation of Marshall Sized Samples

The Marshall method of mix design for determining the optimum asphalt content of an asphalt concrete mixture uses standard test samples 2.5" in height by 4"
in diameter, prepared according to a specified procedure of heating, mixing and compaction of the asphalt-aggregate mixtures (*ASTM Designation: D 1559-89*). To provide adequate data three test samples were prepared for each asphalt content used. Thus an asphalt hot mix design study using seven different asphalt contents requires a total of twenty one test specimens. All specimens follow the gradation requirements corresponding to ODOT 403 specifications as shown in Table 3.1

Table 3.1: Aggregate Gradation

<table>
<thead>
<tr>
<th>Aggregate Gradation Sieve Size</th>
<th>Percentage of Total Aggregate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2&quot; - 3/8&quot;</td>
<td>5%</td>
</tr>
<tr>
<td>3/8&quot; - #4</td>
<td>36%</td>
</tr>
<tr>
<td>#4 - #16</td>
<td>31%</td>
</tr>
<tr>
<td>#16 - #50</td>
<td>19%</td>
</tr>
<tr>
<td>#50 - #200</td>
<td>9%</td>
</tr>
<tr>
<td>Passing #200</td>
<td>0%</td>
</tr>
</tbody>
</table>

Following is a description of the sample preparation procedure.

3.2.1.2 Preparation of Batch Mixes

The weight of each sample batch including both aggregate and asphalt was computed by considering a target unit weight of 149 pcf. Knowing the specimen
volume, the total weight for each sample is determined to be 1229.22 gm. All sized fractions of aggregate were placed in separate pans and heated to a temperature ranging between 300 and 375° F. The compaction mold with an inside diameter of 4" and a height of 3", as well as the base of the compaction hammer which has a flat circular face with a 3 7/8" diameter were also preheated to over 200° F using a bath of boiling water as a convenient method. The heated aggregate is then blended with the required amount of asphalt cement by means of a mixing tool and as quickly and thoroughly as possible to yield a mixture having a uniform distribution of asphalt cement.

3.2.1.2 Compaction

The equipment used for compaction includes the Marshall compaction hammer with a 10 lb. weight and an 18" drop, with a 3 7/8" diameter circular compaction base. The heated mold, placed on a compaction pedestal, was filled with the heated asphalt-aggregate mixture and then both the top and bottom of each sample were compacted by applying fifty blows with the standard Marshall compaction hammer. The mold containing the compacted sample was placed in cool water for 2 minutes, before extruding the sample with a hydraulic jack set up. Extruded samples were placed on a smooth leveled surface before testing.
3.2.2 Preparation of Asphalt Concrete Beams

Three types of asphalt concrete beams were prepared during this research:

- Statically compacted beams: asphalt concrete beams were only subjected to static compaction.
- Dynamically and Statically compacted beams: asphalt concrete beam were subjected to both dynamic and static compaction.
- Statically compacted modified beams: asphalt concrete beams containing asphalt modifiers were only subjected to static compaction.

3.2.2.1 Preparation of Batch Mixes

The required percentages of aggregate were mixed in one batch to produce asphalt concrete beams also with a target unit weight of 149 pcf, as in the Marshall-sized specimens. Gradation requirements also met ODOT 403 specifications shown in Table (3.1). All beams contained 8% of asphalt cement or asphalt cement-modified blends, as a percentage of the total mix. This percentage corresponds to the optimum asphalt cement content, as determined by the Marshall method of mix design, as will be presented later.

3.2.2.2 Static Compaction Procedure

The aggregate mixes and the asphalt cement were heated to a temperature ranging between 300°F and 375°F, along with the compaction mold and the mixing tools. The aggregate was then blended with the required amount of asphalt cement (8% of the total
weight of mix) as quickly and thoroughly as possible to yield a mixture having a uniform
distribution of asphalt cement. The heated mold was then filled with the heated asphalt
cement-aggregate mixture, followed by its static compaction by applying a uniform
pressure of 30 psi through a 2"x15" plate with a hydraulic compression testing machine for
5 minutes. The compacted beam was allowed to cool off in the mold for a few hours,
before its removal, and was usually tested 7 days after preparation and curing.

3.2.2.3 Dynamic Compaction Procedure

The equipment required for the dynamic-static compaction includes a modified
Marshall compaction hammer having a 10 lb. weight and an 18" drop. The compaction
face was modified to include a 6"x2" rectangular plate, instead of the usual 3 7/8" in
diameter circular tamping base. The base of the compaction hammer was also preheated
using a bath of boiling water as a convenient method. The heated mold placed on a
compaction pedestal was filled with the heated asphalt-aggregate mix to about half of the
depth of the mold, and then a total of 30 blows were applied at three locations along the
length of the beam (10 blows per location). The remaining heated asphalt-aggregate mix
was added to the mold as in the first stage, and another series of 30 blows were applied.
Following the dynamic compaction procedure, the beam was subjected to a pressure of 30
psi in a hydraulic compression testing machine for 5 minutes. The compacted beam was
then allowed to cool off in the mold for a few hours, before its removal, and was usually
tested 7 days after preparation and curing.
3.2.2.4 Preparation of Polymer-Asphalt Blends

Most grades of polymers can be readily blended with asphalt cement using a conventional low shear mixer at temperatures ranging between 275-300° F. However, the blending process is improved if the temperature is maintained between 325 and 350 °F. The preferred blending sequence for the modified asphalt concrete mixtures is as follows:

1. Asphalt cement is heated alone to 300° F.
2. The required amount of polymer is added to the heated asphalt cement to produce the required asphalt cement-polymer blend.
3. The blend is maintained hot to a goal temperature ranging between 300 and 350° F for at least 2 hours.
4. The blend is thoroughly mixed by means of a low shear mechanical mixer for at least 15 minutes to obtain a more homogenous blend.
5. Finally, the blend is kept at a temperature of 300° F and ready for use.

3.3 Thermal Cyclic Aging Program

In the current study, evaluation of SBS modified AC-5 mixtures under a thermal cycling aging program is considered. Beams were thermally cycled between 70° F and 0° F in an environmental chamber. The temperature-time profile used in this study is shown in Figure 3.1. This profile was established, initially by embedding a thermocouple in the middle of a beam. The rate of heating and cooling of the asphalt
Figure 3.1: Temperature-Time Profile for Thermal Cyclic Program
mixture was established as shown in Figure 3.1. A temperature plateau was maintained at the lower 0° F temperature level (12 hrs). The length of each cycle was taken as one day. Four levels of thermal cycles were used. These were 7, 14, 21 and 28 cycles. At least three specimens were tested at each level of thermal cycling used in the corresponding fatigue study. Three specimens were also fatigued without thermal cycling, for comparison. Similar experiments were conducted on Marshall sized cylindrical specimens to study the effect of thermal cycling on the indirect tensile strength and the resilient modulus of the SBS modified mixture.

3.4 Specimen Testing

The specimen testing conducted during this research included four different types of tests:

1. Indirect tensile strength test to determine the tensile strength for asphalt concrete mixtures.

2. Resilient modulus test to determine the resilient modulus for asphalt concrete mixtures.

3. Critical energy release rate tests to determine ($J_{ic}$) for asphalt concrete mixtures.

4. Fatigue tests in order to determine the resistance of the asphalt concrete mixtures to fatigue crack propagation.
3.4.1 Indirect Tensile Strength Tests

A mechanical drive testing frame outfitted with an electronic load cell and an LVDT was used to conduct the indirect tensile tests to evaluate the tensile strength of asphalt concrete mixtures. Test specimens 2.5 inches thick and 4 inches diameter were compacted and then tested using curved steel loading strips 0.5 in. wide. The load was applied at a vertical deformation rate of 4 mm/min. and the horizontal deformation was measured with two LVDT’s. Data was recorded on an X-Y plotter. Software was developed to digitize graphical data and to calculate the fracture energy defined as the area under the load-deflection curves over the thickness and the diameter of the cylinder.

3.4.2 Resilient Modulus Tests

The resilient modulus test was conducted in substantial accordance with ASTM D4123. This test was performed using the fatigue testing machine described previously on a 2.5 in. thick x 4 in. diameter specimens. The resilient modulus of the specimens was obtained by applying a dynamic diametral load of 50 lb. through a 0.5 in. wide curved steel loading strip with a curvature equal to that of the specimen. The load was applied at a rate of 1 cycle per second using a haversine wave function. The horizontal deformations were measured with two linear variable displacement transducers one at each side of the specimen. A chart recorder was used to record the loading function and the horizontal deformations after a steady state was reached.
3.4.3 Critical Energy Release Rate Tests

The critical energy release rate tests were conducted with the guide of ASTM E813-81 (standard for $J_{ic}$ testing for metals), since no standard for $J_{ic}$ testing for asphalt concrete exists. Notched asphalt concrete beams 3.5 in. deep, 2 in. wide were used and 10 in. long between supports as shown in Figure 3.2.a The notch depth to beam depth ratio was chosen to be 0.5 in order to limit extensive plastic flow in the unfractured ligament in the notched beams. As indicated before, tests were conducted in an environmental chamber at the required test temperature.

A screw driven static flexure testing machine with a 5000 lb. load cell was used to conduct $J_{ic}$ tests on asphalt concrete beams subjected to symmetrical four point loading. This load configuration produces an increasing static pure bending moment over the middle third of the 15 in. long beams. The machine is provided with an upper fixed head and a lower platen with two end supports 10 inch apart. The beam deflection is measured with an LVDT and the load-deflection curve is recorded by means of an X-Y plotter. After setting the beams on the supports and fitting the loading head on the top surface of the beam, the load is applied gradually at a constant rate until failure is reached.

3.4.4 Fatigue Crack Propagation Tests

Four point bending fatigue crack propagation tests were performed using the repeated pneumatic flexure testing machine previously described, fitted with a 1000 lb.
Figure 3.2.a: Notched Specimen for Critical Energy Release Rate ($J_e$) Tests.

Figure 3.2.b: Notched Specimen for Fatigue Crack Growth Tests.
load cell. Tests were conducted at a constant temperature of 70°F using an invert haversine wave type of load. The load application period was 0.2 sec. followed by a 2 sec. rest period between repeated loads. As in the static flexural test, the support span was equal to 10" and the midspan loading points were separated by 3 1/3".

An initial straight notch 0.25" deep and with dimensions shown in Figure 3.2.b, was saw-cut at midspan and at the bottom face of all beams to initiate cracking around the notch. A maximum load of 65 lb. corresponding to 30% of the ultimate load for the AC-5 mixture at 70°F, was used with continuous cycle load applications from zero to the maximum load. A hysteresis loop (load vs. deformation) was recorded at 0.25" intervals of crack growth using the X-Y plotter.

Software was developed to digitize graphical data and to calculate pertinent areas within the load-deflection curves obtained during fatigue testing. A computer program -GRAB- (written in FORTRAN) was used to digitize graphical data recorded by the X-Y plotter, whereas another computer program -AREA 2- written in BASIC language, was used to calculate the area of the hysteresis loop. A third program MIN (written in BASIC language) was also developed to shift digitized data to the origin of the coordinate system. Listings of all programs are included in Appendix A.

3.5 Scanning Electron Microscopy

Scanning electron microscopy (SEM) analysis on the effect of additive modifier percentage was performed. SEM samples were taken from fractured beams of
SBS modified AC-5 mixtures at various additive percentages. These were compared with SEM samples taken from fractured beams from unmodified AC-5 mixtures. Focus was placed on binder rich areas as well as aggregate surfaces.
Chapter 4

Theoretical Approach

In this chapter Paris' model is reviewed along with the Modified Crack Layer model which are used in this research to analyze fatigue crack propagation data for polymer modified asphalt concrete mixtures. Paris' model basically correlates the stress intensity factor or the energy release rate with the crack speed $da/dN$. The Modified Crack Layer model is essentially developed from the crack layer theory assuming a linear relationship between the crack length and the width of the active zone.

4.1. Paris' Model

In the early 1960s, Paris et al (Paris et al, 1963) demonstrated the use of the fracture mechanics approach for characterizing the resistance of a material to fatigue crack propagation. In this section, various fatigue crack propagation studies using different functions of applied stress or strain are reviewed.
4.1.1 Stress Intensity Factor Concept

There are generally three modes of loading which involve different crack surface displacements, these modes are:

**Mode I:** Opening or tensile mode (the crack faces are pulled apart)

**Mode II:** Sliding or in plane shear (the crack surfaces slide over each other)

**Mode III:** Tearing or anti-plane shear (the crack surfaces move parallel to the leading edge of the crack and relative to each other)

The stress intensity factor, $K$, defines the magnitude of the local stresses around the crack tip, which depends on loading, crack size, crack shape and geometric boundaries with the general form given (for *Mode I*) by:

$$K_1 = f(g) \sigma \sqrt{\pi a}$$  \hspace{1cm} (4.1)

Where:

$K_1$ = Stress intensity factor for *Mode I*

$\sigma$ = Stress applied to component

$a$ = Crack length

$f(g)$ = Correction factor that depends on specimen and crack geometry

Stress intensity factor solutions have been obtained for a wide variety of problems and published in stress analysis handbooks (Murkami, 1987 and Tada, 1985). The stress
intensity factor range \((\Delta K = K_{\text{max}} - K_{\text{min}})\) is usually considered in fatigue crack propagation studies. As the stress intensity factor reaches a critical value, \(K_c\), unstable fracture occurs. The critical value of the stress intensity factor is known as the fracture toughness of the material which can be considered the limiting value of stress intensity just as the yield stress might be considered the limiting value of applied stress. Due to its simplicity, \(K_c\) is widely used to characterize the resistance of materials to crack propagation (fracture toughness).

### 4.1.2 Energy Release Rate

In 1920, Griffith et al. (Griffith et al., 1920) used the first law of thermodynamics to develop a fracture theory for brittle materials. For a plate with a central transverse crack of \((2a)\) where \(a\) is much larger than the plate thickness, subjected to a uniform tensile stress \(\sigma\) in plane stress condition, Griffith showed that \(\gamma\), which is the surface energy of the material, can be written as a function of the applied stress and the crack length as,

\[
2\gamma_s = \frac{\pi \sigma^2 a}{E}
\] (4.2)

Where:

- \(\gamma_s\) = Surface energy
- \(\sigma\) = Applied stress
- \(a\) = One-half of the crack length
Griffith model is only limited to linear elastic materials with a very small plastic zone near the crack tip. In 1948, Irwin (Irwin, 1948) proposed an energy approach which is more convenient for solving engineering problems. Irwin defined the energy release rate \( G \) as the available energy for an increment of crack extension and can be expressed as follows,

\[
G = -\frac{dp}{dA} = \frac{\pi \sigma' a}{E} \tag{4.3}
\]

Where:

- \( G \) = Energy release rate
- \( p \) = Potential energy
- \( A \) = Crack area

It is also found that \( G \) is related to the stress intensity factor for plane stress condition as,

\[
G = \frac{K'^2}{E} \tag{4.4}
\]

and for plane strain condition as,
\[ G = \frac{K^2(1-\nu^2)}{E} \]  

(4.5)

where \( \nu \) is Poisson's ratio.

It is seen that the energy criterion is derived for linear elastic materials and carries the same limitations as the stress intensity factor, \( K \), or the stress intensity range, \( \Delta K \).

4.1.3 J-integral Approach

The J-integral proposed by Rice (Rice, 1968) can be used to evaluate the crack tip elastic-plastic field. Invoking the deformation theory of plasticity, Rice formulated mathematically the energy release rate for materials displaying elastic-plastic response. For a cracked body subjected to a two dimensional deformation field, the J-integral is expressed as:

\[ J = \int_{\Gamma} \left( w \frac{\partial y}{\partial x} - T \frac{\partial u}{\partial x} \right) ds \]  

(4.6)

Where \( \Gamma \) is a closed contour surrounding a crack tip in a stressed solid, \( w \) is the strain energy density \( w = w(x,y) = \int \sigma_{\eta} \, d\varepsilon_{\eta} \), \( T \) is the traction vector perpendicular to \( \Gamma \), \( u \) is the incremental displacement in the x direction and \( ds \) is an element of \( \Gamma \). Rice has also showed that the J-integral around a crack tip is the change in potential energy for a crack increment \( da \). The path independent J-integral proposed by Rice was viewed by
Begley and Landes (Begley and Landes 1972) as a measure of the crack tip elastic-plastic field and can be evaluated from the load displacement curve associated with crack extension during monotonic loading. They tested a series of specimens with different crack depths to evaluate $J$. A relationship between the energy absorbed by the material $U$ (the area under the load-displacement curve and the initial crack length was plotted. Based on the definition of the energy release rate, $J$ was calculated using the compliance relationship;

$$J = -\frac{1}{B} \left( \frac{dU}{da} \right)$$  \hspace{1cm} (4.7)

Where:

$U$ = Total absorbed energy (area under the load/load point deflection curve)

$B$ = Specimen Thickness

$a$ = Crack length

4.1.4 Fatigue Crack Growth Equations

A typical fatigue crack propagation behavior can be divided into three distinct regions as shown in Figure 4.1. Region I cracking behavior is associated with a threshold value as a function of the applied load below which there is no propagation of the crack. However, in region I the stress intensity factor is small enough so that no crack growth is
Figure 4.1: Three Regions of Crack Growth Rate Curve.
considered. In the mid-region, region II, the curve is essentially linear on log-log paper and is represented by Paris' and Erdogan's power law which is presented next. Finally, in region III crack growth rates are extremely high and the stress intensity factor approaches the critical stress intensity factor and unstable crack growth occurs.

Paris et al. correlated the $K$ or $\Delta K$ with the crack speed $da/dN$ using the following equation;

$$\frac{da}{dN} = C \Delta K^m$$  \hspace{1cm} (4.8)

Where $C$ and $m$ are material constants and $\Delta K$ is the stress intensity range. Equation 4.8 is widely known as Paris' Law. The inadequacy of Paris' equation to predict fatigue crack propagation rates, particularly at both low and high values of the function of the applied load has led to the development of other fatigue models. In 1967, Forman et al. (Forman et al., 1967) developed the following relationship which is applicable for regions II and III;

$$\frac{da}{dN} = \frac{C_1 \Delta K^{m_1} - 1}{K_{\text{crit}} - K_{\text{max}}}$$  \hspace{1cm} (4.9)

Where:

$C_1$ and $m_1$ are material constants

$K_{\text{crit}}$ = Critical value of stress intensity factor
\( K_{\text{max}} \) = Stress intensity factor corresponding to \( \sigma_{\text{max}} \)

It should be noted that, the crack growth rate becomes infinite as \( K_{\text{max}} \) approaches \( K_{\text{crit}} \), and the constants \( C_i \) and \( m_i \) in Forman's equation do not have the same values or units as in Paris' equation.

An alternative semi-empirical equation for Regions II and III was developed by Weertman (Weertman, 1966), as follows;

\[
\frac{da}{dN} = \frac{C_2 \Delta K^{m_2}}{K_{\text{crit}}^2 - K_{\text{max}}^2} \tag{4.10}
\]

Equations (4.9) and (4.10) predict fatigue crack growth at both regions 2 and 3, but they do not predict the threshold region. In 1972, Klesnil (Klesnil and Lukas, 1972) modified Equation (4.8) to account for the threshold at region I;

\[
\frac{da}{dN} = C_3 \left( \Delta K^{m_3} - \Delta K_{\text{th}}^{m_3} \right) \tag{4.11}
\]

Where:

\( K_{\text{th}} \) = Stress intensity factor at crack initiation

McEvily (McEvily, 1988) proposed an equation based on a simple physical model to describe the entire crack growth curve. This equation is expressed as;
\[ \frac{da}{dN} = C_4 \left( \Delta K - \Delta K_{th} \right)^2 \left( 1 + \frac{\Delta K}{K_{crit} - K_{max}} \right) \] (4.12)

It can be seen that, all Equations from 4.8 to 4.12 can be integrated to find the fatigue life. Also, to be noted that all the preceding equations assume elastic similitude of the growing crack, therefore they are only valid for a constant amplitude of loading. The path independent \( J\)-integral proposed by Rice was viewed by Dowling and Begley (Dowling and Begley, 1976) as a measure of the crack tip elastic-plastic field and applied to correlate fatigue crack growth with \( J \) under large scale yielding conditions where \( K \) is no longer valid, using the following equation:

\[ \frac{da}{dN} = C^* \Delta J^{m^*} \] (4.13)

Where \( C^* \) and \( m^* \) are material constants and \( \Delta J \) is energy release rate range. Equation 4.13 can be rewritten in a logarithmic form as follows:

\[ \log \frac{da}{dN} = \log C^* + m^* \log \Delta J \] (4.14)

Equation 4.14 can be plotted as a straight line with \( \log C^* \) being the intercept and \( m^* \) the slope.
4.1.5 Crack Speed Index

Pickett and Lytton (Pickett and Lytton, 1983) suggested a combined form of parameters $C^*$ and $m^*$ in Paris law. They proposed the crack speed index which accounts for the effects of both parameters ($C^*$ and $m^*$) in fatigue-fracture behavior. In this approach the crack speed index is defined as:

$$\text{Crack Speed index} = m^* + \log C^* \quad (4.15)$$

Which is considered as a measure of resistance to crack growth. As the crack speed index decreases, the resistance of the material to crack propagation increases.

By conducting a fatigue crack propagation test, Equation 4.14 can be used to evaluate $C^*$ and $m^*$ for asphalt concrete mixtures. The slope of the crack length vs number of cycles curve gives the crack speed $da/dN$ at a certain crack length. The energy release rate ($J$) can be determined by using Equation 4.7.

It can be seen that $K$, $\Delta K$, $J$ and $\Delta J$ have so far been used as correlative tools in a formula based on Paris' equation. Also, it is obvious that, all of the fatigue crack growth laws, which are considered empirical or semi-empirical equations, use regression constants which can not in some cases be considered material constants. However, one of the objectives of the present work is to identify and determine the parameters $C^*$ and $m^*$ in Equation (4.14), which are considered parameters responsible for a mixture's resistance to
crack propagation. Also, to verify if $C^*$ and $m^*$ are real material parameters under different load and environmental conditions. The crack speed index will be also used as a comparative tool for evaluation of the resistance of asphalt concrete mixtures to fatigue crack propagation.

4.2 Modified Crack Layer Model

This section outlines theoretical modifications and adaptations of the crack layer theory to analyze fatigue crack propagation data related to asphalt concrete mixtures (Aglan, 1991), (Aglan et al. 1992). The crack driving force, the resistant moment and the energy dissipated are used in this approach to evaluate parameters responsible in the asphalt concrete mixture resistance to crack propagation. These parameters include $\gamma'$ a material parameter characteristic of the resistance of the material to crack propagation and $\beta'$ which reflects the dissipative character of the material. Knowing $\gamma'$ and $\beta'$ for asphalt concrete mixtures can guide the development of asphalt mixtures with superior resistance to cracking.

4.2.1 Crack Layer Theory

The objective of crack propagation studies is to identify and determine material parameters responsible for its resistance to crack propagation i.e. fracture toughness. A fundamental equation was developed by Chudnovsky (Chudnovsky, 1986) and successfully applied to characterize the resistance of highly strained nonlinear materials (Aglan and
propagation. In this approach, the Crack Layer Theory considers a fracture process as the
propagation of a layer of damaged material around the main crack. The fracture toughness
of a material is then a measure of the resistance to such motion and the region in front of
the actual crack tip where material transformation takes place is called the "active zone".
Active zone evolution is an irreversible process which is adequately described by the
thermodynamics of irreversible processes (Chudnovsky, 1986). The crack layer model
relates the rate of crack propagation to material and fracture parameters controlling
damage accumulation in the crack tip region. The rate of fatigue crack propagation based
on the crack layer approach is expressed as (Chudnovsky, 1986);

$$\frac{da}{dN} = \frac{dD/dN}{\gamma \ast R_1 - J}$$  \hspace{1cm} (4.16)

where:

- \(a\) = Crack length
- \(N\) = Number of fatigue cycles
- \(da/dN\) = Cyclic rate of fatigue crack propagation
- \(dD/dN\) = Cyclic rate of energy dissipation on material transformation associated with
  active zone evolution
- \(J\) = Crack driving force (energy release rate)
$R_1$ = Resistance moment which accounts for the amount of damage associated with the crack advance

$\gamma^*$ = Specific energy of damage (material parameter) which characterizes the resistance of crack propagation (extracted from fatigue crack propagation tests).

4.2.2 Crack Driving Force

The crack driving force can be evaluated at increments of crack length from either the area above the unloading curve or the area under the loading curve, as shown in Figure 4.2. The two definitions coincide if the cyclic load-displacement curve forms a closed loop, and the loading and unloading branches are symmetrical. However, when the loading and unloading branches are not symmetrical, Little (Little et al. 1986) showed that, the area above the unloading curve is more appropriate, due to the fact that crack propagation may occur during loading. Thus, the energy release rate ($J$) is given by;

\[
J = - \frac{dp}{da} \frac{1}{B} \tag{4.17}
\]

where:

$p$ = Potential energy

$B$ = Specimen thickness

$a$ = Crack length
Figure 4.2: Potential and Strain Energy for Cyclic Load-Deflection Curve.
4.2.3 The Resistance Moment

The resistance moment, $R_I$, in the modified model accounts for the amount of damage associated with the crack advance. As shown in Figure 4.3, minicracks emanate from the main crack on both surfaces of the tested specimens as well as inside the specimens. These minicracks mainly exist in the matrix or the aggregate-matrix interface. This phenomenon of minicracking has been observed in pavements in service under fatigue loading (Terrel, 1972). It has also been observed that the amount of transformed (damaged) material due to minicracks in the vicinity of the crack tip increases with the crack advance. Following the Irwin plastic zone size (Irwin, 1958), (McClnstock and Irwin, 1965), Aglan (Aglan et al., 1992) assumed that the width of the transformed material ahead of the crack tip (active zone) is a linear function of the crack length ($a$) up to its critical length ($a_c$). This has been shown to be true in various other materials, including metals (Mustapha, 1989), polymers (Takemori and Kambour, 1981), (Botsis et. al 1987) and polymer composites (Nguyen, Chudnovsky and Moet, 1985). Thus the resistance moment ($R_I$) can be evaluated as follows;

$$R_I = \frac{\partial \nu}{\partial a} \frac{1}{B} \quad \text{(4.18)}$$

Where:

$\nu$ = Volume of the damaged material (active zone) at crack length ($a$) for a specimen with width $B$. 
Figure 4.3: Damage in the Form of Minicracks Associated with Fatigue Crack Propagation in an Asphalt Concrete Beam.
This volume can be expressed as (from Figure 4.4),

\[ v = \frac{ahB}{2} \]  \hspace{1cm} (4.19)

where:

\[ h = \text{Width of the active zone (Figure 4.4) and is expressed as,} \]

\[ h = Ca \]  \hspace{1cm} (4.20)

where:

\[ C = \text{Constant} \]

Thus, by substituting Equation 4.20 into Equation 4.19;

\[ v = \frac{CBa^2}{2} \]  \hspace{1cm} (4.21)

On this basis, by substituting Equation 4.21 into Equation 4.18;

\[ R_1 = Ca \]  \hspace{1cm} (4.22)

Where \( C \) is a constant which depends on the angle of the active zone, and it can be evaluated from Figure 4.4 as;
Figure 4.4: Schematic Illustration of Linear Damage Evolution
\[
\tan \frac{\phi}{2} = \frac{h}{2a} = \frac{C a}{2a}
\] (4.23)

\[
C = 2 \tan \left(\frac{\phi}{2}\right)
\] (4.24)

Where:

\(\phi\) = Active zone angle as shown in Figure 4.4

Therefore \(\gamma^* R_i\) in Equation 4.16 can be evaluated as:

\[
\gamma^* R_i = \gamma^* C a = \gamma' a
\] (4.25)

where \(\gamma'\) replaces \(\gamma^*\) and \(C\) is a material constant characteristic of the material's resistance to crack propagation.

4.2.4 Energy Dissipation

The quantity \(dD/dN\) in the above mentioned model is the cyclic rate of energy dissipation during submicroscopic processes leading to damage formation. It has previously been shown (Aglan, Chudnovsky et al. 1990) that for strain controlled fatigue experiments the value of \(dD/dN\) can be expressed as;
\[
\frac{dD}{dN} = \beta J^2
\]  \hspace{1cm} (4.26)

Where:

\( \beta \) = Coefficient of energy dissipation expressing the portion of the change in work per cycle expended on damage formation during strain controlled testing.

\( J \) = Crack driving force (Equation 4.17)

For stress controlled experiments it is found that the value of \( dD/dN \) can be extracted from the area of the hysteresis loop, expressed as:

\[
\frac{dD}{dN} = \beta W_i
\]  \hspace{1cm} (4.27)

Where:

\( \beta \) = Coefficient of energy dissipation associated with the stress controlled configuration

\( W_i \) = Change in work, measured directly as the area of the hysteresis loop at any crack length (\( a \)) minus the area of the loop just before crack initiation divided by the specimen thickness, as shown in Figure (4.5)

In viscoelastic materials, \( W_i \) includes work expended on damage processes associated with crack growth and history dependent dissipation processes, with both processes being irreversible. The value of \( W_i \) has been successfully evaluated for various
Figure 4.5: Evolution of Hysteresis Loops with Increase in Crack Length.
asphalt concrete mixtures and will be employed in the current study. Taking into account
the above mentioned modifications for stress controlled fatigue, the final form of the
Modified Crack Layer equation for stress controlled fatigue is obtained by substituting
Equations 4.25 and 4.27 into Equation 4.16. Thus, \( \frac{da}{dN} \) is given by;

\[
\frac{da}{dN} = \frac{\beta W}{\gamma' a - J}
\]  

(4.28)

The parameters \( \gamma' \) and \( \beta \) which control the fatigue process, can be determined by
rearranging Equation 4.28. Thus,

\[
\frac{J}{a} = \gamma' - \beta \left( \frac{W}{(da/dN)a} \right)
\]  

(4.29)

Equation 4.29 can be used to evaluate \( \gamma' \) and \( \beta \) for asphalt concrete mixtures. By
conducting a fatigue crack propagation test and recording hysteresis loops at intervals of
crack growth, the energy release rate \( J \) can be determined by using Equation 4.17. The
change in work \( W \), at a certain crack length can also be calculated by subtracting the area of
the hysteresis loop at crack initiation from the area of the hysteresis loop at that crack
length divided by the specimen thickness. The slope of the crack length vs. number of
cycles curve gives the crack speed \( da/dN \) at a certain crack length. If the experimental
results of each asphalt mixture tested are in accord with the proposed model a plot of $J/a$ versus $\ln(Wi/(da/dN)a)$ should give a straight line with $\gamma'$ being the intercept and $\beta'$ the slope.

The Modified Crack Layer model reveals that $\gamma'$ reflects the toughness of the material. A higher value of $\gamma'$ gives a lower crack speed over the entire range of the energy release rate. $\beta'$ represents the change in work expended on damage processes and it also gives an indication about the resistance of the asphalt mixture to cracking. A higher value of $\gamma'$ and a lower $\beta'$ gives a lower crack speed over the entire range of the energy release rate. Useful relationships can be established between $\gamma'$ and $\beta'$ and chemical compositions or processing conditions for different asphalt concrete mixtures. Since the Modified Crack Layer model seems to be a suitable method to evaluate the resistance of asphalt concrete mixtures to fatigue crack propagation, it is adopted in this research, along with the previously described Paris’ model.
Chapter 5

Effect of Processing Conditions and Stress Level

The effect of processing conditions on the fracture resistance of AC-20 asphalt concrete mixtures will be studied in this chapter in view of Paris’ model and Modified Crack Layer model. Also, the invariant nature of Paris’ model parameters namely $C^*$ and $m^*$, as well as the MCL parameters, $\gamma$ and $\beta'$, when the stress level is changed, will be examined.

The AC-20 asphalt concrete mixture selected for this part of the study met gradation requirements corresponding to ODOT (Ohio Department of Transportation) specification item 403, with an optimum asphalt cement content of 8%, as determined by the Marshall method of mix design.

5.1 Effect of Processing Conditions

To study the effect of processing conditions, three identical beams were prepared using the unmodified AC-20 mixture with and without dynamic compaction following the procedure explained in Chapter 3.
5.1.1 Critical Energy Release Rate ($J_{IC}$)

The effect of dynamic compaction on the critical energy release rate ($J_{IC}$) of the AC-20 mixtures is studied in this section. Static four point flexural tests were conducted on notched beams with an initial notch depth of 1.75 inch. Relationships between the load and deflection for statically and dynamically compacted AC-20 specimens as well as for the statically compacted AC-20 specimens are shown in Figure 5.1. $J_{IC}$ values were computed from the area under the load displacement curves (the total energy to failure, $U_t$) and using Equation 2.5. Values of $J_{IC}$ and the maximum load to failure are also shown in Table 5.1.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Ultimate Load (lbs)</th>
<th>$J_{IC}$ (lb-in/in$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC-20 Stat</td>
<td>291</td>
<td>11.98</td>
</tr>
<tr>
<td>AC-20 Stat/Dyn</td>
<td>415</td>
<td>17.11</td>
</tr>
</tbody>
</table>

Figure 5.1 shows that, the area under the load-deflection curve for the dynamically compacted beams is considerably larger than the same area for the statically compacted beams, which is indicative of an increase in the toughness of the mixture. Moreover, the load-deflection curve for the dynamically compacted beams decays slowly after the maximum load is reached in contrast with the sudden drop of the load in the case of the beams where static compaction was only used. Also as shown in Table 5.1, the dynamic
Figure 5.1: Load vs Deflection (Critical Energy Release Rate) for AC-20 Mixtures
compaction of the AC-20 mixture enhances the ultimate load and the critical energy release rate by almost 40%. This is believed to be due to more effective mechanical interlock, and stronger bonds between the binder and the aggregate, promoted by the dynamic preparation method.

5.1.2 Paris Model

The applicability of Paris model on statically and dynamically compacted AC-20 mixtures is studied in this section. Relationships between the crack length ($a$) and the number of cycles $N$ for each mixture are shown in Figure 5.2. Cracking initiated at about 1800 cycles and advanced very rapidly, reaching its fatigue life at about 2500 cycles in the statically compacted AC-20 specimen. Crack initiation in the statically and dynamically compacted AC-20 mixture, started at about 8000 cycles and then advanced at a slower rate than in the statically compacted mixture, reaching its fatigue life at about 14000 cycles. Thus, it is clear that the processing of the AC-20 specimens using the dynamic compaction method increases the life of the beams by over five times, and also introduces a substantial delay in crack propagation. The slope of the curves shown in Figure 5.2 is the average crack speed at a given crack length ($a$).

Values of energy release rate ($J$) are calculated based on Equation 4.7 and plotted as a function of the crack length ($a$) for each mixture and are shown in Figure 5.3, indicating that the energy release rate is always higher in the case of the statically and dynamically compacted mixture than that of the statically compacted mixture for the same
Figure 5.2: Crack Length vs No. of Cycles for AC-20 Mixtures
Figure 5.3: J vs Crack Length for AC-20 Mixtures
crack length. This confirms the considerable increase in the resistance of the mixture when dynamic compaction was used. Values of $J$ were used to evaluate $C^*$ and $m^*$ for each mixture on the basis of Equation 4.14.

Figure 5.4 shows fatigue crack growth rates, plotted on the basis of Equation 4.14, versus energy release rate $J$, in order to extract $C^*$ and $m^*$ for each mixture. Values of $C^*$ and $m^*$ are given in Table 5.2.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>$C^*$ (in$^2$/lb-Cycle)</th>
<th>$m^*$</th>
<th>Coefficient of Determination ($R^2$)</th>
<th>Crack Speed Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC-20 Stat</td>
<td>0.118±0.05</td>
<td>1.116±0.08</td>
<td>0.926±0.02</td>
<td>0.188</td>
</tr>
<tr>
<td>AC-20 Stat/Dyn</td>
<td>0.018±0.03</td>
<td>0.944±0.04</td>
<td>0.943±0.05</td>
<td>-0.801</td>
</tr>
</tbody>
</table>

This table indicates higher values of $C^*$ and $m^*$ for the statically compacted mixture than those for the statically and dynamically compacted mixtures. Lower values of $C^*$ and $m^*$ in the case of the statically and dynamically compacted mixtures indicate its higher toughness, which is also reflected in a lower value of the crack speed index which accounts for the effects of both $C^*$ and $m^*$. As shown in Figure 5.4, for the same rate of crack propagation ($da/dN$), a higher value of the energy release rate is required for statically and dynamically compacted AC-20 than for the statically compacted samples. Thus, more energy is required to maintain the same rate of crack propagation. Both lower
values of $C^*$ and $m^*$ attests to the superiority of the statically and dynamically compacted AC-20 mixture as compared to the statically compacted one. This finding is in accord with the results previously found using the critical energy release rate ($J_{ic}$).

5.1.3 Modified Crack Layer Model

The effect of processing conditions on the fatigue resistance is examined through the Modified Crack Layer model in this section. Values of $J$ and $da/dN$ obtained previously at each crack length will be used to evaluate $\gamma'$ and $\beta'$ for each mixture on the basis of Equation 4.29.

The change in work $W$, for both mixtures is evaluated from the area of the hysteresis loops at increments of crack length. The relationship between $W$; and the crack length ($a$) for each mixture is shown in Figure 5.5. $W$, for the statically and dynamically compacted AC-20 mixture is lower than that of the statically compacted AC-20 mixture for the same crack length, also indicating that more total work has been expended on damage formation for the statically compacted mixture.

The crack speed $da/dN$, the energy release rate $J$ and the change in work $W$, obtained at each crack length were plotted on the basis of Equation 4.29 and are shown in Figure 5.6 for the two mixtures. It is observed that all points nearly plot along a straight line from which $\gamma'$ (the intercept) and $\beta'$ (the slope) can be extracted. The average values of $\gamma'$ and $\beta'$ for each mixture are shown in the following table.
Figure 5.6: $J/a$ vs $Wi/(da/dn)a$ for AC-20 Mixtures
Table 5.3: $\gamma'$ and $\beta'$ for Unmodified AC-20 Mixtures

<table>
<thead>
<tr>
<th>Mixture</th>
<th>$\gamma'$ (in-lb/in$^3$)</th>
<th>$\beta'$</th>
<th>Coefficient of Determination ($R^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC-20</td>
<td>1.11±0.24 x 10^{-2}</td>
<td>2.86±0.20 x 10^{-3}</td>
<td>0.801±0.09</td>
</tr>
<tr>
<td>AC-20</td>
<td>1.99±0.32 x 10^{-2}</td>
<td>2.95±0.15 x 10^{-4}</td>
<td>0.980±0.03</td>
</tr>
</tbody>
</table>

It is observed from Table 5.3 that the value of $\gamma'$ for the statically and dynamically compacted AC-20 mixture is about 1.8 times higher than that for the statically compacted AC-20 mixture, while the value of $\beta'$ for the statically compacted mixture is about 10 times higher than the value for the statically and dynamically compacted AC-20 mixture. A larger value of $\gamma'$ indicates that more energy is required to cause a unit volume to change from undamaged to damaged material, while a higher value of $\beta'$ reflects the larger percentage of energy expended on dissipative processes and damage growth within the active zone. These two observations indicate that the statically and dynamically compacted AC-20 mixture is more resistant to crack propagation than the statically compacted mixtures. These findings are in accord with results previously found using the critical energy release rate and Paris' model.

The enhancement in the fracture toughness of the mixture when using the dynamic/static compaction method can be related to the mechanical interlock which improved the bonds between the asphalt and the aggregates, the decrease in the void ratio and the increase in the unit weight of the mixture. In general, these findings could have an
important application. That is, by implementing a similar compaction procedure in the field, the fracture resistance of AC-20 asphalt concrete mixtures can be considerably improved.

5.2 Effect of Stress Level

In this section, the dependency of both Paris model parameters \( C^* \) and \( m^* \) and the Modified Crack Layer model parameters \( \gamma' \) and \( \beta' \) on the level of stress during fatigue loading is examined. The current study is designed to verify the invariant nature of the Paris' model and the Modified Crack Layer model parameters, which are proposed as materials parameters characteristic of the asphalt concrete mixtures' resistance to crack propagation. Multiple identical asphalt concrete beams were prepared with the AC-20 asphalt cement using static compaction. Static four point flexural tests were conducted at room temperature on three unnotched beams in order to evaluate the ultimate flexural strength and flexural modulus. The average ultimate load and the average flexural strength are shown in Table 5.4.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Ultimate Load (lbs)</th>
<th>Ultimate Flexural Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC-20</td>
<td>350</td>
<td>142.80</td>
</tr>
</tbody>
</table>
Four-point flexural fatigue crack propagation tests were performed on three geometrically identical notched beams at three levels of stress. These levels correspond to 10%, 19% and 29% of the ultimate bending stress shown in Table 5.4. Tests were conducted at a constant temperature of 70° F. All other experimental conditions were kept constant. Following is a fatigue crack propagation analysis using both Paris’ and the MCL models.

5.2.1 Paris’ Model

The applicability and dependency of Paris’ model parameters ($C^*$ and $m^*$), on the level of stress during fatigue loading is examined in this section. Relationships between the crack length ($a$) and the number of cycles $N$ for the average of the three tested specimens of the AC-20 mixture at each stress level considered, are shown in Figure 5.7. Cracking initiated at about 600 cycles and advanced very rapidly, reaching its fatigue life at about 1000 cycles for specimens tested at a stress level of 29%. Crack initiation for specimens tested at a stress level of 19%, started at 1800 cycles and then advanced at a slower rate than in the previous case, reaching its fatigue life at about 2800 cycles. Finally, crack initiation started at 3000 cycles in the specimens tested at a stress level of 10%, and then advanced at a much slower rate, reaching its fatigue life at about 4500 cycles. As expected the specimens subjected to a stress level of 29% endure the shortest number of cycles. This is obvious for both crack initiation and crack propagation. The crack speed was calculated at intervals of crack length from the slope of each curve in Figure 5.7.
Figure 5.7: Crack Length vs No. of Cycles for AC-20 Mixtures at Different Stress Ratios
Values of energy release rate ($J$) are calculated based on the potential energy principle and are plotted as a function of the crack length ($a$) for each stress level in Figure 5.8. The value of $J$ for the specimen subjected to the highest stress, 29%, is always higher than its counterparts for the (19% and 10% stress levels), at the same crack length. The values of $J$ at each crack length will be employed in the present analysis to extract $C^*$ and $m^*$ at each stress level.

Fatigue crack growth rates are plotted in Figure 5.9, on the basis of Equation 4.14, as a function of energy release rate $J$, in order to extract $C^*$ and $m^*$ at each stress level. Values of $C^*$, $m^*$ and crack speed index are given in Table 5.5.

<table>
<thead>
<tr>
<th>Stress Level</th>
<th>$C^*$ (in$^3$/lb-Cycle)</th>
<th>$m^*$</th>
<th>Coefficient of Determination ($R^2$)</th>
<th>Crack Speed Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>29%</td>
<td>0.138±0.003</td>
<td>1.108±0.004</td>
<td>0.974±0.05</td>
<td>0.247</td>
</tr>
<tr>
<td>19%</td>
<td>0.118±0.024</td>
<td>1.116±0.012</td>
<td>0.922±0.09</td>
<td>0.188</td>
</tr>
<tr>
<td>10%</td>
<td>0.171±0.029</td>
<td>1.089±0.015</td>
<td>0.920±0.02</td>
<td>0.322</td>
</tr>
</tbody>
</table>

This Table indicates that, the values of $C^*$ and $m^*$ are almost constant with an average values of 0.142 and 1.104 respectively. This is also evident from Figure 5.10 in which the average line represents the experimental data at different stress levels very well.
Figure 5.9: da/dN vs J for AC-20 Mixtures at Different Stress Ratios
Figure 5.10: One Linear Fit of da/dN vs J for AC-20 Mixtures at Different Stress Ratios
The same trend is observed for the crack speed index which is almost constant with an average value of 0.252. Bearing in mind the severe heterogeneity of the asphalt concrete mixture, the variation in $C^*$ and $m^*$ appears to be insignificant and it can be concluded that the values of $C^*$ and $m^*$ are independent of the stress level.

### 5.2.2 Modified Crack Layer Model

The effect of stress level on the fatigue resistance of the AC-20 asphalt mixtures is examined following the Modified Crack Layer theory. The average values of $\gamma'$ and $\beta'$ as extracted from the MCL model using the fatigue crack propagation data, have been determined based on three tested specimens at each stress level. Comparisons between the values of $\gamma'$ and $\beta'$ at the various stress levels are made. The values of $J$ and $da/dN$ obtained previously at each crack length will be employed in the present analysis to extract $\gamma'$ and $\beta'$ at each stress level.

The change in work $W$, for beams tested at each stress level is evaluated from the area of the hysteresis loops at increments of crack length. Relationships between $W$, and the crack length ($a$) for each stress level are shown in Figure 5.11. The value of $W$, for the specimen subjected to the highest stress, 29%, is higher than that for specimen subjected to 10% and 19% stress level, indicating that more total work has been expended on damage formation for the specimen subjected to the 29% stress level. These relationships between $W$, and the crack length, will also be employed in the current analysis to evaluate $\gamma'$ and $\beta'$.
Figure 5.11: Wi vs Crack Length for AC-20 Mixtures at Different Stress Ratios
The crack speed \( \frac{da}{dN} \), the energy release rate \( J \) and the change in work \( W \), obtained at each crack length were plotted on the basis of Equation 4.29 and are shown in Figure 5.12 for each stress level. It is observed that all points nearly plot along a straight line from which \( \gamma' \) (the intercept) and \( \beta' \) (the slope) can be extracted. The average values of \( \gamma' \) and \( \beta' \) for each mixture are shown in Table 5.6.

<table>
<thead>
<tr>
<th>Stress Level</th>
<th>( \gamma' ) (in-lb/in(^3))</th>
<th>( \beta' )</th>
<th>Coefficient of Determination ( (R^2) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>29 %</td>
<td>1.40±0.21 \times 10^{-2}</td>
<td>3.33±0.19 \times 10^{-3}</td>
<td>0.962±0.08</td>
</tr>
<tr>
<td>19 %</td>
<td>1.11±0.08 \times 10^{-2}</td>
<td>2.86±0.28 \times 10^{-3}</td>
<td>0.801±0.09</td>
</tr>
<tr>
<td>10 %</td>
<td>1.06±0.13 \times 10^{-2}</td>
<td>3.22±0.08 \times 10^{-3}</td>
<td>0.988±0.01</td>
</tr>
</tbody>
</table>

Figure 5.13 shows the experimental data along with the regression straight line based on the average value of \( \gamma' \) and \( \beta' \). It can be seen that this line represents the data at different stress levels very well. Thus, the variation in \( \gamma' \) and \( \beta' \) appears to be insignificant and it is probably safe to conclude that the values of \( \gamma' \) and \( \beta' \) are independent of the stress level.

The consistency of \( \gamma' \) can be understood by examining Equation 4.29, namely
Figure 5.12: \( J/a \) vs \( Wi/(da/dN)a \) for AC-20 Mixtures at Different Stress Ratios
\[
\frac{J}{a} = \gamma' - \beta' \left( \frac{W_i}{(da/dN) a} \right)
\] (4.29)

and taking the term \( W_i/(da/dN) a = 0 \) in order to obtain \( \gamma' \) (the intercept). It is noted that \( \gamma' \) depends on the ratio \( J/a \), as can be seen in Figure 5.12. This ratio which is simply the intercept with the vertical axis, is almost constant for the three stress levels shown and hence \( \gamma' \) is independent of the stress level.

By a similar analysis now assuming that the terms \( J/a \) and \( \gamma' \) are constant in Equation 4.23, it can be seen that \( \beta' \) depends on the ratio \( W_i/(da/dN) a \). It has been observed that both \( W_i \) and \( da/dN \) increase with the increased stress level and it appears that they have increased proportionally since \( \beta' \) has remained constant.

Analysis of stress controlled fatigue crack propagation experiments based on the Modified Crack Layer theory revealed that the parameters controlling the fracture process \( \gamma' \) and \( \beta' \) appear to be independent of the stress level of fatigue loading. The average value of \( \gamma' \) for various stress levels for the AC-20 mixture is evaluated as \( 1.19 \times 10^{-2} \) in-lb/in³ and \( \beta' \) is \( 3.14 \times 10^{-3} \).

As previously seen, the validity of both Paris' and the Modified Crack Layer models has been tested by studying the effect of processing conditions and the stress level. The examination of both \( C^* \), \( m^* \) (Paris' model) and \( \gamma', \beta' \) (MCL model) reflected the superior toughness of the AC-20 mixture subjected to both dynamic and static conditions.
compaction as compared to that of the mixture which had only been statically compacted. Also, the usefulness of $C^*$, $m^*$ (Paris' model) and $\gamma^*, \beta^*$ (MCL model) as material parameters has been shown with studies on the effect of stress level during fatigue testing. It was evident that all parameters showed no dramatic change over the range of stress levels tested, and thus they can be considered independent of the stress level.
Chapter 6

Effect of SBS Content

In this chapter, the effect of SBS (Styrene-Butadiene-Styrene) content on the mechanical and fatigue behavior of AC-5 mixtures is studied. The choice of SBS was influenced by the fact that it was the best among three modifiers tested previously, to improve the fatigue crack propagation resistance of the AC-5 mixture. An optimum AC-5 asphalt cement of 8% was used, as determined by the Marshall method of mix design. SBS contents of 6%, 10% and 15% as a percentage of the binder were used. The effect of SBS content on the general mechanical properties of SBS modified AC-5 asphalt concrete mixtures was investigated and compared with samples of unmodified AC-5 mixtures (0% SBS content). These mechanical properties include the indirect tensile strength, resilient modulus, and the critical energy release rate. Invoking Paris’ and the Modified Crack Layer models, emphasis was placed on developing relationships between additive content and fracture resistance parameters namely $C^*$, $m^*$ for Paris’ model and $\gamma$, $\beta'$ for the Modified Crack Layer model.
Scanning Electron Microscopy (SEM) was also performed on samples of the SBS modified AC-5 mixtures. These SEM results were compared with those of unmodified AC-5 mixtures.

6.1 Chemical Characteristics of SBS Modified Binder

In the current study, focus is placed on SBS (Kraton D4463) because it has been found to be one of the most effective additives, particularly with soft asphalt such as AC-5. SBS, manufactured by Shell Chemical Co., is produced by combining Kraton rubber with plasticizer oils. Properties of SBS are shown in Table 6.1.

Table 6.1: Properties of SBS (Kraton D4463)  
(Shuler et al., 1992)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength, psi</td>
<td>545</td>
</tr>
<tr>
<td>Modulus @ 300% Elongation, psi</td>
<td>35</td>
</tr>
<tr>
<td>Elongation, %</td>
<td>1300</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>0.93</td>
</tr>
<tr>
<td>Melt Viscosity, g/10 min</td>
<td>31</td>
</tr>
<tr>
<td>[Styrene / Rubber] Ratio</td>
<td>30/70</td>
</tr>
<tr>
<td>Physical Form</td>
<td>Pellet</td>
</tr>
</tbody>
</table>
SBS consists of two different polymer blocks, hard polystyrene endblocks chemically crosslinked to soft rubbery midblocks in a three-dimensional rubber network. The hard polystyrene endblocks give SBS rubber its high tensile strength and flow resistance at high temperature, whereas the rubbery mid-blocks are responsible for its elasticity, fatigue resistance and flexibility at low temperature. When SBS rubber is mixed with hot asphalt, the polystyrene endblock domains begin to soften, allowing molecules into the asphalt while the rubbery midblocks start absorbing the asphalt's maltene fraction and swell to many times their initial volume. This swelling causes the SBS rubber phase to dominate the asphalt phase, resulting in a new modified asphalt binder possessing the principal characteristics of rubber. After the asphalt mixture is cooled, the polystyrene endblock domains reharden and form physical crosslinks with the rubbery midblocks, forming a strong, elastic, three-dimensional network again (Shell Chemical Co., 1989). It should be expected that this bonding action of the asphalt with the SBS rubber is largely influenced by the additive content percentage. Below a certain SBS content, the polymer will only act as a filler since the polymer network in the binder does not yet exist. However, above a certain SBS content, the polymer forms a complete network and the asphalt acts as an extender. Between these two limits, a large variation in the binder characteristics exists. Because the bonding action of the asphalt with the SBS rubber enhances the asphalt aggregate adhesion, it is of interest to study the effect of the additive percentage on both the macromechanical and the micromechanical behavior of asphalt concrete mixtures. These effects are addressed in the current study.
6.2 Indirect Tensile Strength

The load versus the vertical displacement curves for the Marshall sized cylindrical specimen for SBS modified AC-5 mixtures with different SBS content (0%, 6%, 10% and 15%) are shown in Figure 6.1. Values of the ultimate indirect strength and fracture energy (Equation 2.2) for each are given in Table 6.2.

<table>
<thead>
<tr>
<th>SBS Content</th>
<th>Indirect Tensile Strength (psi)</th>
<th>Fracture Energy $G_f$ (lb/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>20.82</td>
<td>6.55</td>
</tr>
<tr>
<td>6%</td>
<td>29.16</td>
<td>9.52</td>
</tr>
<tr>
<td>10%</td>
<td>45.14</td>
<td>16.65</td>
</tr>
<tr>
<td>15%</td>
<td>66.97</td>
<td>22.81</td>
</tr>
</tbody>
</table>

Table 6.2 shows that, the mixture with 15% of SBS has a higher tensile strength than the other mixtures. Also the unmodified AC-5 mixture (0% SBS) has the lowest tensile strength. As the percentage of additive increases the indirect tensile strength increases as well. This can be attributed to the effect of the hard polystyrene endblocks.

Also, it is indicated from Table 6.2 that, the fracture energy ($G_f$, which is calculated from the area under the load-deflection curve in Figure 6.1) increased as the
Figure 6.1: Load vs Deflection (Indirect Tensile Strength) for AC-5 Mixtures at Different SBS Contents
percentage of additive increased. This means that more energy is required to fracture samples with 15% of SBS. Thus, it appears that the SBS content has a considerable effect on the toughness of the mixture as indicated by the fracture energy (\(G_f\)). As the percentage of SBS increased the toughness of the mixture increased as well. This behavior was expected because of the strong elastic three dimensional network formed by the polystyrene endblock domain of the SBS modifier.

6.3 Resilient Modulus

Values of the resilient modulus (\(M_r\)) for each mixture are shown in Table 6.3. These represent the average resilient modulus of three identical specimens. It is noted that the \(M_r\) for the 15% of SBS mixture is about 1.4 times higher than that for the unmodified mixture. In general, the value of \(M_r\) increases as the percentage of SBS increases.

<table>
<thead>
<tr>
<th>SBS Content</th>
<th>Resilient Modulus (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>1,265,570</td>
</tr>
<tr>
<td>6%</td>
<td>1,375,640</td>
</tr>
<tr>
<td>10%</td>
<td>1,489,220</td>
</tr>
<tr>
<td>15%</td>
<td>1,767,880</td>
</tr>
</tbody>
</table>
As it was mentioned before, SBS consists of thermoplastic rubber midblocks and polystyrene endblocks. Thus, it appears that, within the range of SBS percentage tested, the hard polystyrene endblocks enhance the resilient modulus of the mixture.

6.4 Critical Energy Release Rate

The effect of SBS content on the critical energy release rate of the AC-5 mixtures is studied in this section. Three beams were fabricated for each SBS to AC-5 asphalt ratio using ODOT item 403 gradation requirements and according to the same procedure mentioned in Chapter 3. Static four point flexural tests were conducted on notched beams with an initial notch depth of 1.75 inch. Average values of the ultimate load and critical energy release rate for each mixture are shown in Table 6.4.

<table>
<thead>
<tr>
<th>SBS Content</th>
<th>Ultimate Load (lbs)</th>
<th>( J_{te} ) (lb.in/in(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>145</td>
<td>6.42</td>
</tr>
<tr>
<td>6%</td>
<td>215</td>
<td>8.95</td>
</tr>
<tr>
<td>10%</td>
<td>370</td>
<td>10.33</td>
</tr>
<tr>
<td>15%</td>
<td>574</td>
<td>13.39</td>
</tr>
</tbody>
</table>
Relationships between the load and deflection for notched beams for each mixture are shown in Figure 6.2. $J_{Ic}$ values were computed from the area under the load displacement curves (the total energy to failure, $U_l$) using Equation 2.5. It is seen that the 15% of SBS mixture has the highest ultimate load and the highest critical energy release rate. Also, it is indicated from Table 6.4 that, the critical energy release rate increases as the percentage of SBS increases nearly in a linear fashion. However, higher percentage of SBS should be tested in order to determine whether or not there is an optimum additive content. It is also noticed that the area under the load-deflection curve for the 15% of SBS mixture is greater than that for the other three mixtures, indicating its higher toughness. This can also be attributed to the strong elastic three dimensional network formed by the polystyrene endblock domain of the modifier.

Thus, it appears that, as the percentage of SBS increases, the rubbery midblocks continue to increase the toughness as indicated by the increased area under the load deflection curve in Figure 6.2. Increasing the percent of SBS above 15% could stiffen the material making it more brittle and less resistant to crack propagation with the polystyrene hard blocks dominating over the rubbery toughening midblocks. However, using a higher SBS content will be more expensive and may result in a less workable mixture with poorer wetting ability associated with the high level of cohesion developed by the polymer.

In practice, a 6% or less additive by weight is commonly used. Although it is not economically feasible to add more than 6% additive, higher contents of 10% and 15% were
Figure 6.2: Load vs Deflection (Critical Energy Release Rate) for AC-5 Mixtures at Different SBS Cont.
used to understand the fundamental changes in the mechanical behavior of modified asphalt concrete mixtures.

6.5 Paris’ Model

In this section, the applicability of Paris’ equation (Equation 4.13) on AC-5 mixtures modified with different SBS contents as well as unmodified AC-5 mixtures, has been examined for the purpose of comparison with the MCL approach.

Relationships between the crack length (a) versus the number of cycles N for each ratio of SBS mixtures, are shown in Figure 6.3. This figure indicates that, cracking initiated at about 1000 cycles and advanced very rapidly, reaching its fatigue life at about 2000 cycles in the case of unmodified AC-5 specimens. In the case of 6% of SBS specimens, cracking initiated at about 1800 cycles and advanced very rapidly, reaching its fatigue life at about 3000 cycles. Crack initiation started at about 2100 cycles in the 10% of SBS mixture and then advanced at a slower rate than in the 6% of SBS specimen, reaching its fatigue life at about 3500 cycles. Finally, crack initiation started at about 2500 cycles in the 15% of SBS mixture and then it advanced at a slower rate than in the 10% of SBS, reaching its fatigue life at about 3800 cycles. The slope of the curves in Figure 6.3 measures the average crack speed at each crack length interval. It can also be seen that the propagation and initiation life time are relatively greater in the case of the 15% of SBS than in the other three mixtures at the same stress level, while the unmodified AC-5 mixture has the shortest initiation and propagation life.
Figure 6.3: Crack Length vs. No. of Cycles for AC-5 Mixtures at Different SBS Contents
Based on the potential energy principle, the energy release rate $J$ is evaluated following Equation 4.7 and plotted against the crack length for each mixture in Figure 6.4. The average value of $J$ at a crack length of 2 inches for the 15% SBS mixture is about 6 times higher than the unmodified AC-5 mixture at the same crack length. In general, as the SBS percentage increases the average value of $J$ increases at the same crack length, which indicates the role of increased SBS content in increasing the fracture resistance of the mixture. The value of $J$ at each crack length is employed in the present analysis to evaluate $C^*$ and $m^*$ for each mixture.

Equation 4.13 was applied to the 0%, 6%, 10% and 15% SBS modified AC-5 asphalt mixtures at 70° F. Values of the energy release rate $J$, obtained previously through Equation 4.7, were plotted as a function of the crack speed $da/dN$ in Figure 6.5. The constants $C^*$ and $m^*$ in Equation 4.13 were obtained for each mixture. Also, the crack speed index was calculated for each mixture through Equation 4.15. Table 6.5 shows the values of $C^*$, $m^*$ and the crack speed index for each mixture.

<table>
<thead>
<tr>
<th>SBS Content</th>
<th>$C^*$ (in²/lb-Cycle)</th>
<th>$m^*$</th>
<th>Coefficient of Determination ($R^2$)</th>
<th>Crack Speed Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>0.063±0.003</td>
<td>1.512±0.14</td>
<td>0.964±0.02</td>
<td>0.311</td>
</tr>
<tr>
<td>6%</td>
<td>0.018±0.007</td>
<td>0.810±0.08</td>
<td>0.961±0.09</td>
<td>-0.935</td>
</tr>
<tr>
<td>10%</td>
<td>0.015±0.004</td>
<td>0.586±0.03</td>
<td>0.899±0.02</td>
<td>-1.238</td>
</tr>
<tr>
<td>15%</td>
<td>0.012±0.001</td>
<td>0.313±0.05</td>
<td>0.933±0.05</td>
<td>-1.608</td>
</tr>
</tbody>
</table>
Figure 6.4: J vs Crack Length for AC-5 Mixtures at Different SBS Contents
Figure 6.5: da/dN vs J for AC-5 Mixtures at Different SBS Contents
This table indicates that $C^*$ decreases with increasing percentage of SBS, which is a sign of increased toughness in the mixture. A similar trend was observed between $m^*$ and SBS content, with $m^*$ decreasing as the percentage of SBS increased. Both lower values of $C^*$ and $m^*$ observed in the case of the 15% of SBS mixture indicate more resistance to crack propagation. Thus, more energy is required to maintain the same crack speed for the 15% of SBS mixture as compared with the other three mixtures. The 10%, 6% and 0% of SBS mixtures follow in decreasing order. Theses trends indicate that, both $C^*$ and $m^*$ can be considered useful tools in the evaluation of the fracture toughness of asphalt concrete mixtures.

Considering the crack speed index, which accounts for the effect of both parameters $C^*$ and $m^*$, it was found that the crack speed index decreased as the SBS content increased, which is indicative of the increase in the mixture toughness. Within the range of SBS percentage tested (0% to 15%) it appears that both the polystyrene endblocks and butadiene rubbery midblocks are working together to improve the fracture toughness of SBS modified asphalt mixtures as indicated by the crack speed index.

6.6 Modified Crack Layer Model

A similar fatigue crack propagation analysis was performed using the Modified Crack Layer model on data generated for the SBS modified AC-5 mixtures with different SBS contents of 0%, 6%, 10% and 15%. The values of $J$ and $d\Delta dN$ obtained previously at
each crack length are employed in the present analysis to evaluate $\gamma'$ and $\beta'$ for each mixture.

The change in work $W$, versus the crack length for typical beams tested for each mixture is shown in Figure 6.6. $W$, for the unmodified AC-5 mixture is always higher than for the 6%, 10% and 15% of SBS mixtures. Thus, more work has been expended on damage formation within the active zone of the unmodified AC-5 specimens. This is also demonstrated by the considerably larger size of the hysteresis loops recorded in the case of the unmodified AC-5 specimens in comparison to those of the 6%, 10% and 15% of SBS specimens. Relationships between $W$, and the crack length ($\alpha$) are later used in the evaluation of $\gamma'$ and $\beta'$.

The crack speed $da/dN$, the energy release rate $J$ and the change in work $W$, obtained at various crack lengths for each mixture were plotted following Equation 4.29 and are shown in Figure 6.7. It is observed that nearly all points plot along a straight line, from which $\gamma'$ (the intercept) and $\beta'$ (the slope) can be extracted. This is also observed in the other two identical specimens tested for each mixture. The average values of $\gamma'$ and $\beta'$ for each mixture are shown in Table 6.6.

These results indicate that the average value of $\gamma'$ for the 15% of SBS mixture is higher than that for the 0%, 6% and the 10% of SBS mixtures. This indicates that more energy is required to cause a unit volume of the 15% of SBS mixture to change from undamaged to damaged material, qualifying this mixture as the most resistant to crack propagation. In decreasing order, the 10%, 6% and then 0% of SBS mixtures follow as
Figure 6.6: Wi vs Crack Length for AC-5 Mixtures at Different SBS Contents
shown in Figure Table 6.6. Also the lowest value of $\beta'$ for the 15% of SBS mixture indicates that a lower percentage of energy is expended on dissipative processes and damage growth within the active zone. Both a higher $\gamma'$ and a lower $\beta'$ in the case of the 15% of SBS mixture make it more resistant to crack propagation i.e. tougher than the other three mixtures.

<table>
<thead>
<tr>
<th>SBS Content</th>
<th>$\gamma'$ (in-lb/in$^3$)</th>
<th>$\beta'$</th>
<th>Coefficient of Determination ($R^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>1.01±0.15 x 10$^{-2}$</td>
<td>5.92±0.12 x 10$^{-3}$</td>
<td>0.865±0.07</td>
</tr>
<tr>
<td>6%</td>
<td>2.13±0.07 x 10$^{-2}$</td>
<td>2.66±0.08 x 10$^{-3}$</td>
<td>0.974±0.03</td>
</tr>
<tr>
<td>10%</td>
<td>2.75±0.17 x 10$^{-2}$</td>
<td>2.12±0.02 x 10$^{-3}$</td>
<td>0.919±0.06</td>
</tr>
<tr>
<td>15%</td>
<td>3.77±0.01 x 10$^{-2}$</td>
<td>1.85±0.05 x 10$^{-3}$</td>
<td>0.865±0.02</td>
</tr>
</tbody>
</table>

The previous results indicate that, the AC-5 asphalt concrete mixture containing 15% SBS (of the total binder) is more resistant to fatigue crack propagation than mixtures containing 0%, 6% or 10% SBS. An optimum SBS content is expected at a higher percentage.

It can be also concluded that both Paris’ and the Modified Crack Layer models successfully discriminated the subtle effects introduced by different SBS contents
within AC-5 asphalt concrete mixtures. Thus, establishing relationships between fatigue parameters such as $C^*$, $m^*$, $\gamma'$ and $\beta'$ and percentage of additives can serve an important role in modified asphalt mixture design. Such relationships can guide the development of asphaltic mixtures with superior resistance to crack propagation (fracture toughness) and to assess the life of these mixtures.

6.7 Scanning Electron Microscopy Analysis

Scanning Electron Microscopy (SEM) analysis on the effect of SBS percentage, reveals a distinct trend in the appearance of the fractured surface from that of the unmodified mixture (Figure 6.8), as the amount of modifier increases. These SEM samples are cut from the fractured surface of the specimen ahead of the initial notch. Since the SBS modifier is being mixed initially with the hot asphalt, asphalt rich areas from the fracture surface were examined.

At 100X magnification a definite trend in the microstructural features (ridging) emerges. This can be seen in Figures 6.9 through 6.11 for the 6%, 10% and 15% of SBS mixtures respectively. This ridging increases in both frequency and size as the percent of SBS in the asphalt increases. The increase in ridging formation in the asphalt rich areas with the increase in SBS percentage appears to be the result of the increased resistance of the matrix to surface separation. Better adhesion between the binder and the aggregate as well as better cohesion within the binder can result in microstretching of the binder, producing these ridges on the fractured surface.
Figure 6.8: Microstructural Features of an Asphalt-Rich Area in the Unmodified AC-5 Mixtures (0% SBS).

Figure 6.9: Microstructural Features of an Asphalt-Rich Area in the 6% SBS Modified AC-5 Mixtures.
Figure 6.10: Microstructural Features of an Asphalt-Rich Area in the 10% SBS Modified AC-5 Mixtures.

Figure 6.11: Microstructural Features of an Asphalt-Rich Area in the 15% SBS Modified AC-5 Mixtures.
Moreover an increase in adhesion, induced by the increase in additive percentage in
the AC-5 asphalt, is evident in Figures 6.12 through 6.14. In these micrographs at 2000X
magnification, the morphology of the fine aggregate particles is compared at different
percentages of SBS. It should be mentioned that these fine aggregate particles are well
below the minimum sieve size used in this study. These particles are fine dust which initially
clings to the aggregate. They are then separated from the surface of the aggregate at the
time of mixing with the hot liquid asphalt and become a particle phase in the mixture. At
0% SBS, the surface of the particles appears to be very clean as shown in Figure 6.12. At
10% SBS, these particles appear to have a thicker coating than at 0% SBS, as shown in
Figure 6.13. At 15% SBS, there is dramatic increase in the amount of binder adhered to the
surface of these particles, as shown in Figure 6.16. Again this attests to the enhancement of
the adhesive properties of the binder with the increase in the percentage of the SBS
modifier.

The interpretation of the SEM results appears to be in complete agreement with
the results from both the mechanical behavior and flexural fatigue investigations. As the
percentage of the SBS modifier increases, the indirect tensile strength, resilient modulus
and critical energy release rate increase. The fracture toughness evaluated using Paris’ and
the Modified Crack Layer models also increased with higher SBS content.
Figure 6.12: Morphology of Fine Aggregate Particles in the Unmodified AC-5 Mixtures (0% SBS).

Figure 6.13: Morphology of Fine Aggregate Particles in the 10% SBS Modified AC-5 Mixtures.
Figure 6.14: Morphology of Fine Aggregate Particles in the 15% SBS Modified AC-5 Mixtures.
Chapter 7

Effect of Temperature and Thermal Cycling

The effect of temperature and thermal cycling on the mechanical properties and the fatigue behavior of both AC-5 and SBS modified AC-5 asphalt concrete mixtures, is studied in this Chapter. Also, the total dissipated energy to failure under fatigue testing is presented as a fracture criteria which accounts for the effect of temperature and stress level.

7.1 Effect of Temperature

Due to the viscoelastic properties of the asphalt binder, temperature can be considered as the most significant variable affecting its performance. If the pavement cools to a low temperature, tensile stresses develop due to the pavement’s tendency to contract. If the tensile stress exceeds the tensile strength of the asphalt concrete mixtures at that temperature, microcracks will develop. On the other hand, if the pavement heats to a high temperature, the stiffness of the asphalt concrete mixture will
proposed to improve the high temperature as well as the low temperature properties of AC-5 asphalt concrete mixtures. Therefore, the mechanical and fatigue properties of SBS modified and unmodified asphalt concrete mixtures at different temperatures will be studied in this section. Specimens were tested at temperatures of 35°, 70° and 85° F. The stress level and all other experimental parameters were kept constant.

7.1.1 Mechanical Performance

The effect of temperature on the mechanical properties of AC-5 and 6% SBS modified asphalt concrete mixtures is studied in this section. These properties include, indirect tensile strength, resilient modulus and the critical energy release rate ($J_{ic}$). Results for each test are presented next.

7.1.1.1 Indirect Tensile Strength

Figures 7.1.a and 7.1.b show the load versus vertical displacement variation for Marshall sized cylindrical specimen using conventional AC-5 and SBS modified mixtures at different temperatures. Values of the ultimate load, indirect tensile strength and fracture energy for each mixture at different temperatures are given in Table 7.1. Fracture energy values were computed from the area under the load displacement curve in Figures 7.1.a and 7.1.b and using Equation 2.2.
Figure 7.1.a: Load vs Deflection (Indirect Tensile Strength Test) for AC-5 Mixtures

Figure 7.1.b: Load vs Deflection (Indirect Tensile Strength Test) for 6% SBS Modified Mixtures
Table 7.1: Indirect Tensile Strength and Fracture Energy for AC-5 and 6% SBS Modified Mixtures at Different Temperatures

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Indirect Tensile Strength (psi)</th>
<th>Fracture Energy $G_f$(lb/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC-5</td>
<td>SBS</td>
</tr>
<tr>
<td>35° F</td>
<td>49.7</td>
<td>65.39</td>
</tr>
<tr>
<td>70° F</td>
<td>27.8</td>
<td>35.50</td>
</tr>
<tr>
<td>85° F</td>
<td>7.0</td>
<td>20.63</td>
</tr>
</tbody>
</table>

As expected, this table indicates that for both mixtures, the indirect tensile strength inversely varies with temperature. Similar behavior was observed by other researchers such as Hewit et al. (Hewit et al., 1967) who stated that, for asphalt concrete mixtures, the tensile strength at 60° F increased more than 10 times when the temperature was reduced to 0° F. The same temperature influence was also found by Tons et al. (Tons et al., 1963). So, in general it can be stated that lower tensile strength characteristics occur at high temperature and high tensile strength characteristics occur at low temperature. However, somewhere between +10° F and -20° F the asphalt concrete approaches the glassy temperature range, and as a result it becomes more glass-like than elastic, which results in a lower tensile strength. Table 7.1 also shows that, for both mixtures, as the temperature decreases the fracture
energy increases, indicating that more energy is required to fracture samples tested at lower temperatures.

The previous results showed that, the tensile strength of AC-5 mixture is extremely sensitive to temperature changes. There was a 7 fold drop in the indirect tensile strength as the temperature increased from 35° F to 85° F. However, when SBS was used the indirect tensile strength decreased only about 3 times. There was also a considerable increase in the indirect tensile strength and fracture energy at each temperature for SBS modified mixtures as compared with AC-5 mixtures. Thus, the hard polystyrene endblocks enhance the tensile strength properties of SBS modified mixtures.

7.1.1.2 Resilient Modulus

Values of the resilient modulus ($M_r$) for AC-5 and 6% SBS modified mixtures at different temperatures are shown in Table 7.2. These represent the average of three identical specimens. Also as expected, $M_r$ decreases as the temperature increases. The average value of $M_r$ at a temperature of 35° F for AC-5 mixtures is about 1.45 times higher than when measured at 70° F, and about 2.6 times higher as compared to that at 85° F. While for SBS modified mixtures, the value of $M_r$ at a temperature of 35° F is about 1.2 times higher than when measured at 70° F, and about 1.8 times higher as compared to that at 85° F. Also, it is noticed that for both mixtures, the $M_r$ value
decreases at a higher rate as the temperature increases from 70° to 85° F than when the temperature increases from 35° to 70° F.

Table 7.2: Resilient Modulus for AC-5 and 6% SBS Modified Mixtures at Different Temperatures

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Resilient Modulus (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC-5</td>
</tr>
<tr>
<td>35°F</td>
<td>1,851,210</td>
</tr>
<tr>
<td>70°F</td>
<td>1,265,570</td>
</tr>
<tr>
<td>85°F</td>
<td>703,730</td>
</tr>
</tbody>
</table>

It can be concluded that the resilient modulus strongly depends on the temperature at which the specimen is tested. This view is also shared by Cochran (Cochran, 1989) and Furber (Furber, 1989), in which they indicated that the resilient modulus decreased significantly with increased temperature.

The resilient modulus test results shown indicate that, SBS has no significant effect on the stiffness at 70° F. However, SBS causes a significant increase in stiffness at high temperature and a significant decrease in stiffness at low temperature as shown in Table (7.2). Therefore, SBS is expected to improve the temperature susceptibility of asphalt concrete mixtures by reducing the variation of the resilient modulus with changes in temperature.
7.1.1.3 Critical Energy Release Rate

The effect of temperature on the critical energy release rate of the AC-5 and 6% SBS modified asphalt concrete mixtures is studied in this section. Static four point flexural tests were conducted on notched beams with an initial notch depth of 1.75 inches. Relationships between the load and deflection for notched beams at three different temperatures are shown in Figures 7.2.a and 7.2.b for each mixture. It is noted from Figure 7.2.a that, the ultimate load increases as the temperature decreases for AC-5 mixtures, while the maximum deflection at failure shows an opposite trend to that observed for the ultimate load. Figure 7.2.b indicates that, the behavior of the 6% SBS mixture at $85^\circ$ F differs significantly from its behavior at both $35^\circ$ and $70^\circ$ F. At 85 F, the load initially increased in a linear fashion up to about 40% of the maximum load and then increased in a nonlinear fashion. After the maximum load was reached it remained almost constant accompanied with a large increase in the deflection. $J_{ic}$ values were computed from the area under the load displacement curves (the total energy to failure, $U_t$) in Figures 7.2.a, 7.2.b and using Equation 2.5. Values of $J_{ic}$ and the maximum load to failure for each mixture are shown in Table 7.3.

Table 7.3 indicates, as in the previous parameters, that the temperature has a great effect on the critical energy release rate which increases as the temperature decreases. Thus, at low temperature more energy is released when a crack advances than at high temperature. This is due to the fact that, at low temperatures, the failure of the asphalt concrete is partly in the asphalt and partly in the aggregate, but at high
Figure 7.2.a: Load vs Deflection (Critical Energy Release Rate Test) for AC-5 Mixtures

Figure 7.2.b: Load vs Deflection (Critical Energy Release Rate Test) for 6% SBS Modified Mixtures
temperatures the failure is in the asphalt cement binder and not in the aggregate. Due
to its sensitivity to changes in temperature $J_{IC}$ may not be considered as a material
constant for AC-5 asphalt concrete mixtures. However, it can be used qualitatively to
compare the resistance of mixtures to crack propagation.

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Ultimate Load (lbs)</th>
<th>$J_{IC}$ (lb.in/in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC-5</td>
<td>SBS</td>
</tr>
<tr>
<td>35°F</td>
<td>270.6</td>
<td>381.2</td>
</tr>
<tr>
<td>70°F</td>
<td>145.7</td>
<td>215.8</td>
</tr>
<tr>
<td>85°F</td>
<td>70.1</td>
<td>129.8</td>
</tr>
</tbody>
</table>

It can be concluded that, over the entire range of temperatures used, a
considerable improvement in the critical energy release rate ($J_{IC}$) was observed when
6% of SBS was used. It is evident from Table 7.3 that SBS improved the fracture
toughness of the AC-5 mixture as indicated by higher values of $J_{IC}$. The hard
polystyrene endblocks give SBS rubber its higher toughness at high temperature, while
the rubbery mid-blocks improved the fracture toughness at low temperature.
7.1.2 Fatigue Crack Growth Analysis

The effect of temperature on the fatigue crack growth behavior of both AC-5 asphalt and 6% SBS modified AC-5 asphalt concrete mixtures is also studied in this chapter. Fatigue crack propagation tests were conducted to evaluate the fracture toughness parameters for both Paris' and the Modified Crack Layer models at temperatures of 35°, 70° and 85° F.

7.1.2.1 Paris' Model

As mentioned before, Paris' equation [Equation. 4.13] has received considerable attention from several researchers as a useful approach to characterize the material's resistance to crack propagation. The applicability of Paris' equation to paving mixtures at different temperatures, is examined in this section, and later compared with the MCL approach.

Relationships between the crack length ($a$) versus the number of cycles $N$ for specimens tested at 35°, 70° and 85° F for both mixtures are shown in Figures 7.3.a, 7.3.b and 7.3.c. Figure 7.3.b indicates that for AC-5 mixtures when tested at 70° F, cracking initiated at about 1000 cycles and advanced rapidly, reaching its fatigue life at about 2000 cycles, while for SBS modified mixtures, cracking initiated at about 1800 cycles, reaching its fatigue life at about 3000 cycles. This indicates that, SBS modified mixtures have higher propagation and fatigue lives than AC-5 mixtures. A similar trend was observed at 35° and 85°, as seen in Figures 7.3.a and 7.3.c respectively. The slopes of the curves in Figures
Figure 7.3.a: Crack Length vs No. of Cycles at 35º F

Figure 7.3.b: Crack Length vs No. of Cycles at 70º F
Figure 7.5.c: Crack Length vs No. of Cycles at 85°F
7.3.a, 7.3.b and 7.3.c measure the average crack speed at each crack length interval. Thus, under the given loading conditions, SBS modified mixtures have better fatigue behavior as indicated by the propagation life and number of cycles to failure ($N_f$). The increase of fatigue life at low temperature in the case of SBS modified mixtures can be related to the effect of the rubbery midblocks which improves low temperature properties. However, at high temperature the increase in fatigue life is related to better adhesion between the modified binder and the aggregate.

The energy release rate $J$ is evaluated for each mixture based on Equation 4.17, and plotted against the crack length for tests conducted at 35°, 70° and 85° F in Figures 7.4.a and 7.4.b. It can be seen that, the value of $J$ at a certain crack length increases as the temperature decreases. However, at the same temperature, it is noted that $J$ is always higher for SBS modified mixtures as compared to the unmodified mixtures. The value of $J$ at each crack length is employed in the present analysis to evaluate $C^*$ and $m^*$ at each temperature.

Equation 4.13 was applied to both AC-5 and 6% SBS modified asphalt concrete mixtures tested at 35° F, 70° F and 85° F. The values of the energy release rate $J$ obtained previously through Equation 4.7 were plotted in a log-log graph as a function of the crack speed $da/dN$ and are shown in Figures 7.5.a and 7.5.b. The constants $C^*$ and $m^*$ in Equation 4.13 were obtained at each temperature, and are shown in Table 7.4 along with the crack speed index for both mixtures at each temperature.
Figure 7.4a: $J$ vs Crack Length for AC-5 Mixtures

Figure 7.4b: $J$ vs Crack Length for 6% SBS Modified Mixtures
Figure 7.5.a: $da/dN$ vs $J$ for AC-5 mixtures

Figure 7.5.b: $da/dN$ vs $J$ for 6% SBS Modified Mixtures
Table 7.4: $C^*$, $m^*$ and Crack Speed Index for AC-5 and 6% SBS Modified Mixtures at Different Temperatures

<table>
<thead>
<tr>
<th>Temperature</th>
<th>$C^*$ (in²/lb-Cycle)</th>
<th>$m^*$</th>
<th>Crack Speed Index</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC-5</td>
<td>SBS</td>
<td>AC-5</td>
</tr>
<tr>
<td>35°F</td>
<td>0.079</td>
<td>0.021</td>
<td>1.461</td>
</tr>
<tr>
<td></td>
<td>±0.01</td>
<td>±0.005</td>
<td>±0.24</td>
</tr>
<tr>
<td>70°F</td>
<td>0.063</td>
<td>0.018</td>
<td>1.512</td>
</tr>
<tr>
<td></td>
<td>±0.003</td>
<td>±0.007</td>
<td>±0.31</td>
</tr>
<tr>
<td>85°F</td>
<td>0.996</td>
<td>0.038</td>
<td>1.759</td>
</tr>
<tr>
<td></td>
<td>±0.015</td>
<td>±0.008</td>
<td>±0.19</td>
</tr>
</tbody>
</table>

Table 7.4 indicates that, for AC-5 mixtures the values of $C^*$ and $m^*$ are almost constant and independent of temperature at 35°F and 70°F, however there is a considerable increase in $C^*$ and $m^*$ at 85°F. A similar trend was also observed in the crack speed index, which is a combined form of parameters $C^*$ and $m^*$.

These trends indicate that, both $C^*$ and $m^*$ can be considered material constants for AC-5 mixtures at temperatures of up to 70°F. However, $C^*$ and $m^*$ are no longer material constants at higher temperatures, due to the loss of loading capacity of the AC-5 mixtures. This view is shared by many researchers including Majidzadeh et al. (Majidzadeh et al., 1972), who stated that, $C^*$ and $m^*$ are material constants for AC-5 asphalt concrete mixtures at low temperature but not at high temperature.

Table 7.4 also indicates that, the values of $C^*$ and $m^*$ are almost constant and independent of temperature for SBS modified mixtures. The same trend was also
observed in the crack speed index. Figure 7.6 shows the experimental data at the three temperatures plotted along with the straight line fit based on the average values of $C^*$ and $m^*$. It can be seen that the average line fit represents the data at the different temperatures reasonably well. Theses trends indicate that, both of $C^*$ and $m^*$ are independent of temperature and can be considered material constants for SBS modified AC-5 mixtures. It is also seen from Table 7.4 that, SBS modified mixtures have a higher fracture toughness within the range of temperature considered as compared to AC-5 mixtures. This is reflected by the lower values of $C^*$, $m^*$ and the crack speed index of the 6% SBS modified mixtures at each temperature.

7.1.2.2 Modified Crack Layer Model

The effect of temperature on the fatigue resistance is examined through the Modified Crack Layer model in this section. Fatigue crack growth analysis was performed on data generated for AC-5 and SBS modified asphalt concrete mixtures at 35°, 70° and 85° F. The values of $da/dN$ and $J$ at each crack length, obtained previously, will be employed in the present analysis to evaluate $\gamma'$ and $\beta'$ at each temperature.

The change in work $W$, versus the crack length for typical beams tested using AC-5 and SBS modified mixtures at 35° F, 70° F and 85° F are shown in Figures 7.7.a and 7.7.b. It is obvious from both figures that the change in work increases as the temperature increases at the same crack length. This is also demonstrated by the considerably smaller size of the hysteresis loops recorded in the specimens tested at 35° F, in comparison with
Figure 7.6: $\frac{da}{dN}$ vs $J$ (One Linear Fit of Data at Different Temperatures) for 6% SBS Modified Mix.
Figure 7.7a: Wi vs Crack Length for AC-5 Mixtures

Figure 7.7b: Wi vs Crack Length for 6% SBS Modified Mixtures
those tested at 70°F and 85°F. However, the value of $W_r$ for AC-5 mixtures is always higher than for SBS modified mixtures at the same temperature, thus, less work has been expended on damage formation within the active zone for SBS modified specimens. These relationships between $W_r$ and the crack length ($a$) will be employed in the current analysis to evaluate $\gamma'$ and $\beta'$ at various test temperatures.

The crack speed $da/dN$, the energy release rate $J$ and the change in work $W_r$ obtained at various crack lengths for each mixture were plotted following Equation 4.29 and are shown in Figures 7.8.a and 7.8.b. It is observed that nearly all points plot along a straight line, from which $\gamma'$ (the intercept) and $\beta'$ (the slope) can be extracted. Values of $\gamma'$ and $\beta'$ for each mixture at each temperature are shown in Table 7.5.

<table>
<thead>
<tr>
<th>Temperature</th>
<th>$\gamma'$ (in-lb/in$^3$)</th>
<th>$\beta'$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC-5</td>
<td>SBS</td>
</tr>
<tr>
<td>35°F</td>
<td>1.19±0.21×10$^{-2}$</td>
<td>3.08±0.11×10$^{-2}$</td>
</tr>
<tr>
<td>70°F</td>
<td>1.01±0.15×10$^{-2}$</td>
<td>2.87±0.08×10$^{-2}$</td>
</tr>
<tr>
<td>85°F</td>
<td>0.54±0.09×10$^{-2}$</td>
<td>2.69±0.13×10$^{-2}$</td>
</tr>
</tbody>
</table>

This table indicates that the value of $\gamma'$ at 35°F and 70°F is almost constant for the AC-5 mixture. This was also found to be the case when testing AC-20 mixtures at
Figure 7.8.a: $J/a$ vs $W/(da/dN)a$ for AC-5 Mixtures

Figure 7.8.b: $J/a$ vs $W/(da/dN)a$ for 6% SBS Modified Mixtures
various stress levels as described in Chapter 5. However, the value of $\gamma'$ was much lower at 85°F than at 35°F and 70°F, which indicates a large decrease in the fracture toughness of the mixture and the lack of enough loading capacity. This is also supported by the very low values of the indirect tensile strength, fracture energy, resilient modulus and $J_{ik}$ obtained on AC-5 mixtures tested at 85°F, as previously discussed.

It was also found that, $\beta'$ which reflects the dissipative character of the material changes with temperature. This was not the case for the stress level tests detailed in Chapter 5. There is an increase in $\beta'$ as the test temperature increases from 35°F to 70°F. At temperatures higher than 70°F, the dissipative coefficient $\beta'$ increased sharply.

Unlike AC-5 mixtures, the value of $\gamma'$ for the 6% SBS modified mixture is almost constant, over the entire range of temperature used, and $\beta'$, which reflects the dissipative character of the material, shows less dependency on temperature, but it increases as the temperature increases. Thus, it appears that the fracture process in general is controlled by $\gamma'$ which reflects the fracture toughness rather than $\beta'$ the dissipative coefficient. The consistency of the specific energy of damage $\gamma'$ with temperature and the dependency of the dissipative coefficient $\beta'$ on temperature have recently been reported by Brillhart and Botsis (Brillhart and Botsis, 1992).

It is also seen from Table 7.5 that the value of specific energy of damage $\gamma'$ is always higher for SBS modified than for AC-5 mixtures, however $\beta'$ takes an opposite trend to that observed for $\gamma'$. Such findings reflect the superior toughness and the favorable
crack growth properties of 6% SBS to AC-5 mixtures. This was also confirmed by the lower values of $C^*$, $m^*$ and the crack speed index in the case of SBS modified mixtures when Paris’ model was used in the analysis.

It can be concluded that, both Paris’ and the Modified Crack Layer models have successfully demonstrated the enhancement on the fatigue crack growth resistance of AC-5 mixtures when modified with 6% SBS. However, experimental results showed that Paris’ model parameters ($C^*$ and $m^*$) are both independent of temperature. The Modified Crack Layer model indicated however, that the dissipative coefficient $\beta'$ strongly depends on temperature, therefore it cannot be considered a material parameter.

### 7.2 Fatigue-Energy Relationships of SBS Modified Mixtures

As mentioned before, fatigue testing of asphalt concrete mixtures has followed two modes of loading: controlled stress and controlled strain. In this section, the controlled stress mode of testing is used to correlate both the total dissipative energy over the fatigue life and the number of cycles to failure to the change in temperature and the applied stress and. The fatigue behavior will be quantitatively defined by performing a series of cyclic four-point flexural fatigue tests utilizing three ranges of stress level and temperature. Relationship between fatigue, total dissipated energy and stress at different temperatures are given in Table 7.6.
Table 7.6: No. of Cycles to Failure and Accumulated Energy vs Stress Level for 6% SBS Modified AC-5 Mixtures at Different Temperatures

<table>
<thead>
<tr>
<th>Stress Level</th>
<th>16.3 psi</th>
<th>34.7 psi</th>
<th>48.6 psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature</td>
<td>$N_f$</td>
<td>Energy (lb-in)</td>
<td>$N_f$</td>
</tr>
<tr>
<td>$40^\circ$ F</td>
<td>39,970</td>
<td>340.2</td>
<td>18,500</td>
</tr>
<tr>
<td>$70^\circ$ F</td>
<td>3,675</td>
<td>292.1</td>
<td>1,210</td>
</tr>
<tr>
<td>$85^\circ$ F</td>
<td>2,064</td>
<td>261.9</td>
<td>595</td>
</tr>
</tbody>
</table>

7.2.1 Fatigue Life-Stress-Temperature Relationships

The number of fatigue cycles to failure ($N_f$) is plotted as a function of temperature and stress level in a 3D plot in Figure 7.9.a. It is shown that fatigue life strongly depends on temperature and stress level. At a particular stress level, longer fatigue lives are found at lower temperatures. This behavior can be explained in terms of specimen stiffness. As the temperature decreases the stiffness of the mixture increases resulting in a smaller deformation and a longer life. Also, at a particular temperature, the number of cycles to failure decreases as the stress level increased. In general fatigue damage increases with increasing stress level.

The least squares regression equation for $N_f$ as a function of temperature and stress level is given as,
Figure 7.9.a: Number of Fatigue Cycles to Failure as a Function of Stress and Temperature.
\[ \ln(N_f) = a + \frac{b}{T} + c \sigma \]  \hspace{1cm} (7.1)

Where,

\[ N_f \] = Number of Fatigue Cycles to Failure.

\[ T \] = Temperature (F)

\[ \sigma \] = Applied Stress (psi)

\[ a \] = 5.627

\[ b \] = 226.866

\[ c \] = -0.043

Equation 7.1 represents the experimental data reasonably well with a coefficient of determination \( (R^2) = 0.999 \). It can be seen that number of fatigue cycles to failure \( (N_f) \) is more sensitive to changes in temperature rather than the stress level. This is also reflected by the relatively lower value of the coefficient \( (c) \) as compared to the coefficient \( (b) \).

### 7.2.2 Energy-Stress-Temperature Relationships

The cyclic stress-strain curves (hysteresis loops) have been used to model SBS modified asphalt concrete mixtures cyclic response under fatigue. The energy dissipated in one load cycle is represented by the area of the corresponding hysteresis loop. Each hysteresis loop represents the amount of energy dissipated on damage
formation. As the number of cycles increases, the amount of energy dissipated increases until failure occurs. Thus the accumulated energy to failure can be used to study fatigue behavior.

In Figure 7.9.b, the total dissipative energy is plotted as a function of temperature and stress level in a 3D plot. It is seen that at a given stress level, as the temperature decreases the total energy dissipated to fatigue failure increases, which means that, the fatigue damage caused by a certain stress level at high temperatures is higher than the fatigue damage caused by the same stress level at low temperatures. Also, at the same temperature, the accumulated energy decreases as the stress level increases.

The accumulated energy to failure can be correlated to the temperature and the applied stress in a multi variable least squares regression analysis using the following equation;

\[ E = a + bT + c \sigma \] \hspace{1cm} (7.2)

Where,

- \( E \) = Total energy dissipated in fatigue failure.
- \( T \) = Temperature (F)
- \( \sigma \) = Applied stress (psi)
- \( a \) = 458.681
Figure 7.9.b: Total Accumulated Energy as a Function of Stress and Temperature.
\[ b = -1.874 \]
\[ c = -2.344 \]

Equation 7.2 represents the experimental data with a coefficient of determination \( R^2 = 0.979 \). Unlike \( N_f \) \( E \) is more sensitive to the change in stress level rather than temperature. This is demonstrated by a higher absolute value of the coefficient \( c \) as compared to the coefficient \( b \).

### 7.2.3 Energy–Fatigue life Relationships

Relationships between the total dissipative energy and number of cycles to failure at different temperatures plotted in a log-log graph are shown in Figure 7.9.c. It is noted that at a certain temperature the total energy dissipated in fatigue failure increases as fatigue life increases.

The accumulated energy to failure can be correlated to fatigue life in a least squares regression analysis using a power law equation,

\[
E = a \left( N_f \right)^b
\]

Where;

- \( E \) = Total energy dissipated in fatigue failure.
- \( N_f \) = Number of Cycles to Failure
- \( a, b \) = Regression Constants.
Figure 7.9.c: Total Accumulated Energy vs Number of Cycles at Different Temperatures
Equation 7.3 is applied to the experimental data shown in Table 7.6. Values of $a$ and $b$ are given in Table 7.7 at each temperature.

<table>
<thead>
<tr>
<th></th>
<th>$a$</th>
<th>$b$</th>
<th>Coefficient of Determination ($R^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40°F</td>
<td>63.52</td>
<td>0.159</td>
<td>0.947</td>
</tr>
<tr>
<td>70°F</td>
<td>73.11</td>
<td>0.169</td>
<td>0.982</td>
</tr>
<tr>
<td>85°F</td>
<td>81.94</td>
<td>0.150</td>
<td>0.970</td>
</tr>
</tbody>
</table>

Table 7.7 indicates that, the values of $a$ and $b$ are almost constant and independent of temperature with an average of 72.86 and 0.159 respectively. This is also evident from Figure 7.9.d in which the average line represents the experimental data at different temperatures very well. Considering the heterogeneity of asphalt concrete mixture, the variation in $a$ and $b$ appears to be insignificant and it can be concluded that they are independent of temperature.

Thus, the total accumulated dissipated energy through fatigue failure can be used as a promising tool to study fatigue, since it is successfully correlated to fatigue life through temperature independent parameters. However, more work has to be done to account for service factors such as frequency and type of loading. Also, the
Figure 7.2.d: One Linear Fit of Total Accumulated Energy vs Number of Cycles at Different Temp.
mixed mode of loading has to be considered, since it would more closely simulate
in-situ pavement conditions, rather than using an equivalent damage concept.

7.3 Effect of Thermal Cycling

Thermal cycling is known to cause transverse cracking in asphalt concrete
pavements, considerably shortening their life, and particularly at low temperatures. This is
the result of higher stresses generated by shrinkage which may exceed the tensile strength
of the asphalt concrete. Environmental aging of asphalt concrete mixtures by low
temperature thermal cycling was conducted and is discussed in this section to examine the
ability of SBS to improve the performance of asphalt concrete mixtures. The mechanical
and fatigue behavior of AC-5 and 6% SBS modified mixtures exposed to different levels
of low temperature thermal cycling is investigated.

7.3.1 Mechanical Performance

The effect of low temperature thermal cycling on the mechanical properties
such as indirect tensile strength, resilient modulus and the critical energy release rate
($J_c$) of AC-5 and SBS modified asphalt concrete mixtures will be studied in this
section. A cyclic thermal aging program was performed in an environmental chamber
between 0° F and 70° using five levels of thermal cycling, namely 0, 7, 14, 21 and 28
cycles. Results for each type of test are presented next.
7.3.1.1 Indirect Tensile Strength

The load as a function of vertical displacement for both AC-5 and 6% SBS modified specimens tested at 70° F and exposed to different numbers of thermal cycles are shown in Figures 7.10.a and 7.10.b respectively. Values of the indirect tensile strength and the fracture energy at each number of thermal cycles are given in Table 7.8.

Table 7.8: Indirect Tensile Strength and Fracture Energy for AC-5 and 6% SBS Modified Mixtures at Different Thermal Cyclic Levels

<table>
<thead>
<tr>
<th>No. of Thermal Cycles</th>
<th>Indirect Tensile Strength (psi)</th>
<th>Fracture Energy $G_f$ (lb/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC-5</td>
<td>SBS</td>
</tr>
<tr>
<td>0</td>
<td>27.8</td>
<td>35.5</td>
</tr>
<tr>
<td>7</td>
<td>22.7</td>
<td>37.1</td>
</tr>
<tr>
<td>14</td>
<td>21.1</td>
<td>37.4</td>
</tr>
<tr>
<td>21</td>
<td>18.2</td>
<td>33.0</td>
</tr>
<tr>
<td>28</td>
<td>16.8</td>
<td>30.8</td>
</tr>
</tbody>
</table>

This table indicates that, for AC-5 mixtures, the tensile strength is inversely proportional to the number of thermal cycles, while it slightly increases as the number of thermal cycles increases up to 14 cycles and then it decreases for 6% SBS modified mixtures. However, the fracture energy which is the work needed to cause tensile failure, considerably decreased for both mixtures with an increase in the number of thermal cycles,
Figure 7.10.a: Load vs Deflection (Indirect Tensile Strength Test) for AC-5 Mixtures

Figure 7.10.b: Load vs Deflection (Indirect Tensile Strength Test) for 6% SDS Modified Mixtures
indicating that less work is needed to cause failure as the number of thermal cycles increases.

These results indicate that, exposure of asphalt concrete mixtures to a high number of thermal cycles decreases their tensile strength. However, values of the tensile strength and the fracture energy of 6% SBS modified mixtures are significantly higher than for AC-5 mixtures at the same level of thermal cycling reflecting the enhancement in the mixture’s performance when adding SBS.

7.3.1.2 Resilient Modulus

Average values of the resilient modulus at 70°F, based on the testing of three identical specimens exposed to different number of thermal cycles for the AC-5 and 6% SBS modified mixtures are given in Table 7.9.

<table>
<thead>
<tr>
<th>No. of Thermal Cycles</th>
<th>Resilient Modulus (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC-5</td>
</tr>
<tr>
<td>0</td>
<td>1,265,570</td>
</tr>
<tr>
<td>7</td>
<td>1,294,270</td>
</tr>
<tr>
<td>14</td>
<td>1,318,740</td>
</tr>
<tr>
<td>21</td>
<td>1,409,980</td>
</tr>
<tr>
<td>28</td>
<td>1,466,590</td>
</tr>
</tbody>
</table>

Table 7.9: Average Resilient Modulus for AC-5 and 6% SBS Modified Mixtures at Different Thermal Cyclic Levels
This table indicates for both AC-5 and SBS modified mixtures, $M_r$, actually increases with the number of thermal cycles, which indicates that the mixture became less flexible as the number of thermal cycles increased. However, even though SBS modified mixtures always exhibit higher $M_r$ values than AC-5 mixtures, the increase only amounts to about 8%, and no conclusion can be drawn regarding the benefits of SBS with regard to $M_r$ and the number of thermal cycles.

### 7.3.1.2 Critical Energy Release Rate

The effect of thermal cycling on the critical energy release rate of the AC-5 and 6% SBS modified mixtures is studied in this section. Static four point flexural tests were conducted on notched beams with an initial notch depth of 1.75 inches. Relationships between the load and deflection for notched beams at each thermal cycling level are shown in Figures 7.11a and 7.11.b. $J_{tc}$ values were computed from the area under the load displacement curves (the total energy to failure, $U_t$) and using Equation 2.5. Values of $J_{tc}$ and the maximum load to failure are also shown in Table 7.10.

The critical energy release rate as well as the ultimate load for each mixture decrease as the number of thermal cycles increases. The decrease in $J_{tc}$ as the number of thermal cycles increases indicates the reduction in the fatigue resistance of the mixtures due to thermal cycling. In general, SBS modified mixtures showed a higher
Figure 7.11.a: Load vs Deflection (Critical Energy Release Rate Test) for AC-5 Mixtures

Figure 7.11.b: Load vs Deflection (Critical Energy Release Rate Test) for 6% SBS Modified Mixtures
value of $J_{lc}$ as compared to AC-5 mixtures at the same level of thermal cycling, which reflects the ability of SBS in improving the fracture toughness of asphalt concrete mixtures. This can be related to the effect of the rubbery properties of the butadiene mid-blocks, as stated before.

<table>
<thead>
<tr>
<th>No. of Thermal Cycles</th>
<th>Ultimate Load (lbs)</th>
<th>$J_{lc}$ (lb.in/ln²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC-5</td>
<td>SBS</td>
</tr>
<tr>
<td>0</td>
<td>145.7</td>
<td>215.8</td>
</tr>
<tr>
<td>7</td>
<td>130.2</td>
<td>198.5</td>
</tr>
<tr>
<td>14</td>
<td>124.4</td>
<td>161.8</td>
</tr>
<tr>
<td>21</td>
<td>99.1</td>
<td>117.8</td>
</tr>
<tr>
<td>28</td>
<td>74.4</td>
<td>100.4</td>
</tr>
</tbody>
</table>

**7.3.2 Fatigue Crack Growth Analysis**

The effect of low temperature thermal cycling on the fatigue crack growth behavior of both AC-5 asphalt and 6% SBS modified asphalt concrete mixtures, is studied in this section. Fatigue crack propagation tests were conducted at 70° F to evaluate the fracture toughness parameters for both Paris' and the Modified Crack
Layer models at different levels of thermal cycling. Three identical beams were prepared using asphalt contents of 8% for each mixture and following the procedure explained in Chapter 3. The SBS content was kept constant at 6% of the binder.

7.3.2.1 Paris' Model

The applicability of Paris' model on AC-5 and SBS modified mixtures exposed to different levels of thermal cycling, is examined in this section. Relationships between the crack length ($a$) and the number of fatigue cycles ($N$) for typical specimens of both mixtures at different number of thermal cycles are shown in Figures 7.12.a and 7.12.b. The fatigue life for AC-5 mixtures decreases rapidly at the first two levels of thermal cycling and then decreases at a slower rate as the number of thermal cycles increases, while SBS essentially neutralizes the effect of thermal cycling up to 14 cycles and the points cluster closely. However, there exists a difference between their behavior when the first derivative of each curve is calculated to obtain the crack speed. SBS modified samples subjected to 21 and 28 thermal cycles have a marked decrease in their fatigue life and a faster crack propagation rate. It is also observed that for both mixtures, as the number of thermal cycles increases the crack speed at the same fatigue cycle increases which is an indication of the damage in the mixture due to thermal cycling. However at the same number of thermal cycles, the crack speed is slower in the SBS modified mixtures as compared to the AC-5 mixtures.
Figure 7.12.a: Crack Length vs No. of Cycles for AC-5 Mixtures

Figure 7.12.b: Crack Length vs No. of Cycles for SBS 6% Modified Mixtures
Based on the potential energy principle, the energy release rate $J$ was evaluated and plotted against the crack length in Figures 7.13.a and 7.13.b. It is noticed that for AC-5 mixtures, $J$ decreases at the same crack length as the number of thermal cycles increases. The lower values of $J$, at a certain crack length, with increased thermal cycling reflect the reduction on the fatigue cracking potential. However, for 6% SBS modified mixtures, as with the crack length ($a$) versus the number of fatigue Cycles ($N$) curves, the effect of thermal cycling is not clearly seen until the 21 and 28 thermal cycles specimens. Lower values of $J$, at a certain crack length, with increased thermal cycling are expected. However, it is interesting to note how the SBS maintains its improved properties up to a certain level of cycling, after which the improved resistance starts to decrease. The value of $J$ at each crack length will be employed in the present analysis to extract $C^*$ and $m^*$ for each mixture.

Fatigue crack growth rates are plotted in Figures 7.14a and 7.14.b., on the basis of Equation 4.13 as a function of energy release rate $J$ in a log-log plot to extract $C^*$ and $m^*$ at each level of thermal cycling. Values of $C^*$, $m^*$ and crack speed index are given in Table 7.11. It is indicated that, for SBS modified mixtures, the values of $C^*$, $m^*$ and the crack speed index slightly increase from 0 to 14 cycles, and then it increases rapidly with an increase in the number of thermal cycles from 14 to 28, while for AC-5 mixtures they increase almost at a constant rate. The increase in $C^*$ and $m^*$ as the number of thermal cycles increases indicates a reduction in the resistance to crack propagation.
Figure 7.13.a: J vs Crack Length for AC-5 Mixtures

Figure 7.13.b: J vs Crack Length for 6% SBS Modified Mixtures
Figure 7.14.a: $da/dN$ vs $J$ (Paris' Model) for 6% SBS Modified Mixtures

Figure 7.14.b: $da/dN$ vs $J$ (Paris' Model) for 6% SBS Modified Mixtures
Table 7.11: $C^*$, $m^*$ and Crack Speed Index for AC-5 and 6% SBS Modified Mixtures at Different Thermal Cyclic Levels

<table>
<thead>
<tr>
<th>No. of Thermal Cycles</th>
<th>$C^*$ (in$^2$/lb-Cycle)</th>
<th>$m^*$</th>
<th>Crack Speed Index</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC-5</td>
<td>SBS</td>
<td>AC-5</td>
</tr>
<tr>
<td>0</td>
<td>0.063</td>
<td>0.018</td>
<td>1.512</td>
</tr>
<tr>
<td></td>
<td>$\pm 0.003$</td>
<td>$\pm 0.007$</td>
<td>$\pm 0.31$</td>
</tr>
<tr>
<td>7</td>
<td>0.171</td>
<td>0.029</td>
<td>1.653</td>
</tr>
<tr>
<td></td>
<td>$\pm 0.011$</td>
<td>$\pm 0.005$</td>
<td>$\pm 0.35$</td>
</tr>
<tr>
<td>14</td>
<td>0.403</td>
<td>0.067</td>
<td>1.750</td>
</tr>
<tr>
<td></td>
<td>$\pm 0.046$</td>
<td>$\pm 0.004$</td>
<td>$\pm 0.14$</td>
</tr>
<tr>
<td>21</td>
<td>1.690</td>
<td>0.273</td>
<td>1.851</td>
</tr>
<tr>
<td></td>
<td>$\pm 0.44$</td>
<td>$\pm 0.012$</td>
<td>$\pm 0.22$</td>
</tr>
<tr>
<td>28</td>
<td>3.664</td>
<td>0.863</td>
<td>1.892</td>
</tr>
<tr>
<td></td>
<td>$\pm 0.68$</td>
<td>$\pm 0.15$</td>
<td>$\pm 0.29$</td>
</tr>
</tbody>
</table>

Thus it seems that, thermal cycling tends to stiffen asphalt concrete mixtures and in turn reduce their fatigue resistance. However, based on the values of $C^*$ and $m^*$ and the crack speed index at each thermal cycle, it can be stated that SBS reduces the effect of thermal cycling aging on modified asphalt concrete mixtures, particularly up to 14 thermal cycles. It also can be concluded that, under the given loading conditions, SBS modified mixtures show a better fatigue behavior in terms of $C^*$ and $m^*$. In addition, the resistance to fatigue crack growth based on Paris’ model is consistent with the mechanical performance of asphalt concrete mixtures.
7.3.2.2 Modified Crack Layer Model

The effect of thermal cycling on the fatigue resistance of AC-5 and SBS modified mixture is examined in this section through the Modified Crack Layer model. The crack speed \((da/dN)\) which was calculated from the slope of each curve in Figures 7.12.a and 7.12.b and the energy release rate \((J)\) at each crack length, will be used in the Modified Crack Layer model to extract \(\gamma'\) and \(\beta'\) for each mixture.

The change in work \(W\), for each mixture at different thermal cycles is evaluated from the area of the hysteresis loops at increments of crack length. Relationships between \(W\), and the crack length \((a)\) for typical beams tested at various levels of thermal cycling are shown in Figures 7.15.a and 7.15.b. The value of \(W\), increases with increasing number of thermal cycles. Thus, increased thermal cycling leads to more work being expended on damage formation and history dependent dissipative processes within the active zone of the AC-5 mixture during fatigue loading.

The crack speed \((da/dN)\), the energy release rate \(J\) and the change in work \(Wi\) obtained at each crack length were plotted on the basis of Equation 4.29 and are shown in Figures 7.16.a and 7.16.b. It is evident from these figures that, the experimental points for all mixtures at each level of thermal cycling have a fairly good straight line fit where \(\gamma'\) is the intercept and \(\beta'\) is the slope. The average values of \(\gamma'\) and \(\beta'\) for both mixtures at each thermal cycle are given in Table 7.12.
Figure 7.15.a: Wi vs Crack Length for AC-5 Mixtures

Figure 7.15.b: Wi vs Crack Length for 6% SBS Modified Mixtures
Figure 7.16.a: $J/a$ vs $Wu/(da/dN)a$ for AC-5 Mixtures

Figure 7.16.b: $J/a$ vs $Wu/(da/dN)a$ for 6% SBS Modified Mixtures
Table 7.12: $\gamma'$ and $\beta'$ for AC-5 and 6% SBS Modified Mixtures at Different Thermal Cyclic Levels

<table>
<thead>
<tr>
<th>No. of Thermal Cycles</th>
<th>$\gamma'$ (in-lb/in$^3$)</th>
<th>$\beta'$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC-5</td>
<td>SBS</td>
</tr>
<tr>
<td>0</td>
<td>1.01±0.15 $\times 10^{-2}$</td>
<td>2.87±0.08 $\times 10^{-2}$</td>
</tr>
<tr>
<td>7</td>
<td>0.89±0.04 $\times 10^{-2}$</td>
<td>2.75±0.02 $\times 10^{-2}$</td>
</tr>
<tr>
<td>14</td>
<td>0.81±0.02 $\times 10^{-2}$</td>
<td>2.95±0.15 $\times 10^{-2}$</td>
</tr>
<tr>
<td>21</td>
<td>0.68±0.16 $\times 10^{-2}$</td>
<td>2.56±0.21 $\times 10^{-2}$</td>
</tr>
<tr>
<td>28</td>
<td>0.49±0.17 $\times 10^{-2}$</td>
<td>1.20±0.09 $\times 10^{-2}$</td>
</tr>
</tbody>
</table>

This table indicates that, for AC-5 mixtures, $\gamma'$ decreases in an almost linear fashion as the number of thermal cycles increases, while the coefficient of energy dissipation $\beta'$ takes an opposite trend to that observed for $\gamma'$. The decrease in the value of $\gamma'$ and the increase in $\beta'$ is indicative of reduced resistance to crack propagation. On the other hand, for SBS modified mixtures $\gamma'$ remains approximately constant from 0 to 14 cycles. Then it decreases slightly with the increase in the number of thermal cycles. The coefficient of energy dissipation $\beta'$ slightly increases with the increasing number of thermal cycles up to 14 cycles and then it increases at a higher rate at higher number of thermal cycles. Thus,
based on the MCL model, a decrease in $\gamma'$ coupled with an increase in $\beta'$ of the mixture, after 14 cycles, indicate a decline in the mixture’s resistance to fatigue crack propagation.

Based on the previous results, a decrease in the fatigue resistance was observed with the increased number of thermal cycles for AC-5 mixtures. Thus, it seems that exposure of the mixture to a high number of thermal cycles tends to increase its stiffness and to decrease its resistance to fatigue cracking. However, the SBS modifier helps maintain a constant fatigue resistance of the mixture over a moderate number of thermal cycles. This can be attributed to the flexibility of the mixture induced by the SBS modifier. The fatigue performance of the asphalt concrete mixture as predicted from Paris’ and the MCL models was found to be consistent with what was predicted from the indirect tensile strength and the critical energy release rate tests.
Chapter 8

Summary, Conclusions and Recommendations

for Future Work

8.1 Summary and Conclusions

The main objectives of this research were to study the mechanical performance and fatigue crack propagation behavior of polymer modified and unmodified asphalt concrete mixtures at different stress levels and environmental conditions. The mechanical performance evaluation was based on the indirect tensile strength, the resilient modulus and the critical energy release rate ($J_{rc}$). Fatigue crack propagation analysis was evaluated through a fracture mechanics approach using Paris’ and the Modified Crack Layer (MCL) models. Fatigue crack growth data included the crack speed ($da/dN$), the energy release rate ($J$) and the change in work expended on damage formation within the crack tip region, these data have been used to extract parameters controlling the fracture process such as $C^*$ and $m^*$ for Paris’ model and $\gamma'$ and $\beta'$ for the Modified Crack Layer model. Also an
Controlled stress fatigue crack propagation tests indicated that both Paris’ and the Modified Crack Layer models, describe reasonably well the fatigue behavior of asphalt concrete mixtures. By analyzing the fatigue crack propagation data, the enhancement of the fracture resistance of the AC-20 is evident when using the dynamic/static compaction technique as described in chapter 5. Both models successfully characterized the resistance of the dynamic/static compacted and static compacted AC-20 mixtures, determining that the dynamic/static compacted mixture had a higher $\gamma'$ and a lower $C^*$, $m^*$ and $\beta'$ compared with the statically compacted mixture. It is also to be mentioned that the critical energy release rate ($J_{ic}$) of the AC-20 mixture increases significantly when using the dynamic compaction, indicating that the introduction of the dynamic compaction technique not only increases the resistance of asphalt concrete mixtures to fatigue crack propagation but its fracture toughness as well. This finding could have an important practical application, such that by implementing a similar compaction technique in the field, the fracture resistance of AC-20 asphalt mixtures can be considerably improved. Also, based on fatigue crack propagation studies on AC-20 asphalt concrete mixtures at 70°F, it was evident that all parameters controlling the fracture processes ($\gamma', \beta', C^*$ and $m^*$) are independent of the stress level.

The effect of Styrene-Butadiene-Styrene (SBS) to asphalt ratio in improving the resistance of asphalt concrete mixtures was examined, as presented in Chapter 6. Four SBS to asphalt ratios of 0%, 6%, 10% and 15% were examined. A 15% of SBS to asphalt ratio produced the most desirable results, by displaying the highest resistance to crack
propagation. This was evident from the higher $\gamma'$ and lower $C^*, m^*$ and $\beta'$ for the 15% of SBS mixture as well as from the considerably higher value of the indirect tensile strength, the resilient modulus and the critical energy release rate. The SBS mixtures did not yield an optimum additive content. It is possible that an increase in the SBS percentage will cause a decrease in the fracture toughness as the polystyrene hard endblocks become dominant making the mixture more brittle. This point should be investigated further. However, higher SBS contents will increase the mixture’s cost substantially.

Scanning Electron Microscopy analysis of the fractured surface revealed ridge formation in binder rich areas which increase in size and intensity as the SBS percentage increased. Better adhesion between the binder and the aggregate as well as better cohesion within the binder result in microstretching of the binder producing these ridges on the fracture surface. It is believed that this is the mechanism by which the SBS modified AC-5 asphalt concrete mixtures acquire their toughness.

The effect of temperature on the mechanical and fatigue performance of 6% SBS modified and unmodified AC-5 asphalt concrete mixtures at temperatures of 35° F, 70° F and 85° F were presented in Chapter 7. Fatigue crack propagation studies reveal that Paris’ model parameters ($C^*$ and $m^*$) showed no change over the range of temperature used. However, when the MCL model was used, it was found that the specific energy of damage $\gamma'$ is independent on the temperature, while the dissipative parameter $\beta'$ increases with an increase in temperature. These findings, coupled with the previous ones qualify $C^*$, $m^*$ and
\( y' \) as a material constant characteristic of the asphalt concrete mixture's resistance to crack propagation.

Experimental evidence has been obtained to state that the SBS modified mixtures have larger tensile strength and fracture toughness over the entire range of temperature used as compared to unmodified AC-5 mixtures. Also, SBS causes a significant increase in stiffness at high temperature and a significant decrease in stiffness at low temperature, therefore SBS is expected to improve the temperature susceptibility of asphalt concrete mixtures. Analysis of the critical energy release rate of SBS modified AC-5 mixtures at low temperature (35°F) suggests that the butadiene midblock introduces nonlinearity of the load deflection curve prior to the maximum load, indicative of the mixture's flexibility at low temperature.

The total accumulated dissipated energy through fatigue failure was introduced as a fracture criteria which accounts for the effect of both temperature and stress level. It was correlated to fatigue life using a power law regression analysis. Thus, the total accumulated dissipated energy through fatigue failure can be used as an alternative method to study fatigue, since it is successfully correlated to fatigue life through temperature independent parameters.

The effect of low temperature thermal cycling was also studied in Chapter 7. Relationships between parameters controlling the fatigue fracture process \( (C^*, m^*, y', \beta') \) and the number of thermal cycles were established. Thermal cycling tends to stiffen AC-5 asphalt concrete mixtures and in turn to reduce its fatigue resistance. This was
demonstrated by the lower value of $\gamma'$ and the higher values of $C^*$, $m^*$ and $\beta'$ at high levels of thermal cycling. This can be related to the thermal fatigue distress exceeding the fatigue resistance of the mixture. However, it was found that $C^*$, $m^*$ and $\gamma'$ for SBS modified mixtures remain almost constant up to 14 thermal cycles. Thus, the SBS modifier helps in maintaining a constant fatigue resistance of the mixture over a moderate number of low temperature thermal cycles. This can be attributed to the flexibility of the mixture induced by the SBS modifier. A decrease in the fatigue resistance was observed at higher number of thermal cycles. The fatigue performance of asphalt concrete mixture as predicted from Paris’ and the MCL models was found to be consistent with what was predicted from the indirect tensile strength, resilient modulus and critical energy release rate tests. However, Paris’ model in general seems to be more consistent in characterizing the resistance of asphalt concrete mixtures to fatigue crack propagation as compared to the MCL model. This was evident from the independence of Paris’ model parameters ($C^*$ and $m^*$) on temperature used. This was not the case for the MCL model which indicated that, the dissipative parameter $\beta'$ strongly depends on temperature.

8.2 Recommendations for Future Work

Building on the data base generated, a future research program is essential to study the effect of different gradations on the mechanical and fatigue performance of polymer modified asphalt concrete mixtures.
This research has been focused on the fatigue evaluation for asphalt concrete mixtures using stress controlled fatigue testing under a constant frequency of loading. It is of great importance to employ the current approach to characterize the resistance of various asphalt concrete mixtures under strain controlled fatigue loading. Also, the effect of loading frequency should be taken into consideration.

Also, it has been shown that the fracture resistance of the AC-20 mixture is considerably improved by using the dynamic/static compaction technique, so future research efforts are needed to optimize the dynamic compaction procedure to develop paving mixtures with higher resistance to cracking.

After the total accumulated dissipated energy through fatigue failure has introduced as a fracture criteria for asphalt concrete mixtures, more work has to be done accounting for service factors such as frequency and type of loading. Also, the mixed mode of loading has to be considered, since it would more closely simulate in-situ pavement conditions, rather than using an equivalent damage concept, to determine whether or not the same energy is required to failure when uniform or non-uniform loading cycles are applied.
Appendix A

Computer Programs Listings
Program No.1: GRAB

ftn7x,s
$cds on
$files 2,2

program get_digitized_data
  common /tr\%/ ratio
  real*4   x1h,y1h,wx,wy
  integer*2 ps,linetype,ncase
  character*30 file1

  c**** initialize devices ****
  c
  np = 0
  ncase = 0

  write(1,*), ' enter output file name'
  read(1,'(a)'), file1
  write(1,*), ' opening file ', file1
  open(unit=2, file=file1, status='unknown')

  write(1,*), ' enter y/x ratio'
  read(1,*), ratio

  c**** check if connection is ok ****
  c
  call zbgn
  call zbint(51,ierr)
  if (ierr.ne.0) write(1,*), ierr
  call zlint(51,ierr)
  if (ierr.ne.0) write(1,*), ierr
  call zvint(51,ierr)
  if (ierr.ne.0) write(1,*), ierr

  c
  The big loop
  c
  25 write(1,30)
  30 format(' press button 1 for page setup /
  &     2 for entering data/
  &     4 for exit/')

  ieco=1
  call zbtn(ieco,ibut)
  if (ibut.eq.1) go to 35
  if (ibut.eq.2) go to 60
  if (ibut.eq.4) go to 300
  go to 25

  c
  Initial the program: determine the parameters
  c
  35 write(1,*), 'enter initial coordinate value (x, y)'
  read(1,*), xint,yint
  call zvloc(xint,yint,wx,wy)
  c
  y1h = y1h*704./12032.
  call zdpmm(x1h,y1h,xmm,ymm)
  c
  write(1,*), 'Origin: ', x1h,y1h,xmm,ymm
x1h = xmm
y1h = ymm

write(1,*), ' enter x axis value ', read(1,*), 'xval
call zlloc(ieco, ibut, x2h, y2h)
c y2h = y2h*8704./12032.
call zdpmm2(x2h, y2h, xmm, ymm)
c write(1,*), 'X point: ', x2h, y2h, xmm, ymm
x2h = xmm
y2h = ymm

c**** calculate transformation matrix ****
c call setran(xint, yint, xval, x1h, y1h, x2h, y2h)
go to 25

c**** start reading in data ****
c 60 continue
70 call zlloc(ieco, ibut, wx, wy)
if (wy .gt. 1.001) then
go to 25
endif
np = np+1

c wy = wy*8704./12032.
call zdpmm2(wx, wy, xmm, ymm)
c write(1,*), 'Point: ', wx, wy, xmm, ymm
wx = xmm
wy = ymm

call tran(wx, wy, x1h, y1h)
write(1,100) np, wx, wy, x1h, y1h
100 format (i5, 4g15.5)
write(2,105) x1h, y1h
105 format(2g15.4)
go to 70

c**** exiting the program, shut down graphics ***
c 300 continue
close(unit=2)
call zlend
call zvend
call zend
write(1,*), 'done'
stop
end

subroutine setran(oxp, oyp, pxp, ox, oy, qx, qy)
real*8 c1, s1, c2, s2, a, b
real*8 dist, scalex, scaley, costheta, sintheta
common /tra/ ratio
common /tmat/ c1,s1,c2,s2,a,b

dist = SQRT((qx-ox)**2 + (qy-oy)**2)
scalex = dist/(pxp-oxp)
scaley = scalex/ratio
costheta = (qx-ox)/dist
sintheta = (qy-oy)/dist
c1 = costheta/scalex
s1 = sintheta/scalex
c2 = costheta/scaley
s2 = sintheta/scaley
a = -ox*c1 - oy*s2
b = ox*s1 - oy*c2

return
dend

subroutine tran (wx,wy,x1,y1)
real*4 wx,wy,x1,y1
real*8 c1,s1,c2,s2,a,b
common /tra/ ratio
common /tmat/ c1,s1,c2,s2,a,b

x1 = c1*wx - s1*wy + a
y1 = s2*wx + c2*wy + b

return
dend

subroutine zdpm2(wx,wy,xmm,ymm)
real*4 wx,wy,xmm,ymm

convert from world coordinates to device coordinates for 9111a

wxmin = -1.
wymin = -1.
xmax = +1.
ymax = +1.
xmin = 0.0
ymin = 0.0
vxmax = 300.8
vymin = 217.6

xmm = ((vxmax-vxmin)/(vxmax-wxmin))*{wx-wxmin} + vxmin
ymm = ((ymax-vxmin)/(ymax-wxmin))*{wy-wxmin} + vmin

return
dend
10 DIM PTSX(3000), PTSY(3000)
20 PRINT: PRINT: INPUT "FILENAME TO BE ANALYZED "; ALTS
25 LPRINT "FILE ANALYZED "; ALTS
30 OPEN ALTS FOR INPUT AS #2
40 N=0
50 INPUT #2, PTSX(N), PTSY(N)
60 IF EOF(2) THEN 90
70 N=N+1
80 GOTO 50
90 N=1
100 PTSX(N)=PTSX(0)
110 PTSY(N)=PTSY(0)
120 IN=N-1
130 AREA=0
140 N=0
145 MAXX=0
146 MAXY=0
150 AREA=AREA+.5*(PTSY(N)*(PTSX(IN)-PTSX(IN+1)))
160 FOR N=1 TO IN-1
161 IF PTSY(N)<MAXY THEN 165
162 MMAY=N
163 MAXY=PTSY(N)
164 IF PTSY(N)<MAXX THEN 170
166 MMAX=N
167 MAXX=PTSX(N)
170 AREA=AREA+.5*(PTSY(N)*(PTSX(N+1)-PTSX(N-1)))
180 NEXT N
190 N=IN
200 AREA=AREA+.5*(PTSY(N)*(PTSX(0)-PTSX(IN-1)))
205 PRINT: PRINT
210 PRINT "AREA OF LOOP (lb-in) "; AREA
211 LPRINT "AREA OF LOOP (lb-in) "; AREA
215 ALOAD=0
220 FOR I=1 TO IN-1
230 IF I>MMAX THEN 400
240 ALOAD=ALOAD+(PTSX(I-1)-PTSX(I))*(PTSY(I)-PTSY(I-1))/2
260 NEXT I
400 PRINT "AREA UNDER LOADING CURVE (lb-in) "; ALOAD
410 LPRINT "AREA UNDER LOADING CURVE (lb-in) "; ALOAD
420 UNLOAD=ALOAD-AREA
430 PRINT "AREA UNDER UNLOADING CURVE (lb-in) "; UNLOAD
440 LPRINT "AREA UNDER UNLOADING CURVE (lb-in) "; UNLOAD
460 ABOX=PTSY(MMAX)*PTSX(MMAX)
470 PRINT "AREA of box to max load (lb-in) "; ABOX
480 ATOP=ABOX+UNLOAD
500 PRINT "AREA above unloading curve (lb-in) "; ATOP
501 LPRINT "AREA above unloading curve (lb-in) "; ATOP
502 LPRINT
503 END
1000 END
Program No. 3: MIN

10 DIM X(500), Y(500)
20 PRINT: PRINT: INPUT "FILENAME TO BE ANALYZED "; ALTTS
30 OPEN ALTTS FOR INPUT AS #1
40 N = 0
50 XMIN = 10000
60 YMIN = 10000
70 INPUT #1, X(N), Y(N)
80 IF EOF(1) THEN 140
90 IF N = 0 THEN 120
100 IF X(N) < XMIN THEN XMIN = X(N)
110 IF Y(N) < YMIN THEN YMIN = Y(N)
120 N = N + 1
130 GOTO 70
140 PRINT: PRINT: INPUT "FILENAME TO STORE PROCESSED DATA "; PS
150 IF X(N) < XMIN THEN XMIN = X(N)
160 IF Y(N) < YMIN THEN YMIN = Y(N)
170 OPEN PS FOR OUTPUT AS #2
180 FOR I = 1 TO N
190 X(I) = X(I) - XMIN
200 Y(I) = Y(I) - YMIN
210 PRINT #2, X(I), Y(I)
220 NEXT I
230 END
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