FATIGUE-FRACTURE RELATION ON ASPHALT CONCRETE MIXTURES

BY

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THESIS

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ABSTRACT

The purpose of this study is to examine potential relationships between fatigue and fracture parameters obtained from standardized laboratory tests. A flexural bending beam test was used to obtain two fatigue parameters, which include a traditional fatigue criterion based on stress or strain to find number of cycles to failure and a new fatigue criterion based on an energy approach known as the plateau value (PV). Similarly, a disk-shaped compact tension DC(T) fracture test was used to obtain fracture energy parameters, including: fracture strength, pre-peak fracture energy, post-peak fracture energy, and total fracture energy. A factorial of the eight possible correlations was computed to determine the strongest association between the fatigue and fracture parameters for the asphalt mixtures investigated. The study was motivated by the desire to investigate the feasibility of predicting time-consuming fatigue test results with fracture test results with fracture test results, which can be obtained much more rapidly.

Based on the results obtained in this study, it was shown that a potentially strong correlation exists between fatigue and fracture mechanisms in asphalt concrete, as characterized by parameters associated with dissipated or consumed energy. As presented through the statistical analysis, the plateau value (PV) and the pre-peak fracture energy ($G_{f\text{-pre}}$) are the most highly correlated parameters from the fatigue and fracture tests, respectively. This study is based upon limited experimental data and is explored an initial starting point to find relationships between fatigue and fracture mechanisms. Much more experimented and analytical work will be needed to fully understand these relationships and to develop a standardized interconversion scheme.
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Chapter 1 Introduction

The fatigue behavior of Hot Mix Asphalt (HMA) pavements has been studied a number of decades; however, it is still a poorly understand phenomenon. Therefore, recent studies have been developed to improve the correlation of fatigue results to natural fatigue behavior occurring in HMA pavements. The ultimate goal of most fatigue studies is to investigate and predict fatigue behavior in pavements so as to prevent this form of distresses, which can induce serious structural damage. This research was conducted to explore a potential alternative to predicting fatigue behavior in HMA mixtures through a fracture testing and statistical correlation. This chapter provides an overview of fatigue and fracture occurring in HMA pavements. In addition, the motivation of the study is discussed.

1.1 Fatigue Characteristics in Asphalt Concrete Pavements

Fatigue cracking, in the perspective of asphalt pavements, is defined as the accumulation of damage under repeated load applications (Pell 1962). As a vehicle passes over a point within a pavement, tensile stress is generated at the bottom of the HMA layer. If the tensile stress exceeds the local tensile capacity of the material, damage and eventually cracking will occur. These cracks will gradually propagate to the surface of the pavement. However, in thick pavements (Uhlmeyer 2000), it was found that the highest tensile stress occurs near the top of the pavement which results in crack generation within the top layer of the pavement. After crack initiation, the cracks propagate down words possibly to the bottom of the pavement structure. This phenomenon is known as top-down cracking.

Fatigue performance of asphalt concrete pavements is commonly evaluated through laboratory tests performed on asphalt concrete test specimens. In terms of the experimental approaches to HMA fatigue testing. Three main types are common; (1) the phenomenological approach; (2) the energy-based approach, and; (3) the fracture mechanics approach. The phenomenological approach studies the relationship between repeated stress or strain in the test specimen and the number of load cycles to failure. This approach, which is based on the stress-number of cycles to failure (Pell 1962), is based on the concept of Miner’s linear law associated with cumulative damage in pavement (Miner 1945). For example, Figure 1.1 shows a general fatigue curve, plotting strain (or stress) against load repetitions on a log-log scale. The energy-
based approach uses the concept of the dissipated energy to evaluate fatigue behavior. This approach assumes that fatigue damage is a depletion of dissipated energy from one load cycle to the next (Carpenter 1997). This approach will be discussed in detail in a later chapter. Finally, the fracture approach, which is based on the concepts of fracture mechanics, studies the development and propagation of cracks. The stages of crack growth are typically divided into three stages known as crack initiation, propagation, and unstable fracture (K. K. Majidzadeh 1971). This approach tries to relate a characteristic of the crack propagation in the pavement to fracture parameters such as the stress intensity factor ($K_{IC}$).

![Image of Load Repetitions to Failure vs. Strain](Load Repetitions to Failure vs. Strain.png)

Figure 1.1 Typical Fatigue Curve (Carpenter, 2003)
1.2 Fracture Characteristics in Asphalt Concrete Pavements

In general, fatigue tests are based upon the assumption that the continuum theory holds true. This theory assumes material has homogeneous properties. However, this assumption is violated whenever a crack forms in the material; i.e., it be considered a discontinuous material. In the early 1900’s, fracture mechanics was introduced in order to study discontinuities in materials (Bazant 1998). Bazant introduced fracture mechanics in the study of cracked pavements.

Fracture mechanics was initially applied to asphalt concrete pavements in the early 1970’s (Majidzadeh 1975). The goal of his study was to relate the crack growth rate to the stress intensity factor in conjunction with Paris’ law (Paris 1963), where the number of cycles to failure can be related to stress intensity factor through the following equation:

\[
N_f = \int_{C_0}^{C_f} \frac{1}{AK^n} \, dc
\]

where
- \(N_f\) = the number of cycles to failure,
- \(C_0\) = the starting flaw,
- \(C_f\) = the final crack length,
- \(A\) and \(n\) = material parameters, and
- \(K\) = stress intensity factor (in N/mm\(^{0.5}\)).

The crack length used in the above equation was obtained by conducting a simple beam test under cyclic loading. The stress intensity factor was then determined using the crack length (Majidzadeh 1971) in order to predict the fatigue life in asphalt concrete pavements.

Later, another fracture parameter was introduced to describe asphalt concrete fracture behavior; namely fracture energy (\(G_f\)), which can also be obtained from a fracture test. Fracture energy can be explained as the amount of work required to generate a new surface or crack of unit length (Bazant 1998). This parameter has been used in conjunction with a cohesive zone fracture model to describe the fracture behavior of asphalt concrete (Paulino 2004). The cohesive zone model has the ability to describe softening type damage response in a fracture process zone located a head of a crack tip.
A single-edge notched beam, one of the early fracture tests, is commonly used to measure fracture energy. An advantage of the test is that the beam size can be easily adjusted to obtain proper fracture surface area (Wagoner 2005a). However, a limitation of this test is that the testing specimen is not easily obtained from field cores. Later, Wagoner and Buttlar (2005b) developed a new fracture test, called the disk-shaped compact tension DC (T) test. One of the reasons behind this development is that most typical laboratory compaction techniques usually produce cylindrically shaped specimens. In addition, cored field samples are also cylindrical. Fracture energy is typically measured using the DC (T) test. The fracture energy is defined as the area under a curve of the plot between a load and crack mouth opening displacement (CMOD) divided by the fractured area as shown in Figure 1.2.

![Figure 1.2 Determination of Fracture Energy](image)

**Figure 1.2 Determination of Fracture Energy**

1.3 Problem Statement

Fatigue cracking is considered to be a primary source of structural distress in asphalt concrete pavements and has been a common research topic for many pavement engineers. A better prediction of fatigue life pavement will help to improve pavement design procedures. Typically, a fatigue failure is defined as a phenomenon in which a pavement subjected to repeated loads undergoes cumulative damage within the pavement structure. When an underlying material is unable to bear the applied loads, the flaw or cracks are initially started. However, current fatigue analysis approaches have a limitation in that they cannot provide an explicit
description of the asphalt concrete pavement behavior. In turn, this cannot be taken as fundamental knowledge relating the material properties to loading effects. In addition, traditional fatigue tests are time-consuming and often have poor repeatability. Therefore, the current approaches need to be improved to better represent the actual fatigue characteristics of the pavement in a practical manner.

There have been efforts to use fracture mechanics to describe HMA fatigue behavior (K. e. Majidzadeh 1975; Ramsamooj 1991). Initially, crack growth was related to the stress intensity factor to predict a number of load cycles to failure in fatigue life. However, this approach was based on the Paris’s law (Paris 1963) which describes the crack growth utilizing only the stress intensity factor. Consequently, it was not sufficient and more terms were required to describe the fatigue mechanism in asphalt concrete pavement. A fracture approach is believed to be a better method to describe and predict fatigue behavior in the pavement. In regard to this consideration, this study will examine the correlation between fatigue and fracture testing parameters. This study is designed to find an initial relationship between these parameters to serve as a starting point for advanced research interconnecting both mechanisms.

1.4 Objectives of Study

The objectives of this thesis were:

1. To determine and analyze fatigue parameters used to produce correlations to fracture characteristics of the asphalt concrete mixtures corresponding to different mix properties such as asphalt type, asphalt content, and air void level;

2. To determine and analyze fracture parameters of the mixtures which are identical in composition as those used for the fatigue analysis; and

3. To examine statistical relationships between fatigue and fracture parameters based upon their laboratory testing results in order to explore the feasibility of using simpler fracture tests as a surrogate for fatigue tests.
1.5 **Scope of Study**

The mixtures used in this study were in the loose state and obtained from a study exploring the effect of reclaimed asphalt pavement (RAP) on fatigue performance. Five different mixtures with varying mix properties such as binder type, percentage of binder content, and aggregate structure were used in the study. The four-point bending beam fatigue test (AASHTO T321) and DC(T) fracture test (ASTM D73B-076) were utilized to investigate the laboratory performance of the HMA mixtures. The following are the main procedures utilized during the study:

1. Review previous works related to both fatigue and fracture mechanisms;
2. Analyze fatigue data obtained from another study as mentioned above;
3. Fabricate fracture test specimens from Superpave Gyratory Compactor (SGC) produced, and determine fracture parameters from testing results;
4. Use the analyzed data in steps 2 and 3 to compute correlations between fatigue and fracture parameters through statistical analysis; and
5. Finally, make conclusions and recommendations for further studies.

1.6 **Organization of Thesis**

CHAPTER ONE: propose an introduction of the research containing the problem statement, objective and scope of study, and the organization of the thesis.

CHAPTER TWO: discuss the literature review findings, including an introduction of fatigue testing of asphalt mixtures and how to evaluate fatigue performance from a laboratory testing result. In addition, fracture testing of asphalt mixtures and fracture characteristics based on testing results. Finally, address previous work associated with both fatigue and fracture tests.

CHAPTER THREE: explain the experimental design of the study, including a plan of the study, material selections, and testing procedures to evaluate the asphalt mixtures.

CHAPTER FOUR: present testing results of both fatigue and fracture tests, and examine relationships between fatigue and fracture parameters.

CHAPTER FIVE: conclude the findings of the study, make recommendations based on the results, finally, and propose further research.
Chapter 2 Literature Review

This literature review presents preceding information associated with fatigue and fracture works of asphalt concrete materials. It mainly focuses on two primary topics: fatigue behavior and fracture behavior. Particularly, in fatigue review, general behavior, current approach, and various effects on fatigue performance will be discussed. On the other hand, fracture mechanics and its application of recent works to predict fatigue behavior will be reviewed in this section.

2.1. Review of Fatigue Behavior

Fatigue cracking is generally known as one of the major distresses in flexible pavements. Several potential causes lead to this type of cracking such as material selection, poor construction procedure, environmental conditions, and pavement subjected to unexpectedly rapid growth of the traffic. A typical feature of fatigue cracking is interconnected cracks on pavement surfaces. This is sometimes called alligator cracking because of its appearance.

Fatigue cracking is defined as the accumulation of damage under repeated load applications in an asphalt pavement (Pell 1962). A tensile stress is generated at the bottom of the HMA layer in the pavement structure. If the tensile stress is greater than the strength of the material, a flaw is formed initially. A crack starts gradually propagating to the surface of the pavement. However, in another case in thick pavements (Uhlmeyer 2000), the highest tensile stress happens on top of the pavement resulting in cracks are generated from the top to bottom of the pavement layer. They then propagate down to the bottom of the pavement structure, which is also called top-down cracking.

2.1.1 Evaluation of Fatigue Life in an Asphalt Concrete Mixture

Fatigue performance of the flexible pavement is commonly used in investigations using a correlation of its performance from the laboratory testing. Experimental approaches, commonly used to inspect its behavior in the laboratory, can be primarily divided into three types: the phenomenological approach, the energy-based approach, and the fracture mechanics approach. This section will only mention the first two approaches. The fracture mechanics approach will be discussed in more focus in the next section.

In the phenomenological approach, the fatigue performance is shown as the relationship between the stress or strain in an asphalt mixture and the number of load repetitions. (Pell 1962).
Pell defined a fatigue life based upon the relation between the stress or strain and the number of cycles to failure. It employed the concept of Miner’s linear law of cumulative damage (Miner 1945). Typically, a traditional fatigue curve is shown by plotting a relation between the stress or strain at the bottom of the asphalt concrete layer against number of load cycles to the 50 percent reduction in initial stiffness on a log-log scale as shown in Figure 2.1.

![Figure 2.1 Traditional Fatigue Curve](image)

A traditional fatigue analysis is based on the initial value of the stress or strain and the number of load cycles to failure. Failure was defined as the number of load cycles to 50% reduction in initial stiffness. Based on this relationship, there were several fatigue models developed to predict fatigue cracking. Generally, fatigue models are divided into two main types: the strain-based models and the strain modulus-based models. Laboratory testing indicates there is a relation between the strain at the bottom of the asphalt concrete layer and the number of load applications in the pavement. This relation can be expressed as the following:

$$N_f = K \left( \frac{1}{\varepsilon} \right)^a$$  \hspace{1cm} (2.1)

where  \( N_f \) = number of load repetitions to cracking,

\( \varepsilon \) = predicted asphalt concrete strain (mm/mm),

and  \( K \) and \( a \) = factors depending on the composition and properties of the AC mixture.
The energy-based approach uses the concept of the dissipated energy to evaluate a fatigue performance. The concept behind this approach is that when a material is subjected to cyclic loading, it will accumulate damage. Therefore, this damage can be defined as the deterioration, which occurs in the material before failure. When a load is applied to a material there will be a stress that induces a strain. The area under the stress-strain curve represents the energy being put in to the material as shown in Figure 2.2.

![Stress-Strain Hysteresis Loop (Flexural Fatigue Testing) (Carpenter 1997)](image)

**Figure 2.2 Stress-Strain Hysteresis Loop (Flexural Fatigue Testing) (Carpenter 1997)**

The dissipated energy in a linear viscoelastic material for a flexural fatigue test is calculated using the following equation:

$$ W_i = \pi \sigma_i \varepsilon_i \sin \phi_i $$

(2.2)

where

- $W_i$ = Dissipated energy at load cycle $i$,
- $\sigma_i$ = stress amplitude at load cycle $i$,
- $\varepsilon_i$ = strain amplitude at load cycle $i$,
- $\phi_i$ = phase angle between the stress and strain wave signals.

The dissipated energy during each loading cycle affects the strain level of the mixture which leads to the assumption that the fatigue life can be predicted as an accumulation of dissipated energy from one load cycle to the next (Van Dijk 1977). In later studies, it was discovered that not all dissipated energy in the fatigue test up to failure is assumed to do damage
to the material. It is the change in dissipated energy that is responsible for damage (Carpenter 1997). This relation was found to be independent of test conditions and mode of loading.

A damage curve (Carpenter 1997) that relates the percentage of dissipated energy producing damage to the material under cyclic loading was provided based on the concept of change in dissipated energy. The constant value of percentage of dissipated energy that produces damage to the material under cyclic loading was defined as the Plateau Value (PV). It was found that the PV is highly dependent on the initial loading conditions, stress, strain, and dissipated energy. The plateau value represents the period during which the percentage of dissipated energy going into damage is constant for each load cycle. Plotting the constant plateau values of the dissipated energy ratio ($\Delta \text{DE}/\text{DE}$) and the number of cycles at the failure point on the traditional log-log scale will provide a straight line similar to the traditional fatigue curve as shown in Figure 2.3.

![Figure 2.3 Plateau Value of DER vs. Number of Load Cycles (Ghuzlan, 2001)](image)

Therefore, the failure criteria of fatigue performance can be predicted based upon the dissipated energy approach with the following equation:

$$N_f = C(PV)^b$$  \hspace{1cm} (2.3)

where $N_f$ = number of load repetitions to cracking.
PV = plateau value of dissipated ratio (ΔDE/DE),
and C and b = factors depending on the composition and properties of the AC mixture.

2.1.2 Varieties of Mixtures Affecting Fatigue Performance

Different types of mixtures or compositions can provide different fatigue performances. A variation in components of the mixture can be: asphalt content, asphalt type, asphalt volume, aggregate type, aggregate gradation, and air voids. Generally, fatigue behavior of samples produced in a laboratory is mostly affected due to the change of asphalt content and air void content. Several researchers’ results show that the increase of mixture stiffness is a result of increasing the asphalt content (Jimenez 1962). It also indicates that the more air voids in the mixture, the less fatigue life (Bazin 1967).

The Effect of Asphalt Type

Several works of preceding research show the asphalt type effects on fatigue performance. Two different grades of asphalt were tested under a controlled stress mode at different temperatures (P. Pell 1967). The two different asphalts were 90-100 penetration bitumen and 40-50 penetration bitumen. As a result, the fatigue performances are different in such a way that the higher penetration yields a shorter fatigue life. In other words, mixes with lower penetration have a less steep fatigue line than the one made with higher penetration. Moreover, Epps (1969) performed the fatigue test with three different grades of asphalt: 85-100, 60-70, and 40-50. The mixes were made with an asphalt content of 6 percent. He found that the mix made with stiffer binder had the higher stiffness.

Pell (1975) tested the mixes with different grades and asphalt contents under a controlled stress testing mode. He claimed that binder type is a main factor that influences fatigue performance. For hotter weather with a thicker pavement, it was suggested to use stiffer asphalt cements to have a better fatigue performance (Myre 1990). He conducted an experiment to investigate the influence of binder grade on fatigue performance. Three types of binder grade were used to make mixtures. They were tested with two different modes of loading: controlled stress and controlled strain. He concluded that the mixes with stiffer binder provided better fatigue performance within controlled strain fatigue testing.
The Effect of Asphalt Content

Some previous research (Jimenez 1962) showed that a mixture made at the optimum asphalt content gave the highest fatigue life. Monismith (1969) claimed that as asphalt content increases, the fatigue life increases. In addition, most results of these researchers indicated that the tests were done within controlled stress mode. However, Myre (1990) tested different mixes made with different asphalt binder contents based on controlled strain fatigue testing, and concluded that there were no significant differences in fatigue performance of the mixes. Figure 2.4 shows the effect of different asphalt contents with different surface texture of aggregate on fatigue life.

![Figure 2.4 The Effect of Asphalt Content on Fatigue Life (Jimenez, 1962)](image)

The Effect of Air Voids

One of the mix variables affecting fatigue performance is the difference in air void content of the mixture. Several researchers showed that the decrease of fatigue life was a result of increasing air void content. Moreover, Santucci (1969) conducted the fatigue test based on controlled strain mode of loading. He found that the higher air void content, the shorter fatigue life. Raithby and Ramshaw (1972) investigated the effect of compaction on fatigue performance. The result showed that the fatigue life is improved by reduction in void content. In other words, higher air void content produces mixtures with shorter fatigue life.
The Effect of Aggregate Types and Gradation

Epps and Monismith (1971) had an expectation of the effects on fatigue behavior of mixtures with different aggregate surface textures. The hypotheses were that angular aggregates with rough surface texture and open gradation were expected to be difficult to compact resulting in higher air void content, and rough aggregate, on the other hand, may be easier to compact and thus having lower air void content. Pell and Cooper (1975) claimed using more rounded gravel gave a longer fatigue life than crushed rock. It was concluded that this effect on fatigue performance was accounted for by the mixes made with the round aggregate.

Fatigue performance is also affected using different types of aggregate. Bazin and Saunier (1967) evaluated the fatigue behavior based on several types of aggregate: a sand mixture, rolled lean mix, and crushed dense mix. They concluded that the type of aggregate plays an important role in the fatigue behavior of the mixes as shown in Table 2.1.

Table 2.1 Factors affecting Mixture Stiffness and Fatigue Life (Epps and Monismith, 1971)

<table>
<thead>
<tr>
<th>Factor</th>
<th>Change in Factor</th>
<th>Effect of Change in Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>On Stiffness</td>
</tr>
<tr>
<td>Asphalt Type</td>
<td>Increase</td>
<td>Increase</td>
</tr>
<tr>
<td>Asphalt Content</td>
<td>Increase</td>
<td>Increase</td>
</tr>
<tr>
<td>Air Void</td>
<td>Decrease</td>
<td>Increase</td>
</tr>
<tr>
<td>Aggregate Type</td>
<td>Increase angularity</td>
<td>Increase</td>
</tr>
<tr>
<td>Aggregate Gradation</td>
<td>Open-Grade</td>
<td>Increase</td>
</tr>
<tr>
<td>Temperature</td>
<td>Decrease</td>
<td>Increase</td>
</tr>
</tbody>
</table>

The influence of aggregate gradation was investigated by several researchers. Monismith and Deacon (1969) concluded that in controlled stress fatigue testing, the dense-graded mixtures provided longer fatigue life than the open-graded mixtures. Epps and Monismith (1971) mentioned it is difficult to evaluate the effect of aggregate type or gradation, since using different aggregate gradations result in different optimum asphalt content and air voids level. However, Pell and Taylor (1969) drew the conclusion that fatigue life was not significantly affected by using different types and gradations.
2.2. Review of Fracture Behavior

Generally, pavement performances are evaluated using experimental results from laboratories. However, typical tests are based on an assumption of a continuum theory, which is that testing materials have homogeneous properties. Whenever the material starts having a flaw or crack, it has been considered to be a discontinue material. The assumptions are unacceptable. Therefore, fracture mechanics was introduced to study a discontinuity of the materials. In the early 1900’s, it was the first fracture mechanics was used in order to investigate a crack developing in a pavement layer (Bazant 1998). The concept of fracture mechanics can explain the cracking mechanism: crack initiation and propagation until reaching failure.

As mentioned above, the main purpose of fracture mechanics is to better understand the occurrence and development of a crack in a pavement. Many researchers have studied the crack growth rate, employing Paris’ law (Paris 1963). Paris’ law can be expressed as:

\[
N_f = \int_{C_o}^{C_f} \frac{1}{AK^n} \, dc
\]

(2.4)

where \( N_f \) = the number of cycles to failure,
\( C_o \) = the starter flaw,
\( C_f \) = the final crack length,
\( A, n \) = material parameters,
And \( K \) = stress intensity factor (in N/mm\(^{1.5}\)).

Theoretically, fracture behavior was able to evaluated using linear elastic fracture mechanics (LEFM) or nonlinear fracture mechanics. The linear elastic fracture mechanics approach provides a fundamental background for understanding fracture mechanics (Anderson 1995). However, the application of LEFM may not provide the most accurate results based on the response of an asphalt concrete pavement. Generally, bituminous material exhibits a softening curve after the peak load instead of a brittle failure which is associated with LEFM materials. Therefore, nonlinear fracture mechanics was introduced to study AC mixtures. The following paragraphs will review main concepts of linear elastic and nonlinear fracture mechanics which are used to apply to the study fatigue cracking behavior, including recent works of several researchers using both approaches.
LEFM can be described by two methods: energy approach and stress intensity approach. Although these methods approach crack formation differently, the solutions are equal. To predict fatigue resistant performance in HMA, most researchers used stress intensity approach to study its mechanics. Stress field at the crack tip is proposed analytical solutions. An important concept of this approach is that the stress fields can be described by a stress intensity factor (K). Using the stress intensity approach, the failure of the specimen can be defined by the stress intensity factor at that load, denoted as \( K_C \).

The stress intensity approach was employed to evaluate asphalt concrete mixtures to predict fatigue life by the early 1970’s (K. K. Majidzadeh 1971). Seven different mix designs were used to verify fracture parameters. A simple fatigue beam test was conducted under cyclic loading within a dynamic Havensine load function of 5 Hz frequency. Crack propagation under applied stress and the corresponding number of fatigue cycles were observed and recorded. In addition, the crack lengths on both sides of the test beam were measured and recorded. The evaluation of the stress intensity parameter, or fracture toughness, \( K_{IC} \), was evaluated at various stress rate applications. Evaluation parameters for fatigue response include number of fatigue cycles to failure (\( N_f \)), and parameter A, n in the Power law equation which is mentioned in the previous section as \( \frac{dc}{dN} = AK^n \).

Later, Ramsamooj (1991) conducted his laboratory test to make a prediction of fatigue life of asphalt concrete beams from fracture tests. The principles of fracture mechanics were utilized and covered a full range of loading from low to high. More than 46 different mixtures, consisting of various types of asphalt and several gradations, were used to determine critical stress-intensity factor, and fatigue life of asphalt concrete beams. The relation of fatigue cycles to failure and stress-intensity factor under any configuration of loading conditions was expressed in his research as the following:

\[
\frac{\Delta c}{\Delta N} = \frac{1}{24K_{IC}^2 \sigma^2} (\Delta K_i^4 - K_o)
\]

(2.5)

where \( \Delta K_i \) is the applied stress-intensity factor range.
Based on his results, the conclusion was drawn that the fatigue life of several types of asphalt concrete mixtures can be predicted from simple fracture tests. Statistical analysis indicates no significant difference between the experimental data and the theoretical predictions.

However, the application of linear elastic fracture mechanics for asphalt concrete is unreasonable because of the large fracture process zone (inelastic zone) around the crack tip. In addition, one of the researcher’s works (Cotterell 1996) showed that the application of LEFM may not yield accurate results for asphalt concrete materials because it exhibits a softening curve after the peak load instead of a brittle failure as shown in Figure 2.5. The softening curve occurs because the material has the ability to carry load due to aggregate bridging and interlocking as the crack propagates. Therefore, they utilized non-linear elastic fracture mechanics to study HMA mixtures and claimed that it is more appropriate to better understand the fracture mechanics of asphalt concrete material. For nonlinear fracture mechanics, two parameters, J contour integral (J-integral) and fracture energy (G_f), were considered to characterize the fracture behavior. A following section will review the basis of these approaches and their applications for a prediction of fatigue performance.

![Figure 2.5 Illustration of Softening Part in Asphalt Concrete Material (Bazant, 1995)](image)

The first parameter, the J contour integral (J-integral), was identified to potentially describe the fracture characteristics of asphalt concrete. The J-integral, developed by Rice (1968), is a path independent line integral that characterizes the energy release rate for nonlinear
materials, and is measured by the plastic strain field of the elastic-plastic materials. The energy can be experimentally determined as the area under the load-displacement curve for the different notch lengths. The energy based on this approach can be calculated by the following equation:

\[ J = \int_{r}^{r_1} [U_d(a, \varepsilon)dy - T \frac{\partial u}{\partial x} dx] \]  

(2.6)

Recently, the J-integral concept has been successfully used by several researches to characterize crack growth under plastic deformations. El Haddad et al. (1979) have investigated short cracks in the vicinity of notches. Moreover, Sehitoglu (1981) used the J-integral approach to analyze elastic-plastic mechanics of notched members. The J-integral estimates are made to characterize cracks growing in the vicinity of notches. The crack growth rate and the range in the J-integral (\( \Delta J \)) are related through a power law. Therefore, the crack propagation life can be estimated using the elastic-plastic fracture mechanics, or J-integral approach. The calculation is done by integrating the crack growth rate equation, which related \( da/dN \), crack growth rate, and \( \Delta J \) by a power law:

\[ N_p = \int_{l_i}^{l_f} \frac{dt}{C(\Delta J)^m} \]  

(2.7)

Crack initiation length, \( l_i \) is defined as a crack size similar to \( l_f \). According to his laboratory results, fatigue life prediction based on the J-integral method was valid at all stress levels. However, later study of Shah et al (1995) claimed that the application of the J-integral may be questionable due to the large inelastic zone, which cannot account for the selected local contour, \( r \).

The second parameter in nonlinear fracture mechanics is fracture energy. The fracture energy (\( G_f \)) can be defined as the amount of work to create a new surface or crack of unit length (Bazant 1998). It was used in conjunction with a cohesive zone model to describe the fracture behavior of asphalt concrete (Paulino 2004), since the cohesive zone model (CZM) is believed to be descriptive of the softening response of the fracture zone, which is located at a crack tip due to aggregate. The CZM uses three material properties to describe the cracking process: critical stress, critical crack tip opening, and fracture energy. Based on a review of nonlinear fracture mechanics, the fracture energy approach was selected as the most appropriate parameter to describe asphalt concrete fracture. Fracture energy accounts for the large fracture process zone.
Fracture energy can be determined by calculated an area under a plot of the load vs. displacement divided by a fracture surface as defined by the following equation:

\[ G_F = \frac{A_f}{BL} \]  
\[ (2.8) \]

where  
\( G_F \) = Fracture energy (J/m²),  
\( A_f \) = Area under Load-CMOD curve (kNmm),  
\( B \) = Thickness of specimen (mm),  
and  
\( L \) = Ligament length (mm).

There are several fatigue tests used to extract the fracture energy in a laboratory. The most common is the single-edge notched beam or SE(B) fracture test. The SE(B) geometry was utilized to determine fracture energy because of the stress states of a simply supported beam are relatively simple. Moreover, the beam size can be adjusted for a larger fracture area. Figure 2.6 Shows typical dimensions of SE(B) geometry used in common fatigue testing in accordance with ASTM E399 [2002].

However, the SE(B) geometry has shortcomings in its application. Firstly, beam configurations are not fabricated during HMA mixture design; therefore, it is a required additional step to determine fracture energy. Secondly, this geometry also could not be obtained from a field pavement because of limitation of coring machines. Lastly, a large amount of material is required to make the beam configuration.

![Figure 2.6 Single-Edge Notched Beam Geometry (Wagoner and Buttlar, 2005a)](image)

Later, researchers Wagoner and Buttlar (2005b) developed disk-shaped compact tension DC(T) geometry for determining the fracture energy. One reason behind this development is to
make fracture testing practical. A cylindrical-shaped specimen can be fabricated during the mixture procedure using typical gyratory compaction; therefore, a fracture test can be incorporated into the asphalt concrete mixture design process without an additional step. The DC(T) geometry is shown Figure 2.7 in accordance with the ASTM E 399.

Figure 2.7 DC(T) Geometry Dimensions (Wagoner, 2005b)

From their findings, fracture energy determined with the DC(T) test showed similar trends as the SE(B) test, where fracture energy is influenced by the test temperature and asphalt binder type. Therefore, the DC(T) geometry would utilize fundamental fracture testing and determine fracture parameters for asphalt concrete mixture.
Chapter 3 Experimentation

3.1 Experimental Plan

In this particular study, fatigue and fracture data were analyzed and a comparison was to examine their correlations. Every test involved in the study has been done at the Advanced Transportation Research and Engineering Laboratory (ATREL) at the University of Illinois. All fatigue testing data was obtained from the study of RAP material’s effects on fatigue performance as mentioned in the scope of the study. Five mixtures from the fatigue study were used to test and determine fracture parameters utilizing the Disk-shaped compact tension (DC(T)) test. Both fatigue and fracture parameters were obtained from the test data. The analysis procedure of the data will be discussed in Chapter 4.

In this Chapter, the contents were divided into three main sections: materials, specimen preparation, and testing procedures as shown below:

3.2 Materials

As stated previously, all material used in this study was received from the study of RAP material’s effects on fatigue performance. Five different mixtures were selected to represent variables of asphalt concrete mixtures. These mixtures are typical asphalt concrete mixtures used in different parts of the state of Illinois. The mixtures being investigated in this study provide a wide range of mix-design properties such as various asphalt contents, asphalt types, and levels of air voids, to account for differences in pavement performances corresponding to various mixture properties. Moreover, all mixtures included reclaimed asphalt pavement (RAP) aggregate.

Aggregate gradation is also important factor that affects mixture properties. In this study, five differently virgin aggregates: limestone (CM11), dolomite (CM16), crushed dolomite (FM20), and natural sand (FM02) were used. In addition, different sizes of RAP were included in the mix designs. Graditions for all mixtures used in this study are listed below in Table 3.1.
Table 3.1 Gradation for All Mixtures Using in this Study

<table>
<thead>
<tr>
<th>Sieve No. (mm)</th>
<th>MIX IDENTIFICATION (from Fatigue RAP Project)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DAVE</td>
</tr>
<tr>
<td>25</td>
<td>100.0</td>
</tr>
<tr>
<td>19</td>
<td>95.3</td>
</tr>
<tr>
<td>12.5</td>
<td>78.5</td>
</tr>
<tr>
<td>9.5</td>
<td>67.8</td>
</tr>
<tr>
<td>4.75</td>
<td>41.2</td>
</tr>
<tr>
<td>2.36</td>
<td>25.6</td>
</tr>
<tr>
<td>1.18</td>
<td>18.5</td>
</tr>
<tr>
<td>0.6</td>
<td>13.9</td>
</tr>
<tr>
<td>0.3</td>
<td>7.9</td>
</tr>
<tr>
<td>0.15</td>
<td>5.6</td>
</tr>
<tr>
<td>0.075</td>
<td>4.5</td>
</tr>
</tbody>
</table>

The physical properties including asphalt type, asphalt content, maximum theoretical specific gravity of the asphalt concrete mixture ($G_{mm}$), and bulk specific gravity of aggregate are shown in Table 3.2.

Table 3.2 Physical Properties for Mixtures Using in This Study

<table>
<thead>
<tr>
<th>Mix ID</th>
<th>Binder Type</th>
<th>Modified Binder</th>
<th>Asphalt Content (%)</th>
<th>% RAP</th>
<th>$G_{mm}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>DAVE</td>
<td>PG 58-22</td>
<td>No</td>
<td>4.5</td>
<td>30%</td>
<td>2.498</td>
</tr>
<tr>
<td>STAN</td>
<td>PG 64-22</td>
<td>No</td>
<td>4.5</td>
<td>10%</td>
<td>2.486</td>
</tr>
<tr>
<td>BILL</td>
<td>PG 58-28</td>
<td>No</td>
<td>4.7</td>
<td>27.5%</td>
<td>2.493</td>
</tr>
<tr>
<td>JIM</td>
<td>PG 58-22</td>
<td>No</td>
<td>5.0</td>
<td>27.5%</td>
<td>2.488</td>
</tr>
<tr>
<td>SSURF</td>
<td>PG 76-28</td>
<td>Yes</td>
<td>5.8</td>
<td>15%</td>
<td>2.583</td>
</tr>
</tbody>
</table>

3.3 Preparation of Specimens

As mentioned previously, there are two laboratory testing methods involved in this study, fatigue testing and DC(T) fracture testing. Two specimen geometries, a beam geometry for the fatigue test and a cylindrical geometry for the DC(T) test, were compacted. This particular study only needed to test the DC(T) fracture samples since all fatigue testing data was obtained from the RAP fatigue study. However, in order to illustrate how to prepare the beam fatigue specimen, this section will provide a brief procedural description of sample preparation.
**Flexural Beam Fatigue Preparation**

Laboratory testing of the fatigue study was performed on several asphalt mixtures at ATREL facility. The first step was to compact the asphalt concrete beams. Mixtures were heated to 135°C – 165°C depending on the types of asphalt concrete. Then the samples were compacted to get reach the desired level of air voids using the rolling wheel compactor (RWC).

The RWC is shown in Figure 3.1 Rolling Wheel Compactor (RWC) works by applying a vertical pressure with the assistance of a movable table. This movable table moves back and forth to obtain the desired air voids. The mold is fixed to the movable table in the RWC and the steel wheel starts to compact through the vertical load. A steel mold, with dimensions of 375mm x 125mm x 75mm, was used for the fatigue study. Both the asphalt mixture and the mold were heated to the compaction temperatures. After the oven aging period, the heated mixture was placed into the mold. The sample was rodded and the cover of the mold was placed at on top of it. While being compacted, the vertical load is increased gradually. During compaction, the cover moves down slowly, until the cover remains seated on top of the mold.

Voids in the specimen are controlled by the mixture weight inside the constant volume mold. Therefore, different weights were used for different mixtures to obtain specific levels of air voids. After compaction, volumetrics were checked for each compacted brick to check air voids. The asphalt concrete bricks were cut to obtain two beams from each brick using a diamond masonry saw. According to the SHRP standards the dimensions of the beam fatigue specimen are 380 ± 6 mm in length, 63 ± 6 mm in width, and 50 ± 6 mm in height. At least 6 mm were cut from both sides of the specimen to provide smooth outer surfaces.
Figure 3.1 Rolling Wheel Compactor (RWC)

DC(T) Specimen Preparation

Initially, the mixtures were heated to their compacting temperatures corresponding with their asphalt binder grade. Compaction of all mixtures was completed using an Industrial Process Controls (IPC) Servo Pac Superpave™ gyratory compactor at 30 RPM and a gyration angle of 1.25 degrees. After compaction, the gyratory specimens were cooled down to room temperature. In addition, the bulk specific gravity ($G_{mb}$) of the specimens was determined before the sawing process began.

According to the testing standards of the DC(T) test (ASTM 7313-07), samples require must conform with a specimen size of 50-mm thick by 150-mm diameter. The first step of specimen fabrication was to cut the ends of each gyratory sample to the desired thickness of 50 mm using a water-cooled masonry saw. Next, the location of the loading holes was made. The loading holes were fabricated with a water-cooled core drill using a core bit with a 25 mm outside diameter using the horizontally-mounted core drill shown in Figure 3.2 Core Drill Used for Fabricating the Loading Holes (The flat edge can be cut into the specimen using a water-cooled masonry saw.) Finally, the notch can be fabricated using the same masonry saw as show in Figure 3.3.
Figure 3.2 Core Drill Used for Fabricating the Loading Holes

Figure 3.3 Notching Made by Saw Cut
3.4 Testing Procedures

3.4.1 Fatigue Test

The 4-point beam bending test is the most common testing apparatus to measure fatigue characteristics of asphalt concretes. In order to be more explicit about how the fatigue testing was completed, this section will briefly discuss the fatigue testing equipment and testing conditions being used on the mixtures using in this study.

The pneumatic beam fatigue apparatus was utilized to test the asphalt concrete beams. The equipment consists of three main components: the testing frame, the environmental chamber, and the control data acquisition system (CDAS). Figure 3.4 shows a picture of the Fatigue Beam Apparatus.

![Figure 3.4 Fatigue Beam Apparatus](image)

The testing frame is a controlled third point loading frame that satisfies the AASHTO TP8-94 for sample positioning. A load cell is used to measure the force applied to the specimen. In addition, the maximum force the machine can apply is 5 KN. A 1 mm stroke LVDT is used to measure the deflection of the specimen. The LVDT measures the deflection at the center of the asphalt specimen. In term of loading cycles, the machine can run up to 100 million load cycles. For the environmental chamber, it contains the testing frame and specimens inside. The chamber can maintain temperatures between 2 °C and 60 °C. All tests were conducted as specified in SHRP standards at 20 ± 0.5 C (AASHTO TP8-94). Temperature transducers measure the temperature at both the skin and core of the specimen.
In terms of data acquisition, the CDAS automatically controls the operation of the beam cradle during the test. Also, it directly controls the valve to apply the requested loading rate. The control system automatically adjusts the output waveform to match the input waveform producing very precise control. The normalized input means that any transducer with +/- 10v output range can be plugged into any channel, which enhances the flexibility of the data acquisition module. The CDAS with the personal computer controls the load deformation during testing and collects the data at the same time.

Generally, the testing procedure of the fatigue beam test is in accordance with the ASTM 7460. Testing conditions used in this study followed the following steps. First, the asphalt concrete specimens were stored in the chamber for at least two hours to reach the required test temperature. Two modes of loading were used in the fatigue study of the RAP project. The following parameters were used in the fatigue study:

- Mode of loading: controlled-stress, and controlled-strain,
- Wave shape: haversine (in controlled-strain) and sine in controlled-stress testing),
- Load pulse width 100 ms (10 Hz), and
- Temperature 20 °C.

At least three specimens were tested to established a representative fatigue curve. Testing was conducted at varying strain/stress levels to generate a fatigue curve for the material.

**Parameters from testing results**

Seven test parameters which are automatically obtained from the data acquisition system are determined from the fatigue test: maximum tensile stress, maximum tensile strain, flexural stiffness, modulus of elasticity, phase angle, dissipated energy, and the cumulative dissipated energy. The following formulas are used to calculate the different test parameters during the test:

1. Maximum Tensile Stress (kPa):

   \[
   \sigma_t = \frac{3000aP}{wh^2} \tag{3.1}
   \]

   where  
   - \(a\) = distance between reaction and load clamps;
   - \(P\) = peak force (N);
   - \(W\) = beam width (mm); and
h = beam height (mm);

2. Maximum Tensile Strain (mm/mm):

\[ \varepsilon_t = \frac{128h}{23a^2} \]  

where \( \delta \) = peak deflection at center of beam (mm).

3. Flexural stiffness (MPa):

\[ S = \frac{\sigma_t}{1000\varepsilon_t} \]  

4. Modulus of Elasticity (MPa):

\[ E = \frac{Pa}{8Wh} \left[ \frac{23a^2}{4h^2} + k(1 + \nu) \right] \]  

where \( k \) = actual shear stress divided by average shear stress, and \( \nu \) = Poisson’s ratio.

5. Phase Angle (degree):

\[ \phi = 360 \frac{fs}{s} \]  

where \( s \) = time lag between P and \( \delta \), in seconds, and \( P \) = load frequency (Hz).

6. Dissipated Energy per cycle (kPa):

The dissipated energy is calculated by the area within the stress-strain hysteresis loop for each captured data pulse.

7. Cumulative Dissipated Energy (Mpa):

Cumulative dissipated energy is the summation of the dissipated energy per cycle.

3.4.2 DC(T) Fracture Test

As mentioned in Chapter 2, in order to make the DC(T) fracture test practical, the test geometry should be simple and capable of being fabricated from specimens during the mixture design process which is the most typical laboratory gyratory compaction. Since these criteria were met, the DC(T) fracture test is promising fracture test. In this study, the DC(T) test will be used to study the influence of temperature, loading rate, and specimen thickness on the fracture energy.
All DC(T) fracture test was conducted using an Instron 8500 machine. This apparatus consists of three main components: the loading frame, the environmental chamber, and the control data acquisition system (CDAS). Figure 3.5 shows a picture of testing sample in the Instron Log machine.

![Figure 3.5 DC(T) Test in Instron Machine](image)

A load frame was monitored with a 10 kN load cell which is used to measure the force applied to the specimen. The maximum force the machine can apply is 100 kN. The crack mouth opening displacement (CMOD) was monitored with an Epsilon Model clip-on gage containing a gage length of 5 mm and travel of 6.35 mm. Along with the load and CMOD, other quantities were measured using other various instruments. The crack initiation and propagation was monitored using the crack detection gages attached to the surface of the specimen. The environmental chamber contains the testing frame and specimens inside. The chamber has a capability of controlling the temperature between -30 and 30 C within ± 0.1 C.
The testing procedure of the DC(T) fracture test is in accordance with the ASTM 7313-07. Testing conditions used in this study followed the following steps. First, the asphalt concrete specimens were stored in the chamber for at least two hours to reach the required test temperature to ensure that the test results were not influenced by different temperature regimes within each specimen. Then, once the temperature conditioning was completed, the specimen was placed into the loading fixture and a seating load was applied (approximately 0.2 kN). The following test parameters were used in the DC(T) fracture test:

- The standard CMOD rate was 1 mm/min based upon the time to peak load,
- The test was continued until the load dropped below 0.1 kN,
- Wave shape: sine, and
- Temperature -12 °C.

Five replicating specimens were tested to established a representative load-CMOD curve for each mixture. Testing was conducted at the standard testing temperature of -12 °C.

**Testing result Parameters**

One of the fracture testing parameters, the fracture energy \( (G_f) \), can be extracted from the DC(T) fracture test results and was used in this study. By definition, the fracture energy can be determined from a calculation of the area under the load-displacement curve as described in Chapter 2.
Chapter 4 Analysis of Results

This chapter presents testing results of fatigue and fracture tests. Furthermore, correlations between testing parameters from these two tests were made. In order to provide a clear organization on the analysis of the results, this chapter was divided into three main sections. The first section describes the fatigue parameters were from the fatigue study as mentioned in the scope of this paper. The second section displays the fracture parameters derived from the DC(T) testing results. The last section examines possible correlations between the testing parameters in order to make a tentative relationship between the fatigue and fracture mechanisms.

4.1 Determination of Fatigue parameters

A four-point bending beam fatigue test was performed on the various mix designs which included a variety of asphalt binder types, asphalt binder contents, air voids contents, and different gradations. Beams (dimensions of 375mm x 125 mm x 75 mm) were tested at a testing temperature of 20 °C in accordance with AASHTO TP8-94. Five loose mixtures, collected from field samples during construction had a target air void content of approximately 7 %. The physical properties of these mixes were showed in Table 3.2 of the Chapter 3.

4.1.1 A Traditional Fatigue Life (Nf50)

The failure criterion in fatigue testing is defined based on the mode of loading. Two modes of loading were used in the fatigue testing and included: constant stress usually used for relatively thick pavements, and constant strain testing used for conventional flexible pavements. In this particular study, all mixtures were tested under controlled-strain mode conditions. The failure criterion based on control strain testing has been suggested as a 50% reduction in the initial stiffness or as a 50% reduction in the initial stress or initial force. According to Bazin (1967) after significantly many cycles, the stress will be very small and it will be very difficult to break the sample. Therefore, there is no clear fracture in the sample.

Traditional fatigue analysis is based on the initial value of the stress or the strain and the number of load cycles to failure. Failure was defined as the number of load cycles to 50% reduction in initial stiffness. Therefore, the traditional fatigue curve is obtained through plotting
the initial stress versus the corresponding number of load cycles to the 50% stiffness on a log-log scale. Based on the testing results, some of the fatigue parameters were determined in this section. The first parameter is a traditional fatigue life which is defined as a decrease in initial stiffness by 50 percent.

The following figures (Figure 4.1–4.3) show plots of load cycles to true failure \( (N_{tf}) \) versus stress at the bottom of the flexural beam (mm/mm) under the controlled-strain mode of loading. Each plot has separately shown three replicates of testing specimens using the same mix designs. However, each replicate has different levels of air voids due to compacting procedures. Therefore, traditional fatigue curves are displayed in a slightly different fashion as shown in the following figures.

As described in Chapter 2, the failure criterion in constant strain testing is widely defined as 50% reduction in the initial stiffness or 50% reduction in the initial stress (Bazin 1967). This definition represents a consistent measure life in the sample. Table 4.1 shows the failure criteria as the number of cycles to failure to 50% reduction in the initial stress for all mixtures used in the study.

<table>
<thead>
<tr>
<th>MIX_ID</th>
<th>Rep*</th>
<th>AV (%)</th>
<th>Strain Rate</th>
<th>Initial Stress (kPa)</th>
<th>( N_{f50} ) (cycle)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DAVE</td>
<td>1</td>
<td>7.94</td>
<td>300</td>
<td>1,277</td>
<td>120,930</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.29</td>
<td>700</td>
<td>2,560</td>
<td>14,400</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>8.55</td>
<td>1,000</td>
<td>2,821</td>
<td>1,540</td>
</tr>
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<td>STAN</td>
<td>1</td>
<td>6.58</td>
<td>300</td>
<td>1,110</td>
<td>155,400</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.37</td>
<td>700</td>
<td>2,410</td>
<td>4,810</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>7.15</td>
<td>1,000</td>
<td>2,203</td>
<td>660</td>
</tr>
<tr>
<td>BILL</td>
<td>1</td>
<td>7.30</td>
<td>300</td>
<td>819</td>
<td>380,900</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.15</td>
<td>700</td>
<td>1,805</td>
<td>20,590</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>6.85</td>
<td>1,000</td>
<td>1,675</td>
<td>4,060</td>
</tr>
<tr>
<td>JIM</td>
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<td>7.77</td>
<td>300</td>
<td>986</td>
<td>51,250</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.81</td>
<td>700</td>
<td>2,337</td>
<td>26,880</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>7.70</td>
<td>1,000</td>
<td>850</td>
<td>3,390</td>
</tr>
<tr>
<td>SSURF</td>
<td>1</td>
<td>7.60</td>
<td>300</td>
<td>1,040</td>
<td>46,880</td>
</tr>
<tr>
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<td>2</td>
<td>6.81</td>
<td>700</td>
<td>2,129</td>
<td>42,800</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>6.43</td>
<td>1,000</td>
<td>1,493</td>
<td>6,610</td>
</tr>
</tbody>
</table>
Figure 4.1 Traditional Fatigue Curve (at the Strain Rate of 300 Microstrain)
Figure 4.2 Traditional Fatigue Curve (at the Strain Rate of 700 Microstrain)
Figure 4.3 Traditional Fatigue Curve (at the Strain Rate of 1,000 Microstrain)
4.1.2 Ratio of Dissipated Energy Change (RDRC) Approach

The dissipated energy approach, which is reviewed in the literature section, is employed to evaluate the fatigue behavior in HMA material. Carpenter and Jansen (1997) suggested using this concept and introduced the change in dissipated energy to relate damage accumulation and fatigue life. Their work was expanded by Ghuzlan and Carpenter (2000) and correlated well with fatigue results. In this work, researchers use the ratio of dissipated energy change (RDEC) as an energy parameter to describe HMA fatigue damage.

The concept of RDEC states that the dissipated energy during a cyclic loading which can be expressed as the area under the stress-strain hysteresis loop describes the creation of damage in the material. Specifically, the relative change value of dissipated energy has a direct relation to damage accumulation. This approach provides a true indication of the damage being done to the mixture from one cycle to another by comparing the previous cycle’s energy level and determines how much of it caused damage (Ghuzlan 2001). The RDEC ratio can be represented as:

\[
\text{RDEC}_a = \frac{DE_a - DE_b}{DE_a \times (b-a)}
\]  

(4.1)

where \( \text{RDEC}_a \) = the average ratio of dissipated energy change at cycle a, compared to the next cycle b;

\( DE_a \) and \( DE_b \) = the dissipated energy produced in load cycle a and b (kPa); and

\( a \) and \( b \) = load cycle a and b, respectively (kPa).

The damage curve represented by RDEC vs. loading cycles can be distinctively divided into three stages as shown in Figure 4.4. Damage can be described by the development of a plateau after the initial stage (stage I). This plateau stage (stage II), which is a period where there is a constant percentage of energy turned into damage, will expand throughout the service life until reaching a dramatic increase in RDEC which a sign of initiation of the last stage (stage III).
In a stage II, the RDEC value is almost constant. Ghuzlan and Carpenter (2003) defined a plateau value (PV as the RDEC value corresponding to the 50% stiffness reduction load cycle \( N_{f50} \)). The PV value is important because it provides a unique relationship with fatigue life for different mixtures, loading modes and loading levels (Ghuzlan et al 2000 and 2001). Moreover, this procedure provides a consistent methodology to develop an energy-damage value (PV).

**PV-value Calculation**

In this RDEC approach, PV is defined as the RDEC value at the 50% initial stress reduction failure point \( N_{f50} \). The PV value can be calculated using Equation 4.3:

\[
PV = \frac{1 - \left( \frac{100}{N_{f50}} \right)^k}{100}
\]  

(4.2)

where \( N_{f50} \) = the initial stress or stiffness reduction load cycle \( N_{f50} \); and

\( k \) = the exponential slope of the power equation for the regressed DE-LC curve.

In order to obtain DE-LC (dissipated energy vs loading cycle) relationship, the curve was plotted from standard fatigue testing data. For example, a fitted curve for DE-LC relation was shown in Figure 4.5 DE-LC Plot with Fitted Curve (Shen, 2006).
Based upon the fatigue testing results of this study, each mix was tested at different strain levels of 300, 700, and 1,000 microstrain. Figure 4.6 to 4.11 display plots of the RDEC and DE-LC ($N_{tf}$) corresponding to each testing strain level. As described above, the $K$ parameter being used to calculate a plateau value can be obtained from the DE-LC ($N_{tf}$) plot.
Figure 4.6 The RDEC-Ntf Plot for Determination of PV Value
(at a Rate of 300 Microstrain)

Figure 4.7 The DE-Ntf Plot for Determination of K Parameter
(at a Rate of 300 Microstrain)
Figure 4.8 The RDEC-Ntf Plot for Determination of PV Value
(at a Rate of 700 Microstrain)

Figure 4.9 The DE-Ntf Plot for Determination of K Parameter
(at a Rate of 700 Microstrain)
Figure 4.10 The RDEC-Ntf Plot for Determination of PV Value
(at a Rate of 1,000 Microstrain)

Figure 4.11 The DE-Ntf Plot for Determination of K Parameter
(at a Rate of 1,000 Microstrain)
Table 4.2 shows the summary of fatigue parameters which were determined from the fatigue testing data and analysis. The number of cycles to 50% reduction of the stiffness (Nf50) and the K-value parameter were used to calculate plateau value (PV) using the Equation 4.2.

Table 4.2 Plateau Value (PV) for mixes in this study

<table>
<thead>
<tr>
<th>MIX ID</th>
<th>Rep*</th>
<th>AV (%)</th>
<th>Strain Rate</th>
<th>Nf50 (cycle)</th>
<th>K</th>
<th>Plateau Value (PV)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DAVE</td>
<td>1</td>
<td>7.94</td>
<td>300</td>
<td>120,930</td>
<td>-0.043</td>
<td>3.554E-07</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.29</td>
<td>700</td>
<td>14,400</td>
<td>-0.058</td>
<td>4.013E-06</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>8.55</td>
<td>1,000</td>
<td>1,540</td>
<td>-0.130</td>
<td>8.145E-05</td>
</tr>
<tr>
<td>STAN</td>
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<td>6.58</td>
<td>300</td>
<td>155,400</td>
<td>-0.045</td>
<td>2.895E-07</td>
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<td>2</td>
<td>7.37</td>
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<td>4,810</td>
<td>-0.072</td>
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<tr>
<td></td>
<td>3</td>
<td>7.15</td>
<td>1,000</td>
<td>660</td>
<td>-0.049</td>
<td>6.889E-05</td>
</tr>
<tr>
<td>BILL</td>
<td>1</td>
<td>7.30</td>
<td>300</td>
<td>380,900</td>
<td>-0.048</td>
<td>1.260E-07</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.15</td>
<td>700</td>
<td>20,590</td>
<td>-0.040</td>
<td>1.938E-06</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>6.85</td>
<td>1,000</td>
<td>4,060</td>
<td>-0.116</td>
<td>2.819E-05</td>
</tr>
<tr>
<td>JIM</td>
<td>1</td>
<td>7.77</td>
<td>300</td>
<td>51,250</td>
<td>-0.044</td>
<td>8.577E-07</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.81</td>
<td>700</td>
<td>26,880</td>
<td>-0.084</td>
<td>3.119E-06</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>7.70</td>
<td>1,000</td>
<td>3,390</td>
<td>-0.080</td>
<td>2.323E-05</td>
</tr>
<tr>
<td>SSURF</td>
<td>1</td>
<td>7.60</td>
<td>300</td>
<td>46,880</td>
<td>-0.042</td>
<td>8.949E-07</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>6.81</td>
<td>700</td>
<td>42,800</td>
<td>-0.024</td>
<td>5.601E-07</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>6.43</td>
<td>1,000</td>
<td>6,610</td>
<td>-0.034</td>
<td>5.104E-06</td>
</tr>
</tbody>
</table>

Rep* = Replication of Specimens

4.2 DC(T) Testing Results and Analysis

This section presents results from the DC(T) fracture tests. Similar mixtures used to determine the fatigue parameters were compacted using the gyratory compactor with dimensions of 6-inch in diameter by 2.5-inch in thickness. The testing temperature of -12 °C was selected as suggested according to ASTM D7313-07. Fracture parameters were obtained from DC(T) fracture testing results to be compared to the fatigue parameters. Four fracture parameters; total fracture energy, pre-peak load fracture energy, post-peak load fracture energy, and fracture strength, were used in this study.

- **Total Fracture Energy** - The total fracture energy is calculated by the area under a load-displacement curve divided by fracture-surface area as defined in Equation 4.4. It is generally expressed in units of Joules per square meter (J/m²).
\[ G_F = \frac{A_f}{BL} \]  \hspace{1cm} (4.3)

where \( G_F \) = Fracture energy (J/m\(^2\)),
\( A_f \) = Area under Load-CMOD curve (kN-mm),
\( B \) = Thickness of specimen (mm), and
\( L \) = Ligament length (mm).

- **Pre-Peak Fracture Energy** - A parameter similar to total fracture energy, but differs in the calculation of the area under a load-displacement curve. This parameter only integrates the area between the start of the recorded data to the peak load.

- **Post-Peak Fracture Energy** - An equation is the same as total fracture energy, but only difference is that the calculation of the area under a load-displacement is the integration of the area between the peak load and the end of the recorded data.

- **Fracture Strength** - It was calculated using the standard formula for computing the plane-strain fracture strength of metallic materials under the DC(T) test configuration in accordance with ASTM E399-90.

\[ S_f = \frac{2P(2W+a)}{B(W-a)^2} \]  \hspace{1cm} (4.4)

where \( S_f \) = Fracture strength (MPa),
\( P \) = Maximum load sustained by sample (N),
\( B \) = Thickness of specimen (mm), and
\( W \) and \( a \) = dimensions which are defined by ASTM E399-90.

Figure 4.27 to 4.31 show the load-CMOD (displacement) plots of each mix used in this study. Three replicates of each testing specimen were tested. Table 4.3 shows the four-fracture parameters discussed in the above content.
Figure 4.12 Fracture Energy of DAVE Mix-ID

Figure 4.13 Fracture Energy of STAN Mix-ID
Figure 4.14 Fracture Energy of BILL Mix-ID

Figure 4.15 Fracture Energy of Jim Mix-ID
Figure 4.16 Fracture Energy of SSURF Mix-ID
**Table 4.3 A Summary of Fracture Parameters from DC(T) Testing Result**

<table>
<thead>
<tr>
<th>Mix_ID</th>
<th>Binder</th>
<th>% AC</th>
<th>Rep*</th>
<th>Air Void (%)</th>
<th>Peak Load (kN)</th>
<th>Fracture Strength (MPa)</th>
<th>Fracture Energy (Gf) - J/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Pre- Peak CoV</td>
</tr>
<tr>
<td>DAVE</td>
<td>PG 58-22</td>
<td>4.5</td>
<td>1</td>
<td>7.44</td>
<td>2.632</td>
<td>3.828</td>
<td>6.92</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>7.61</td>
<td>2.994</td>
<td>4.355</td>
<td>47</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>8.10</td>
<td>2.702</td>
<td>3.930</td>
<td>40</td>
</tr>
<tr>
<td>STAN</td>
<td>PG 64-22</td>
<td>4.5</td>
<td>1</td>
<td>7.75</td>
<td>2.746</td>
<td>3.994</td>
<td>10.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>7.91</td>
<td>2.652</td>
<td>3.857</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>8.56</td>
<td>2.264</td>
<td>3.293</td>
<td>24</td>
</tr>
<tr>
<td>BILL</td>
<td>PG 58-28</td>
<td>4.7</td>
<td>1</td>
<td>7.97</td>
<td>2.626</td>
<td>3.819</td>
<td>3.08</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>7.83</td>
<td>2.727</td>
<td>3.966</td>
<td>44</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>8.11</td>
<td>2.566</td>
<td>3.732</td>
<td>31</td>
</tr>
<tr>
<td>JIM</td>
<td>PG 58-22</td>
<td>5.0</td>
<td>1</td>
<td>8.29</td>
<td>2.770</td>
<td>4.029</td>
<td>2.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>7.67</td>
<td>2.717</td>
<td>3.952</td>
<td>49</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>8.22</td>
<td>2.628</td>
<td>3.823</td>
<td>41</td>
</tr>
<tr>
<td>SSURF</td>
<td>PG 76-28</td>
<td>5.8</td>
<td>1</td>
<td>7.62</td>
<td>3.500</td>
<td>5.091</td>
<td>4.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>7.44</td>
<td>3.225</td>
<td>4.691</td>
<td>79</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>7.98</td>
<td>3.207</td>
<td>4.665</td>
<td>82</td>
</tr>
</tbody>
</table>

*Rep* = Replication
According to expected trends of both fatigue and fracture results, the ranking of resistant ability of the mixtures were made based upon trends reported by Epps 1969 as shown in Table 4.4.

**Table 4.4 Ranking of Mixtures**

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Binder</th>
<th>% AC</th>
<th>Expected Fatigue/ Fracture Raking * (in controlled strain)</th>
<th>Fracture Energy (Gf) Ranking</th>
<th>Fracture Strength (Sf) Ranking</th>
<th>Fatigue Ranking (Microstrain)</th>
</tr>
</thead>
<tbody>
<tr>
<td>STAN</td>
<td>PG 64-22</td>
<td>4.5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5, 5, 2</td>
</tr>
<tr>
<td>DAVE</td>
<td>PG 58-22</td>
<td>4.5</td>
<td>4</td>
<td>4</td>
<td>2</td>
<td>4, 4, 3</td>
</tr>
<tr>
<td>JIM</td>
<td>PG 58-22</td>
<td>5.0</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3, 2, 4</td>
</tr>
<tr>
<td>BILL</td>
<td>PG 58-28</td>
<td>4.7</td>
<td>2</td>
<td>2</td>
<td>4</td>
<td>2, 3, 1</td>
</tr>
<tr>
<td>SSURF</td>
<td>PG 76-28</td>
<td>5.8</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1, 1, 5</td>
</tr>
</tbody>
</table>

* Based upon trends reported by Epps 1969

**Note**: 1 is the best.

### 4.3 Correlation between Fatigue and Fracture Parameters

The analysis of the fatigue and fracture test results of the mixtures used in the study was carried out in sections 4.1 and 4.2, respectively. The fatigue parameters, which are comprised of the number of load to 50%-stiffness reduction \(N_{f50}\) and the plateau value (PV), were presented in Table 4.1. On the other hand, the fracture parameters are comprised of fracture strength \(S_f\), pre-peak fracture energy \(G_{f,pre}\), post-peak fracture energy \(G_{f,post}\), and total fracture energy \(G_f\) and are presented in Table 4.2. As stated in the objective of this study, the correlations between fatigue and fracture parameters were investigated in order to determine potential relationships between these two mechanisms. The followings are eight potential correlations of each strain level between fatigue and fracture parameters:
### A- For the strain level of 300 microstrain:

<table>
<thead>
<tr>
<th>Parameters (vs.)</th>
<th>Correlation</th>
<th>$R^2$</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{f50}$</td>
<td>$S_f$</td>
<td>$N_{f50} = 3E+24(S_f)^{5.4}$</td>
<td>0.408</td>
</tr>
<tr>
<td></td>
<td>$G_{f-Pre}$</td>
<td>$N_{f50} = 474,655e^{-0.032(G_f-pre)}$</td>
<td>0.404</td>
</tr>
<tr>
<td></td>
<td>$G_{f-Post}$</td>
<td>$N_{f50} = 2E+06e^{-0.007(G_f-post)}$</td>
<td>0.268</td>
</tr>
<tr>
<td></td>
<td>$G_f$</td>
<td>$N_{f50} = 1E+06e^{-0.006(G_f)}$</td>
<td>0.303</td>
</tr>
<tr>
<td>PV (vs.)</td>
<td>$S_f$</td>
<td>$PV = 5E-10(S_f) - 2E-06$</td>
<td>0.460</td>
</tr>
<tr>
<td></td>
<td>$G_{f-Pre}$</td>
<td>$PV = 2E-08(G_{f-pre}) - 2E-07$</td>
<td>0.539</td>
</tr>
<tr>
<td></td>
<td>$G_{f-Post}$</td>
<td>$PV = 4E-09(G_{f-post}) - 9E-07$</td>
<td>0.444</td>
</tr>
<tr>
<td></td>
<td>$G_f$</td>
<td>$PV= 3E-09(G_f) - 8E-07$</td>
<td>0.473</td>
</tr>
</tbody>
</table>

### B- For the strain level of 700 microstrain:

<table>
<thead>
<tr>
<th>Parameters (vs.)</th>
<th>Correlation</th>
<th>$R^2$</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{f50}$</td>
<td>$S_f$</td>
<td>$N_{f50} = 28.515(S_f) - 94,111$</td>
<td>0.756</td>
</tr>
<tr>
<td></td>
<td>$G_{f-Pre}$</td>
<td>$N_{f50} = 793.18(G_{f-pre}) - 13,850$</td>
<td>0.304</td>
</tr>
<tr>
<td></td>
<td>$G_{f-Post}$</td>
<td>$N_{f50} = 215.93(G_{f-post}) - 60,070$</td>
<td>0.894</td>
</tr>
<tr>
<td></td>
<td>$G_f$</td>
<td>$N_{f50} = 172.88(G_f) - 51,487$</td>
<td>0.905</td>
</tr>
<tr>
<td>PV (vs.)</td>
<td>$S_f$</td>
<td>$PV = 1E+30(S_f)^{-9.891}$</td>
<td>0.715</td>
</tr>
<tr>
<td></td>
<td>$G_{f-Pre}$</td>
<td>$PV = 0.5931(G_{f-pre})^{-3.255}$</td>
<td>0.880</td>
</tr>
<tr>
<td></td>
<td>$G_{f-Post}$</td>
<td>$PV= 3E+12(G_{f-post})^{-6.987}$</td>
<td>0.828</td>
</tr>
<tr>
<td></td>
<td>$G_f$</td>
<td>$PV = 1E+11(G_f)^{-6.338}$</td>
<td>0.843</td>
</tr>
</tbody>
</table>
C- For the strain level of 1,000 microstrain:

<table>
<thead>
<tr>
<th>Parameters Correlation</th>
<th>R-square</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>( S_f ) ( N_{f50} = 4.3378(S_f) - 14,396 )</td>
<td>0.658</td>
<td>(4.21)</td>
</tr>
<tr>
<td>( G_{f, Pre} ) ( N_{f50} = 123.12(G_{f, pre}) - 2,296.6 )</td>
<td>0.818</td>
<td>(4.22)</td>
</tr>
<tr>
<td>( G_{f, Post} ) ( N_{f50} = 35.68(G_{f, post}) - 10,292 )</td>
<td>0.918</td>
<td>(4.23)</td>
</tr>
<tr>
<td>( G_f ) ( N_{f50} = 28.162(G_f) - 8,702 )</td>
<td>0.903</td>
<td>(4.24)</td>
</tr>
<tr>
<td>( S_f ) ( PV = 0.158e^{0.002(S_f)} )</td>
<td>0.695</td>
<td>(4.25)</td>
</tr>
<tr>
<td>( G_{f, Pre} ) ( PV = 0.0004e^{0.06(G_{f, pre})} )</td>
<td>0.853</td>
<td>(4.26)</td>
</tr>
<tr>
<td>( G_{f, Post} ) ( PV = 5E+13(G_{f, post})^{-7.08} )</td>
<td>0.981</td>
<td>(4.27)</td>
</tr>
<tr>
<td>( G_f ) ( PV = 1E+12(G_f)^{-6.325} )</td>
<td>0.969</td>
<td>(4.28)</td>
</tr>
</tbody>
</table>

Appendix A displays the plots of the correlation among fatigue and fracture parameters which are expressed in the mathematic equations shown above.

**Table 4.5 The Correlation Matrix for Fatigue and Fracture Parameters**

<table>
<thead>
<tr>
<th>Fatigue parameter</th>
<th>( N_{f50} )</th>
<th>( PV )</th>
<th>( Average )</th>
<th>( CoV )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( S_f ) ( N_{f50} )</td>
<td>0.408</td>
<td>0.756</td>
<td>0.658</td>
<td>0.460</td>
</tr>
<tr>
<td>( G_{f, Pre} ) ( N_{f50} )</td>
<td>0.404</td>
<td>0.904</td>
<td>0.818</td>
<td>0.539</td>
</tr>
<tr>
<td>( G_{f, Post} ) ( N_{f50} )</td>
<td>0.268</td>
<td>0.894</td>
<td>0.918</td>
<td>0.444</td>
</tr>
<tr>
<td>( G_f ) ( N_{f50} )</td>
<td>0.303</td>
<td>0.905</td>
<td>0.903</td>
<td>0.473</td>
</tr>
<tr>
<td>( N_{f50} )</td>
<td>0.346</td>
<td>0.865</td>
<td>0.824</td>
<td>0.479</td>
</tr>
<tr>
<td>( PV )</td>
<td>0.346</td>
<td>0.865</td>
<td>0.824</td>
<td>0.479</td>
</tr>
<tr>
<td>( Average )</td>
<td>38.23</td>
<td>27.60</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 4.5 shows the correlation matrix between fatigue and fracture parameters in a term of statistical analysis of the R-square. Several points of discussions can be initiated from these results:

- By comparing fatigue parameters, the average R-square of the relationship between PV and other fracture parameters is higher than that of the Nf50 with the fracture parameters. In addition, the coefficient of variation of the PV values is less than that of the Nf50. Therefore, the plateau value (PV) should be considered a better fatigue parameter than the traditional fatigue criteria in order to make a correlation to fracture parameters.

- A comparison of the fracture parameters’ average R-square value was also made. It was shown that the relationship of the pre-peak fracture energy (Gf-pre) with fatigue parameters has the highest average R-square than the others. In addition, Gf-pre has an acceptable value of the coefficient of variation. Therefore, the pre-peak fracture energy would be a good representative of the fracture parameters to be used to predict fatigue behavior.

- By utilizing the plateau value (PV), the rate of loading can be altered and in turn affects the relationship between fatigue and fracture parameters. By increasing the strain level (to 1,000 microstain), it was shown that the correlation between the parameters is improved compared to those using the lower strain level according to the relatively high R-square value.
Chapter 5 Summary, Conclusions, and Recommendations

5.1 Summary

Fatigue behavior of asphalt concrete pavement has been studied by many researchers for several decades. Many experiments and models have been proposed to investigate its behavior in order to predict and to minimize damage to pavement by fatigue. Typically, fatigue cracking is thought to be an accumulation of damage under repeated load applications in an asphalt concrete pavement and often linked to tensile stress at the bottom of the HMA layer.

Prediction of fatigue performance is usually attempted by through laboratory experiments. As discussed in the literature review, pavement performance can be affected by various factors such as the type of asphalt binder, binder content, type of aggregate, aggregate structure, air void level, etc. Therefore, different types of mixtures comprised of various binder types and air void levels were used in this study. To determine relationships between fatigue and fracture, several fatigue and fracture parameters were obtained and correlated. The selected mixtures were fabricated and tested under the flexural-beam fatigue test and DC(T) fracture test.

The fatigue tests were conducted using a controlled-strain mode of loading. Three different strain rates were used included 300, 700, and 1,000 microstrains. A dissipated energy approach was used to obtain fatigue parameters, including: the number of cycle to 50 percent reduction in stiffness ($N_{f50}$) and plateau value (PV).

The same mixtures tested in the fatigue test were tested with the DC(T) fracture test at -10 \degree C. Fracture parameters such as fracture strength ($S_f$), pre-peak fracture energy ($G_{f-pre}$), post-peak fracture energy ($G_{f-post}$), and total fracture energy ($G_{f}$) were calculated using values extracting from load-CMOD plots.

Eight different correlations were made using a factorial combination of fatigue and fracture parameters.
5.2 Conclusions of the Study

Conclusions that can be drawn based on the results of this study include:

- Based on the fatigue result, both number of cycle to failure ($N_f$) and plateau value (PV) decreased with increasing strain rate. The change in dissipated energy versus number of cycles to failure have showed that the slope or K-parameter (which is used to determine the plateau value), was higher or steeper when the strain rate was increased. This agrees with the results of previous studies (Capenter, 2004).

- Based on the fracture results, it can be concluded fracture energy ($G_f$) increased when using a softer or more modified asphalt binder grade, or when asphalt content was increased. As expected, the fracture test was much less time consuming than the fatigue test.

- However, it was noticed that the loose mixes being used in this study were stored for more than 1 year. Therefore, the behavior of these mixtures might be influenced by the aging of the binder.

- The correlations between fatigue and fracture parameters indicated that the plateau value (PV) has a better relationship with fracture parameters than the traditional fatigue criteria ($N_{f50}$). Therefore, the plateau value (PV) should be selected to be a representative fatigue parameter to relate to the fracture test results. This is perhaps not surprising since the plateau value (PV) and fracture energy ($G_f$) parameters are both linked to energy consumption associated with cracking.

- The pre-peak fracture energy ($G_{f-pre}$) displayed the highest statistical value of correlation to the fatigue parameters, followed closely by total fracture energy. Therefore, the pre-peak fracture energy and total fracture energy appear to be the most promising parameters for the prediction of fatigue behavior.

- Furthermore, the correlations have shown that at a strain level of 300 microstrain, an inverse trend exists between the fatigue and fracture parameters investigated. Conversely, at the strain level of 700 and 1,000 microstrain, an increase of any fracture parameters results in an increase in fatigue parameters. The ranking of mixtures according to expected trends might suggest that the fracture energy values may be more trustworthy then the fatigue values.
5.3 Recommendations for Future Research

The correlation between fatigue and fracture parameters in this study provides a starting point for interconnecting these mechanisms. Further study would be recommended in the following areas:

- The effect of other test variables such as testing temperature, rate of loading, and specimen conditioning should be evaluated.
- The effect of other mix variables such as air void levels, aggregate structure, and other volumetric properties should be evaluated.
- The reason for the change in ranking of mixtures from the 300 microstrain level to the 700 and 1,000 microstrain levels is not clear. Further investigation is needed to determine if this is a true change in ranking of fatigue resistance, or if there are test artifacts present in tests conducted at lower strain levels, at least for the test results analyzed in this study.
REFERENCES


APPENDIX A

Figure A.1 The $N_{f50} - S_f$ Correlation for the mixes (At 300 Microstrain)

$y = 3E+24x^{-5.4}$
$R^2 = 0.4026$

Figure A.2 The $N_{f50} - G_{f, \text{pre}}$ Correlation for the mixes (At 300 Microstrain)

$y = 4.74655e^{-0.032x}$
$R^2 = 0.4044$
Figure A.3 The $N_{f50} - G_{f\text{-Post}}$ Correlation for the mixes (At 300 Microstrain)

$y = 2E+06e^{-0.007x}$  
$R^2 = 0.2682$

Figure A.4 The $N_{f50} - G_{f\text{-Total}}$ Correlation for the mixes (At 300 Microstrain)

$y = 1E+06e^{-0.006x}$  
$R^2 = 0.3033$
Figure A.5 The PV–$S_f$ Correlation for the mixes (At 300 Microstrain)

Figure A.6 The PV–$G_{f,Pre}$ Correlation for the mixes (At 300 Microstrain)
Figure A.7 The PV – $G_{f,\text{Post}}$ Correlation for the mixes (At 300 Microstrain)

Figure A.8 The PV – $G_{f,\text{Total}}$ Correlation for the mixes (At 300 Microstrain)
Figure A.9 The $N_{f50}$ – $S_f$ Correlation for the mixes (At 700 Microstrain)

\[ y = 28.515x - 94111 \]
\[ R^2 = 0.7563 \]

Figure A.10 The $N_{f50}$ – $G_{f-Pre}$ Correlation for the mixes (At 700 Microstrain)

\[ y = 793.18x - 13850 \]
\[ R^2 = 0.9039 \]
Figure A.11 The $N_{f50} - G_{f\text{-Post}}$ Correlation for the mixes (At 700 Microstrain)

\[ y = 215.93x - 60070 \]
\[ R^2 = 0.8941 \]

Figure A.12 The $N_{f50} - G_{f\text{-Total}}$ Correlation for the mixes (At 700 Microstrain)

\[ y = 172.88x - 51487 \]
\[ R^2 = 0.9051 \]
Figure A.13 The PV–$S_f$ Correlation for the mixes (At 700 Microstrain)

Figure A.14 The PV–$G_{f,pre}$ Correlation for the mixes (At 700 Microstrain)
Figure A.15 The PV – \( G_{f\text{-Post}} \) Correlation for the mixes (At 700 Microstrain)

\[ y = 3E+12x^{-6.987} \]
\[ R^2 = 0.8279 \]

Figure A.16 The PV – \( G_{f\text{-Total}} \) Correlation for the mixes (At 700 Microstrain)

\[ y = 1E+11x^{-6.338} \]
\[ R^2 = 0.8427 \]
Figure A.17 The $N_{f50} - S_f$ Correlation for the mixes (At 1,000 Microstrain)

\[ y = 4.3378x - 14396 \]
\[ R^2 = 0.6578 \]

Figure A.18 The $N_{f50} - G_{f-Pre}$ Correlation for the mixes (At 1,000 Microstrain)

\[ y = 123.12x - 2296.6 \]
\[ R^2 = 0.8184 \]
Figure A.19 The $N_{f50}$ – $G_f$-Post Correlation for the mixes (At 1,000 Microstrain)

$$y = 35.68x - 10292$$
$$R^2 = 0.9175$$

Figure A.20 The $N_{f50}$ – $G_f$-Total Correlation for the mixes (At 1,000 Microstrain)

$$y = 28.162x - 8702$$
$$R^2 = 0.9026$$
Figure A.21 The PV–$S_f$ Correlation for the mixes (At 1,000 Microstrain)

Figure A.22 The PV–$G_{f,\text{Pre}}$ Correlation for the mixes (At 1,000 Microstrain)
Figure A.23 The PV – $G_{f,\text{Post}}$ Correlation for the mixes (At 1,000 Microstrain)

Correlation of PV-$G_{f,\text{Post}}$ (@ 1,000 microstrain)

$y = 5E+13x^{-0.08}$

$R^2 = 0.9811$

Figure A.24 The PV – $G_{f,\text{Total}}$ Correlation for the mixes (At 1,000 Microstrain)

Correlation of PV-$G_{f}$ (@1,000 microstrain)

$y = 1E+12x^{-6.325}$

$R^2 = 0.9685$
VITA

Chaiwat Na chiangmai was born in Chiang Mai city, Thailand on December 22, 1981. He graduated from a high school of the Prince’s Royal College in March 2000 and entered Chiang Mai University in June of the same year, where he received his Bachelor degree in Civil Engineering in 2004. During the years of 2004 to 2008, he has been working for a Bureau of Highway Research and Development of the department of highway (DOH), Thailand. He began graduate school at the University of Illinois at Urbana-Champaign in August 2008 working toward a M.S. degree in Civil Engineering. After completion of his M.S. degree, Chaiwat will begin work at University of Illinois at Urbana-Champaign toward a Ph.D. in Civil Engineering.