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Durable Recycled Superpave Mixes in Kansas

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The use of economical and environment-friendly recycled asphalt materials has become increasingly popular for asphalt pavement construction. In general, reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS) are used in hot-mix asphalt (HMA). However, as higher amounts of RAP/RAS material are being promoted, the potential for premature pavement distresses, especially cracking, is increasing. In this research, four recycled Superpave mixtures (SR) obtained from Kansas Department of Transportation (KDOT) projects with varying RAP and RAS contents have been evaluated. Two of these mixtures contained 10% RAP and 5% RAS, while the other two contained 25% RAP but no RAS. Illinois semicircular bending (IL-SCB) and Florida indirect tensile strength (FL-IDT) tests were performed to assess mixture cracking and fracture properties. These test results showed that mixtures containing 10% RAP and 5% RAS have relatively low fracture energy (FE) and flexibility index (FI) but higher resilient modulus. However, creep compliance and energy ratio (ER) of these mixtures are lower. These results indicate that mixtures containing 10% RAP and 5% RAS are stiffer, more prone to cracking, and tend to absorb less fracture energy. Mixtures with 25% RAP and no RAS showed the opposite behavior.

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Final Report

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PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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Abstract

The use of economical and environment-friendly recycled asphalt materials has become increasingly popular for asphalt pavement construction. In general, reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS) are used in hot-mix asphalt (HMA). However, as higher amounts of RAP/RAS material are being promoted, the potential for premature pavement distresses, especially cracking, is increasing. In this research, four recycled Superpave mixtures (SR) obtained from Kansas Department of Transportation (KDOT) projects with varying RAP and RAS contents have been evaluated. Two of these mixtures contained 10% RAP and 5% RAS, while the other two contained 25% RAP but no RAS. Illinois semicircular bending (IL-SCB) and Florida indirect tensile strength (FL-IDT) tests were performed to assess mixture cracking and fracture properties. These test results showed that mixtures containing 10% RAP and 5% RAS have relatively low fracture energy (FE) and flexibility index (FI) but higher resilient modulus. However, creep compliance and energy ratio (ER) of these mixtures are lower. These results indicate that mixtures containing 10% RAP and 5% RAS are stiffer, more prone to cracking, and tend to absorb less fracture energy. Mixtures with 25% RAP and no RAS showed the opposite behavior.

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Chapter 1: Introduction

1.1 Background

The paving industry is continuously striving to increase the quality of asphalt products, which comprise approximately 94% of paved roads in the United States. Asphalt is the most widely used construction material in the world, and it is 100% recyclable. The use of recycled asphalt materials reduces material costs because virgin materials are replaced by asphalt and aggregates in recycled asphalt materials. In addition, use of recycled materials reduces the demand for non-renewable natural resources, such as virgin aggregate and asphalt binder, and eliminates the need for landfilling (West, 2015).

Reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS) are commonly used in asphalt pavements. RAP refers to asphalt material removed from pavement during resurfacing, rehabilitation, or reconstruction. RAS is the manufacturing waste and roofing tear-off from roof replacement. RAS typically contains higher asphalt binder contents. In the early 1990s, the Federal Highway Administration (FHWA) and the U.S. Environmental Protection Agency (EPA) estimated that more than 90 million tons of asphalt pavements were recycled annually (Copeland, 2011). Based on a report from the National Asphalt Pavement Association (NAPA), use of RAS significantly increased from 2009 to 2014, leading to approximately 1.9 million tons of recycled RAS in 2014 (Hansen & Copeland, 2015). Figure 1.1 shows the RAP and RAS reclaiming process.

Figure 1.1: RAP and RAS
Source: West (2015)

It is to be noted that the performance of asphalt pavements should not be compromised by the use of recycled materials. The asphalt industry aims to determine an optimum mixture in which recycled asphalt materials exhibit equal or improved performance compared to conventional mixtures.

1.2 Problem Statement

The Kansas Department of Transportation (KDOT) now allows Superpave mixtures with over 25% of RAP, as well as mixtures with 10% RAP and 5% RAS. Although these mixtures are accepted under current specifications, complications with mixture durability have arisen because aged binder from RAP and RAS is incorporated into the mixtures, thereby altering performance properties of the mixtures. Use of increased amounts of reclaimed materials raises the potential for premature pavement distresses, especially cracking. Therefore, performance of mixtures containing RAP or RAS must be investigated.

1.3 Objective

The primary objective of this study was to investigate cracking resistance of recycled Superpave mixtures with varying RAP and RAS contents. Four Superpave mixtures were selected for this study, and two test procedures, the Illinois semicircular bending test (IL-SCB) and the Florida indirect tensile strength test (FL-IDT), were used to assess Superpave mixture cracking properties.

1.4 Report Outline

This report is divided into five chapters. Chapter 1 states the background, problem statement, and objective of the research. Chapter 2 provides a summary of the literature review of recycled materials and a brief description of IL-SCB and FL-IDT tests and related research work. Chapter 3 describes the research methodology, materials used, and test procedure details. Chapter 4 presents the results obtained from the IL-SCB and FL-IDT tests and analysis of the results, and Chapter 5 summarizes conclusions and recommendations based on this study.

Chapter 2: Literature Review

2.1 Introduction

Use of recycled materials such as RAP and RAS in asphalt pavement construction provides economic and environmental benefits. The use of recycled asphalt materials reduces material costs and the demand for non-renewable natural resources such as aggregates. Although some Departments of Transportation (DOTs) allow increased amounts of recycled materials in hot-mix asphalt (HMA) mixtures, the effect of high percentages of recycled materials on long-term pavement performance is still a major concern. As a result, most DOTs in the United States use only 15–25% RAP, and RAS is usually limited to 5% (Tavakol, 2016). This chapter presents a comprehensive literature review of RAP and RAS use in HMA mixtures, cracking resistance of recycled Superpave mixtures, and descriptions of SCB and IDT cracking tests.

2.2 Reclaimed Asphalt Pavement (RAP)

The FHWA defines RAP as "removed or reprocessed pavement materials that contain asphalt binder and aggregates during resurfacing, rehabilitation, or reconstruction operations" (Copeland, 2011). Asphalt pavements were first recycled in 1915, but asphalt recycling did not become popular until the 1970s when the 1973 oil embargo caused skyrocketing crude oil prices (West, 2015).

Two major factors influence the use of RAP: economic savings and environmental benefits. Use of RAP reduces costs of materials, transportation, and disposal. RAP is a valuable alternative to virgin materials. Considering material and construction costs, use of 20–50% of RAP provides savings of 14–34% (Al-Qadi, Elseifi, & Carpenter, 2007). The process of recycling asphalt also provides an optimal cycle between natural resources and reclaimed materials (Copeland, 2011).

RAP is typically produced through pavement milling operations, and full-depth pavement demolition. Milling is a sub process of pavement rehabilitation in which distressed upper layers of pavement are removed to a given depth. Millings can be used directly in new asphalt mixtures, thereby reducing costs associated with further screening or crushing (Copeland, 2011). For full-depth demolition, heavy equipment breaks pavement into small pieces, but this method of pavement removal is slow and results in large chunks of pavement rubble that must be transported for crushing and screening to a manageable size for recycling (Copeland, 2011).

Despite many advantages associated with RAP use in mixtures, durability concerns prevent use of large percentages of RAP. Researchers have found that incorporation of RAP in HMA improves rutting performance but degrades fatigue and thermal performance (Al-Qadi et al., 2007). One primary disadvantage of asphalt mixtures with RAP or RAS is aged binders that are significantly stiffer than virgin binders. At small percentages (up to 20%), aged binders do not significantly affect asphalt mixture properties; however, when higher percentages of RAP are introduced into mixtures, increased binder stiffness can significantly influence binder performance and, consequently, mixture performance (Al-Qadi et al., 2007). Therefore, most state DOTs restrict the amount of RAP used in asphalt mixtures. Research conducted by FHWA, however, has shown that the performance and life span of pavements containing up to 30% RAP are identical to pavements without RAP (Copeland, 2011).

2.3 Recycled Asphalt Shingles (RAS)

RAS, which is processed from shingle manufacturers' waste and roof tear-offs, is most commonly used in pavement. The earliest usage of RAS began in the 1980s. Based on a survey by NAPA, RAS use significantly increased since 2009. In 2014, 19 state DOTs allowed RAS in asphalt mixes and about 2 million tons of RAS were recycled (Hansen & Copeland, 2015).

Asphalt shingles are composed of asphalt cement, fibers, fine aggregate, and mineral filler (Hansen, 2009). Approximately 1.2 million tons of manufacturer waste (MW) shingles and 12 million tons of post-consumer (PC, or roof tear-off) shingles are recycled each year (Hansen & Copeland, 2015). Asphalt stiffness, asphalt content, and potential for deleterious or hazardous materials are the primary distinguishing factors between MW and PC shingles. PC RAS typically has higher asphalt content than MW RAS, but PC RAS asphalt is much stiffer than the asphalt in MW RAS. PC RAS is also much more likely to contain deleterious materials. As a result, some states only allow MW RAS for road construction work (West, 2015).

AASHTO MP 23 (2015) is the current standard specification that covers use of RAS in asphalt mixtures. Most DOTs that permit use of RAS in asphalt mixtures currently limit RAS to

5% or less by mix weight because overall mixture stiffness decreases if the percentage of recycled shingles is higher than 5% (Hansen, 2009). Use of up to 5% RAS in HMA mixtures has demonstrated minimum impact on mixture performance. Individual state DOTs also have specific requirements for RAS use in combination with RAP.

2.4 Cracking Resistance Tests of Hot-Mix Asphalt

Cracking negatively affects the serviceability and quality of flexible pavement structures. Therefore, a simple, practical cracking resistance test that identifies asphalt mixture susceptibility to cracking is essential. Based on performance-based specifications, two approaches can be used to evaluate cracking resistance. The first approach estimates the number of loading cycles before cracks initiate at a certain temperature, and the other approach investigates the degree of damage on an undamaged sample with repetitive loading (Ahmed, 2015). Indirect tensile strength (IDT), direct tension (DT), semicircular bending (SCB), bending beam fatigue, and Texas overlay (OT) tests are frequently used to evaluate cracking resistance properties. The IL-SCB and FL-IDT tests were used in this study to investigate cracking resistance.

2.4.1 Semicircular Bending Test

Chong and Kuruppu (1984) initially proposed the SCB test method to study fracture properties of rock materials. Due to ease of sample preparation and test procedure, this test method has become a favored test for asphalt mixtures. The test, which evaluates fracture resistance parameters of an asphalt mixture, is basically a three-point bending test of a semicircular-shaped asphalt specimen with a notch at the bottom to ensure the crack propagates along the notch.

Li and Marasteanu (2010) evaluated low-temperature fracture resistance of asphalt mixtures with varying binder type, aggregate type, and air void content using the SCB test. Loading rate and notch length were also varied. The test was conducted at three temperatures: -6 °C, -18 °C, and -30 °C. Test results indicated that higher air voids resulted in lower fracture resistance. In addition, higher asphalt performance grade (PG) resulted in higher fracture energy (FE) at -30 °C (Li & Marasteanu, 2010; Elseifi, Mohammad, Ying, & Cooper, 2012; Nsengiyumva, 2015).

Al-Qadi et al. (2015) used SCB test geometry to investigate the cracking potential of asphalt mixtures with varying displacement rates and temperatures. Several plant-produced mixtures were tested to obtain FE value at temperatures from -30 °C to 30 °C with varying loading rates. Based on the results, FE was found to be stable at a high displacement rate and reached peak value with a loading rate of 5–100 mm/min. Also, peak fracture values were always obtained around 25 °C. A new IL-SCB test method was explored by Al-Qadi et al. (2015) at a temperature of 25°C and loading displacement rate of 50 mm/min. According to the IL-SCB test method (AASHTO TP 124, 2016), half disc-shaped specimens with thicknesses of 50±1 mm and diameters of 150±1 mm were used, and a notch was cut along the specimen axis in the middle to a depth of 15±1 mm with a 1.5±0.1 mm width. The air void content of test specimens was 7.0±0.5%. The configuration of the SCB test specimen is shown in Figure 2.1. Figure 2.2 shows a typical output of the SCB test or a load versus load-line displacement curve.

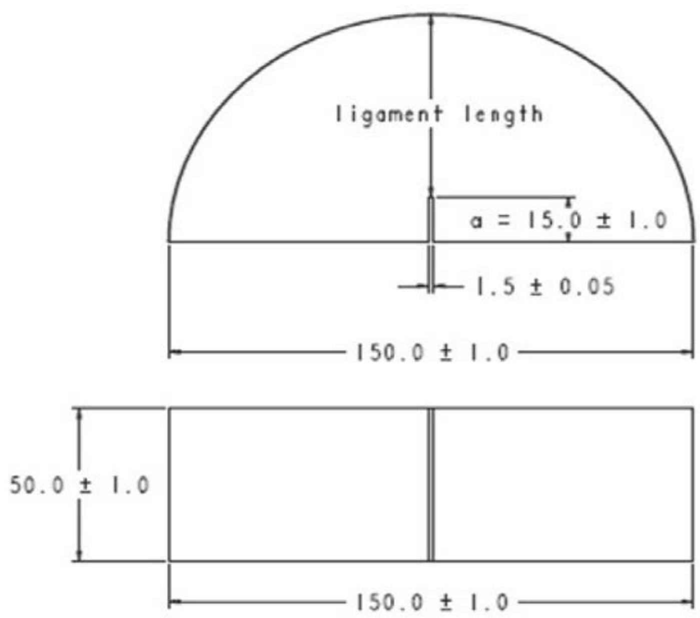


Figure 2.1: SCB Test Specimen Configuration
Source: AASHTO TP 124 (2016)

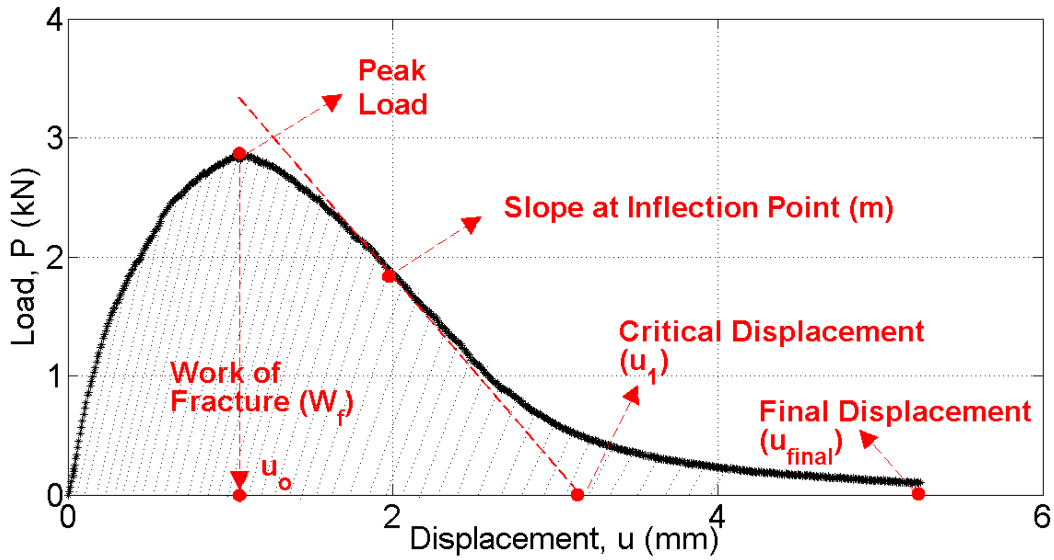


Figure 2.2: Recorded Load Versus Load-Line Displacement Curve
Source: AASHTO TP 124 (2016)

Fracture energy (FE) G_f was calculated by dividing the work of fracture (the area under the load-displacement curve in Figure 2.2) by the ligament area (the product of the ligament length and the specimen thickness). The FE equation is as follows (AASHTO TP 124, 2016):

$$G_f = \frac{W_f}{Area_{lig}} \times 10^6 \qquad \textbf{Equation 2.1}$$

Where:

G_f = fracture energy (Joules/m^2);

W_f = work of fracture (Joules); and

$Area_{lig}$ = ligament area (mm^2).

The calculated FE describes the overall capacity of an asphalt mixture to resist cracking damage. A mixture with high FE generally has high damage resistance (AASHTO TP 124, 2016).

In some cases, FE is not sufficient enough to evaluate cracking resistance of asphalt mixtures. For example, two asphalt mixtures may have identical FE values, but they have different load-displacement curves or different cracking responses, requiring a parameter that can describe fracture processes and overall pattern of the load-displacement curves (Al-Qadi et al., 2015). Thus, a parameter based on the load-displacement curve of the SCB test, flexibility index (FI) was introduced to assess cracking resistance of asphalt mixtures. FI is calculated by dividing FE by the slope of the load-displacement curve after the post-peak load, as shown below (AASHTO TP 124, 2016):

$$FI = \frac{G_f}{|m|} \times A$$ **Equation 2.2**

Where:

$|m|$ = absolute value of post-peak load slope m (kN/mm); and

A = unit conversion and scaling, A is equal to 0.01.

FI values are used to rank cracking resistance of asphalt mixtures in order to identify brittle mixes prone to premature cracking. Based on fatigue cracking measurements and structural analysis, Al-Qadi et al. (2015) determined a mixture is crack resistant with FI values between 2.0 and 6.0.

2.4.2 Florida Indirect Tensile Strength Test

The FL-IDT test, also known as the Superpave IDT test, was developed at the University of Florida in 1992 (Shu, Huang, & Vukosavljevic, 2008; Huang, Shu, & Vukosavljevic, 2011). This test can be used to obtain fatigue and fracture properties of asphalt mixtures. The FL-IDT test includes the resilient modulus test, the creep compliance test, and the IDT strength test. Based on these tests, a viscoelastic, fracture mechanics-based crack growth model for asphalt mixtures was developed. Two parameters, dissipated creep strain energy (DCSE) and energy ratio (ER), were also introduced to explain crack development and propagation in asphalt mixtures (Shu et al., 2008).

2.4.2.1 Resilient Modulus Test

Resilient modulus, which can be used to evaluate material quality, can also be an input for pavement design, evaluation, and analysis. The resilient modulus test requires repetitive application of a haversine waveform load on the cylindrical samples. The loads and resulting horizontal deformations are continuously recorded, but only the last five loading cycles of the total applied load pulses are selected to calculate the resilient modulus (ASTM D7369, 2011):

$$M_R = \frac{P_{cyclic}}{\Delta H \times t} \times (0.27 + \nu) \qquad \textbf{Equation 2.3}$$

Where:

M_R = instantaneous or total resilient modulus of elasticity, *MPa* (psi);

ΔH = recoverable horizontal deformation, mm (in.);

t = specimen thickness, mm (in.);

v = instantaneous or total Poisson's ratio;

P_{cyclic} = $Pmax$ - $Pcontact$ = applied cyclic load to specimen, *N* (lb);

$P_{contact}$ = contact load, *N* (lb); and

P_{max} = maximum applied load, *N* (lb), P_{max} is suggested to be 15% of peak load obtained from the ITS test.

Typical output of the resilient modulus test is shown in Figure 2.3.

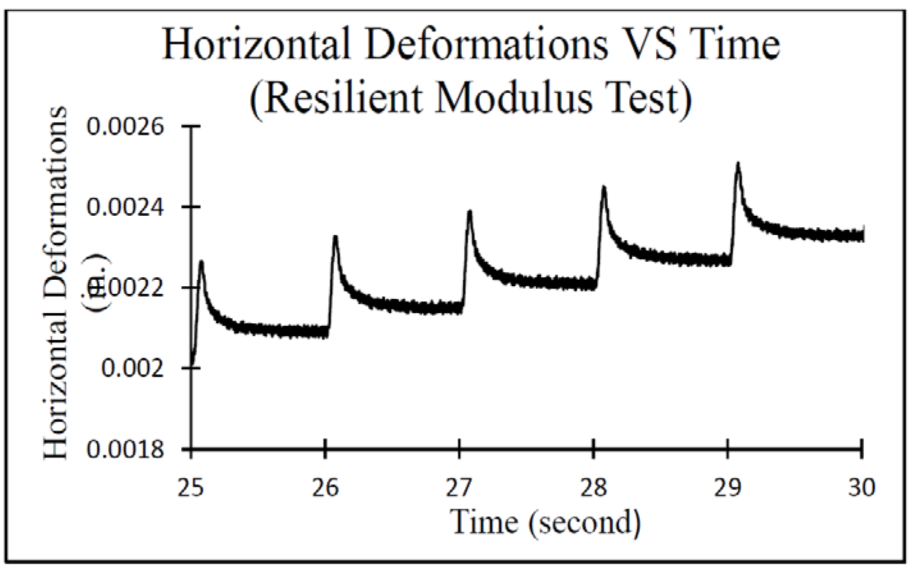


Figure 2.3: Typical Resilient Modulus Test Output
Source: Gong (2011)

2.4.2.2 Creep Compliance Test

Creep describes the relationship between the time-dependent strain and applied stress for viscoelastic materials (Gong, 2011). Creep test results are used to determine the master relaxation modulus curve and fracture parameters, which control thermal crack development and define asphalt mixture fracture resistance. In the creep test, a static load is applied along the axis of the test specimen and held constant. Horizontal and vertical deformations near the center of the specimen are measured and used to calculate creep compliance as a function of time. Creep compliance is calculated as follows (AASHTO T 322, 2011):

$$D(t) = \frac{\Delta X_{tm,t} \times D_{avg} \times b_{avg}}{P_{avg} \times GL} \times C_{cmpl} \qquad \textbf{Equation 2.4}$$

Where:

$D(t)$ = creep compliance at time t (kPa^{-1});

$\Delta_{tm,t}$ = trimmed value of horizontal deformation (mm);

D_{avg} = average diameter of the three replicates (mm);

b_{avg} = average thickness of the three replicates (mm);

P_{avg} = average creep load of the three replicates (kN);

GL = gauge length (mm);

C_{cmpl} = 0.6354 × $(\frac{X}{Y})^{-1}$ – 0.332 is the creep compliance coefficient; and

X/Y is the ratio of horizontal deformations to vertical deformations.

Figure 2.4 shows typical results of the creep compliance test.

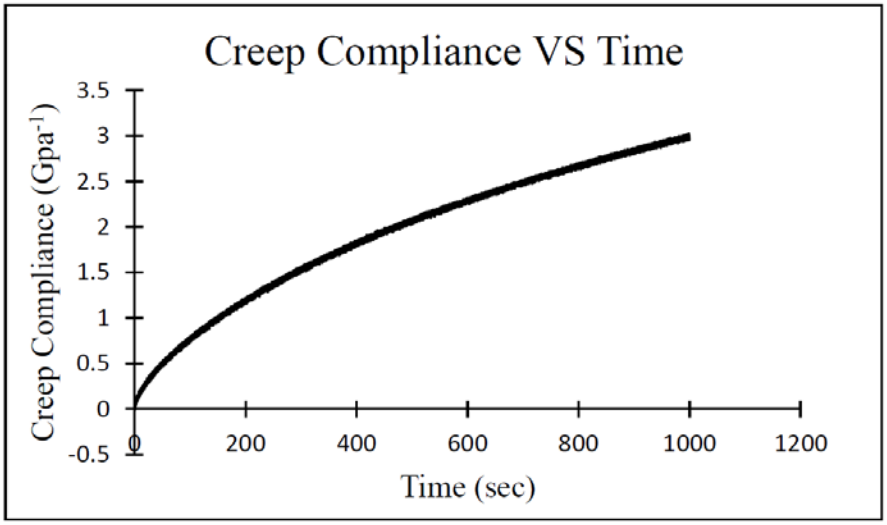


Figure 2.4: Typical Creep Compliance Test Output
Source: Gong (2011)

2.4.2.3 Indirect Tensile Strength Test

The IDT strength test, which determines tensile strength and strain of asphalt mixture specimens, has applications in cracking performance tests such as thermal cracking, fatigue cracking, and moisture-induced cracking. In the IDT test, a load is applied at a constant rate along the vertical axis of test specimens until the specimen fails; horizontal and vertical deformations and loads are measured. Because the IDT strength test is destructive, the test should be conducted after performing resilient modulus and creep tests. Based on AASHTO T 322 (2011), ITS can be calculated as follows:

$$S(t) = \frac{2 \times P}{\pi \times t \times D}$$ **Equation 2.5**

Where:
S(t) = indirect tensile strength, (psi);
P = maximum load, (lb);
t = specimen thickness, (in); and
D = specimen diameter, (in).

2.4.2.4 Energy Ratio

Researchers at the University of Florida have studied asphalt mixture cracking performance and derived ER from a fracture mechanics model (Roque & Lopp, 2008). Energy ratio (ER), calculated from mixture parameters obtained in the Superpave IDT test, is defined as dissipated creep strain energy at the failure of the mixture ($DCSE_f$) divided by the minimum dissipated creep strain energy required to resist damage ($DCSE_{min}$), as shown in the following equation:

$$ER = \frac{DCSE_f}{DCSE_{min}}$$ **Equation 2.6**

Each mixture has a damage threshold, and non-healable macro-damage appears when that threshold is exceeded. Therefore, the higher the DCSE, the longer the fatigue life of asphalt mixtures. Consequently, the ER must be greater than 1.0 in order for the mixture to be acceptable. However, the ER cannot completely define mixture cracking performance because if a mix has a $DCSE_f$ lower than 0.75, the mixture could fail, and if the ER is lower than 1 but the $DCSE_f$ is higher than 2.5, the mixture should perform well (Roque, Birgisson, Drakos, & Dietrich, 2004).

The $DCSE_f$ is obtained from the resilient modulus and IDT strength tests by analyzing the stress-strain (σ−ε) curve shown in Figure 2.5. $DCSE_f$ refers to the energy dissipated during one loading cycle, while a part of the fracture energy which was recovered is referred to as elastic energy (EE). The following equation explains the relationship between DCSE, FE, and EE:

$$DCSE_f = FE - EE$$ **Equation 2.7**

FE, which is the total energy applied to the specimen during the cracking process, can be obtained by calculating the hatched area under the stress-strain curve to the failure strain (ε_f), as shown in Figure 2.5.

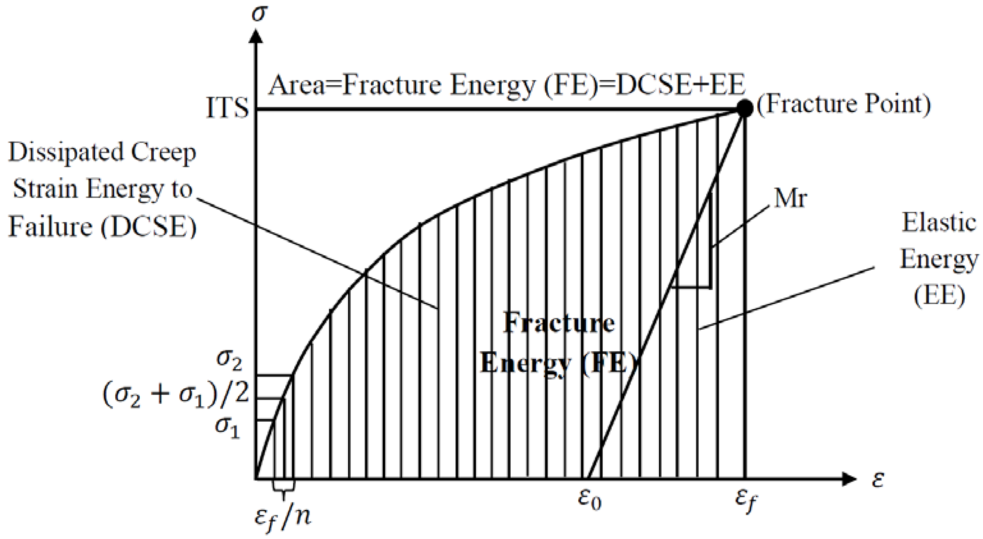


Figure 2.5: Relationship Between FE, EE, and DCSE$_f$
Source: Gong (2011)

Herein,

$$FE = \int_0^{\varepsilon_f} S(\varepsilon)\ \cdot d\varepsilon \qquad \textbf{Equation 2.8}$$

Where ε_f is the failure strain.

EE is the energy regained when external forces are removed. It is the area under the line beginning at the fracture point and ending at ε_0 in a slope of M_R, which was obtained from the resilient modulus test. The equation for EE is as follows (Shu et al., 2008):

$$EE = \frac{1}{2} S_t(\varepsilon_f - \varepsilon_0) \qquad \textbf{Equation 2.9}$$

DCSE$_{min}$, a function of creep compliance, is the minimum DCSE to resist cracking. Creep compliance *D(t)* can be represented using the following power function (Shu et al., 2008):

$$D(t) = D_0 + D_1 t^m$$ **Equation 2.10**

Where: D_0, D_1, and m are fracture mechanics parameters obtained from the creep compliance curve in logical coordinates shown in Figure 2.6. D_1 describes the initial part of the creep compliance curve, while m-value expresses the long-term creep strain rate of the same curve. An asphalt mixture with a low m-value indicates minimal damage accumulation (Gong, 2011).

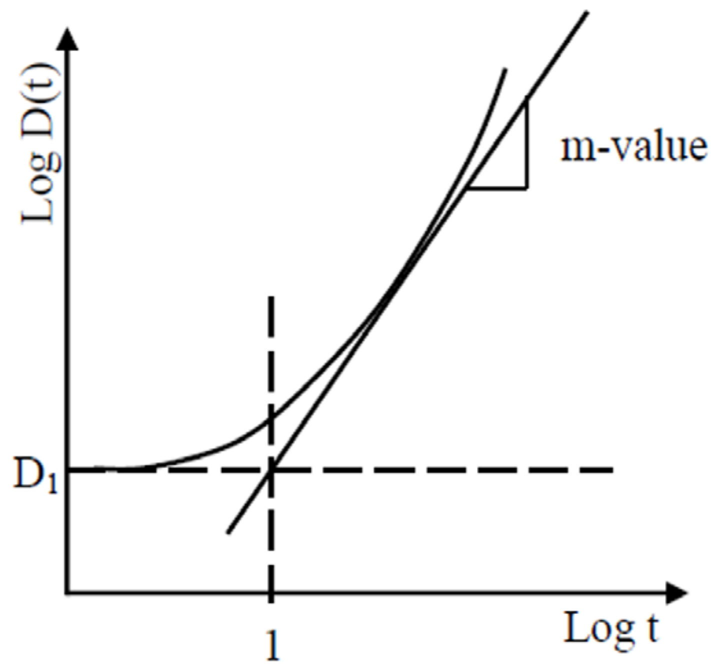


Figure 2.6: Creep Compliance Curve in Logical Coordinates

Thus, $DCSE_{min}$ can be calculated by the following equation:

$$DCSE_{min} = \frac{m^{2.98} \times D_1}{A}$$ **Equation 2.11**

Where: *A* is a coefficient factor that shows $DCSE_{min}$ to be dependent on both pavement structure characteristics and asphalt mixture tensile strength. *A* can be determined as a function of tensile strength (S_t) and tensile stress (σ) in asphalt pavement (Roque et al., 2004):

$$A = 0.0299 \times \sigma^{-3.1} \times (6.36 - S_t) + 2.46 \times 10^{-8} \qquad \textbf{Equation 2.12}$$

A default value 1,035 kPa (150 psi) is typically used for tensile stress unless a different value is given (Du, 2010; Kim, Sholar, & Byron, 2009).

2.5 Summary

RAP and RAS are commonly used in asphalt pavement mixtures, but cracking performance of these mixtures with RAP or RAS changes due to incorporation of aged asphalt binder. Researchers have studied various tests to characterize cracking resistance of HMA mixtures, with investigative emphasis on SCB and IDT tests. These tests not only obtain fatigue and fracture properties of asphalt mixtures to evaluate cracking potential, but they also correlate well with field performance.

Chapter 3: Laboratory Experiments

3.1 Material Sources

This study utilized four Superpave mixtures with varying RAP contents from projects in Kansas. Two mixtures, D1 and D4 from projects 70-89 KA-4136-01 and 400-11 KA-0740-01, contained 10% RAP and 5% RAS. The other two mixtures, D3 and D6 from projects 183-26 KA-3671-01 and 50-38 KA-3680-01, both contained 25% RAP but no RAS. All four mixtures had 12.5-mm nominal maximum aggregate size (NMAS), known as SR-12.5A. The binder grade of three mixes (D1, D3, and D4) was PG 64-28. Mix D6 had a higher asphalt binder grade, PG 70-28. Table 3.1 details the Superpave materials information.

Table 3.1: Project Material Information

Mix ID	Project Number	Mix Designation	Mix Source	Binder Grade	Recycled Material Content	Asphalt Content (%)	Virgin Asphalt Content (%)
D1	70-89 KA-4136-01	SR-12.5A	District 1	PG 64-28	10% RAP + 5% RAS	6.67	75
D3	183-26 KA-3671-01	SR-12.5A	District 3	PG 64-28	25% RAP	5.60	74
D4	400-11 KA-0740-01	SR-12.5A	District 4	PG 64-28	10% RAP + 5% RAS	6.30	67
D6	50-38 KA-3680-01	SR-12.5A	District 6	PG 70-28	25% RAP	5.20	72

3.2 Sample Preparation

Preparation of SCB and IDT tests specimens requires sample compaction, determination of specimen air voids, sample trimming, and preconditioning.

3.2.1 Sample Compaction

In this study, samples were compacted using a Superpave gyratory compactor (SGC). Before compaction, the molds and mixtures were preheated to the compaction temperature. The approximate weight of each compacted specimen was estimated to meet air void requirements. After ensuring the compaction temperature had been met, the pre-weighed loose mixture was charged into the mold using a chute. The mold was transferred into the SGC and the compactor was set to the "height" mode of 130 mm. Both SCB and IDT test methods require this sample height. Compaction stopped automatically when the SCG reached the specified height. The compacted sample was then removed from the mold and cooled for a few minutes. A total of 40 specimens, 10 for each mixture, were compacted.

3.2.2 Air Void Content Determination

Theoretical maximum specific gravity (G_{mm}) of loose mixture was determined according to Kansas test method KT-39 (2015). Bulk specific gravity (G_{mb}) of the compacted specimens was tested according to KT-15 (2015) test procedure.

3.2.2.1 G_{mm} Test Procedure

The sample size for the G_{mm} test depends on the NMAS. Since all mixtures used in this study had a 12.5-mm NMAS, a minimum of 1,500 gm loose samples were weighed and put into a calibrated flask. Then water at 25±1 °C (77±2 °F) was added to cover the sample in the flask. The flask was placed on an agitator, and a vacuum was applied for 14 minutes to remove trapped air bubbles. The flask was then immersed in a water bath at 25±1 °C (77±2 °F) for 10±1 minutes, and the weight was taken. G_{mm} was calculated using Equation 3.1.

$$G_{mm} = \frac{\text{Dry sample weight}}{\text{Dry sample weight} - \text{Weight of water displaced by sample}} \qquad \textbf{Equation 3.1}$$

3.2.2.2 G_{mb} Test Procedure

The KT-15 method was followed to determine G_{mb} for the compacted specimens. Dry weights were taken after the compacted samples cooled to room temperature, and then the compacted samples were immersed in the water bath at 25±1 °C (77±2 °F) for 4±1 minutes; submerged masses were taken. Then the samples were removed from the water and rolled on a damp towel to remove excess water from the samples' surface. The saturated surface dry (SSD) mass was then taken. G_{mb} was calculated using Equation 3.2:

$$G_{mb} = \frac{\text{Dry sample weight}}{\text{SSD weight} - \text{Submerged weight}}$$ **Equation 3.2**

Finally, the air voids were calculated using Equation 3.3:

$$V_a = 100 \times (1 - \frac{G_{mb}}{G_{mm}})$$ **Equation 3.3**

After determining the air voids of all 40 specimens, any specimen that did not meet the air void requirements was discarded. For SCB and IDT tests, the target air void was 7 ± 0.5%. Table 3.2 lists all specimens for the SCB and IDT tests.

Table 3.2: Air Voids of Specimens for SCB and IDT Tests

Mix ID	Specimen No.	Air Void (%)	Applicable Test
D1	#3	6.90	SCB Test
	#4	7.38	IDT Test
	#6	7.39	
D3	#6	6.65	SCB Test
	#4	6.76	IDT Test
	#5	6.86	
D4	#6	6.94	SCB Test
	#4	7.19	IDT Test
	#5	7.21	
D6	#1	6.93	SCB Test
	#2	7.35	IDT Test
	#4	7.41	

3.2.3 Sample Trimming

For the SCB test, an SCG-compacted specimen was cut into two specimens with thicknesses of 50±1 mm. Then these two round specimens were cut in the middle, resulting in four semicircular specimens with diameters of 150±2 mm and thicknesses of 50±1 mm. A notch with a depth of 15±1 mm and width of 1.5±0.1 mm was then cut in the bottom of the specimen to ensure the crack would propagate along the notch. Three semicircular samples were prepared for each mixture.

For the IDT test, a compacted specimen was sawn into two cylindrical specimens with thicknesses of 50 ± 1 mm. Four specimens were prepared for each mixture; one specimen was used to determine the applied load of the resilient modulus test by conducting indirect tensile strength test. An alignment device was used on both sides of the specimen to make the diametral axis horizontal and vertical, with parallel axes on two faces. The location of the diametral axis should avoid the abnormally large aggregate particles. Then four gauge points were mounted on each face of the specimen along the vertical and horizontal diametral axes with gauge lengths of 38 mm. The gluing jig was removed after the gauge points were properly set and glued.

3.2.4 Preconditioning

For preconditioning, the specimens had to be maintained within 0.5 °C of the desired test temperature, 25 °C, during the test period. Therefore, test specimens were preconditioned in the UTM-25 machine at 25 °C, for 2±0.5 hours before conducting cracking tests.

3.3 Illinois Semicircular Bending Test

The IL-SCB test was conducted according to AASHTO TP 124 (2016) standard test method. In this study, the IL-SCB test was conducted on an AMPT machine, as shown in Figure 3.1. UTS-034 software was used to evaluate the FE of the asphalt mixture.

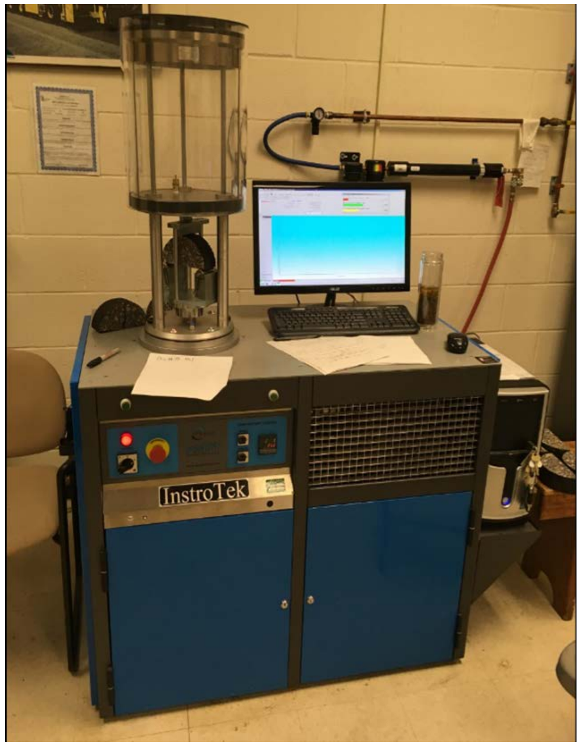


Figure 3.1: AMPT Machine for the IL-SCB Test

The prepared specimen, which rested on a three-point bending support frame, was placed in the loading chamber of the AMPT machine. Specimen alignment was adjusted so that the notch was directly beneath the loading head, as illustrated in Figure 3.2. Following test specimen setup adjustments, the loading chamber was closed to achieve a test temperature of 25 °C. The SCB test began once the test temperature was reached. A ramp load was applied to the specimen at a rate of 50 mm/min until the specimen failed. Once the peak load was reached, the test stopped automatically. Displacements and loads dependent on time were recorded.

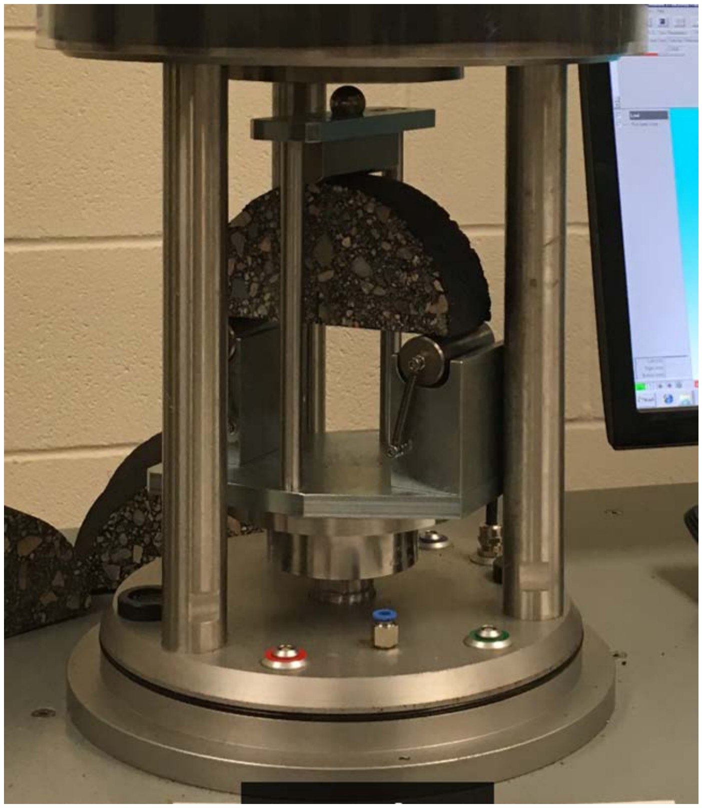


Figure 3.2: SCB Test Setup

3.4 Florida Indirect Tensile Strength Test

FL-IDT tests included the resilient modulus, creep, and IDT tests. Two horizontal and two vertical linear variable differential transformers (LVDTs) were attached to the gauge points to measure deformations on two faces of the specimen before placing the test specimen into the loading device. IPC Asphalt Tester software was used to conduct the FL-IDT test.

3.4.1 Resilient Modulus Test

An IDT test must be run to determine the load level for the resilient modulus test. In this study, following ASTM D7369 (2011) test standard, the target peak load was taken about 15% of the tensile strength measured in the IDT test. The specimen was mounted onto the loading frame, and the specimen position was adjusted to ensure that the vertical LVDT and loading strips were in the same line. Figure 3.3 illustrates the IDT test setup. The LVDTs were zeroed prior to testing, and testing began when the loading chamber temperature stabilized to the test temperature of 25 °C. A repeated haversine load was applied to the specimen for 0.1 second, and a rest period of 0.9

seconds was imposed for 100 cycles. Load and deformation for the last five loading cycles were recorded once the resilient deformation stabilized. After the specimen had been tested along the first diametral axis, the specimen was rotated 90° and the test was repeated.

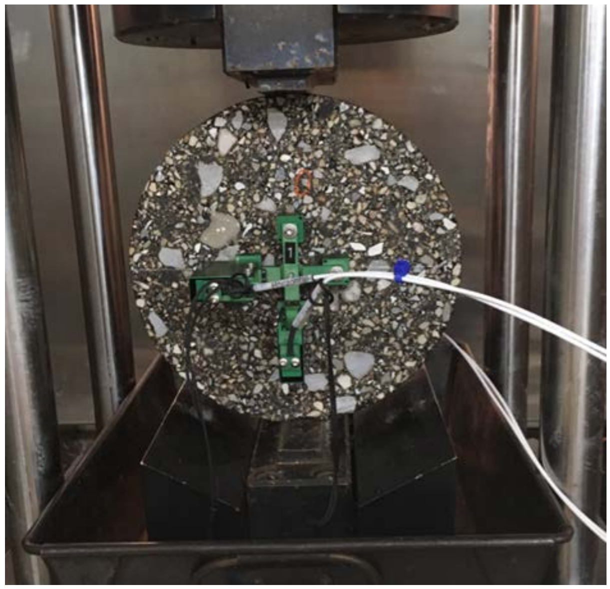


Figure 3.3: IDT Test Setup

3.4.2 Creep Compliance Test

After completion of the resilient modulus test, the creep compliance test was performed on the same specimen according to the AASHTO T 322 (2011) test method. Once the temperature and deformation were stabilized for 5 to 10 minutes, a static load that produced horizontal deformation within the range of 0.00125 to 0.0190 mm for specimens with diameters of 150 mm was applied to the specimen for 1,000 seconds. If either limit was violated, the test was stopped and restarted with an adjusted load after a 5-minute recovery period. The data acquisition frequency was 10 Hz for the first 10 seconds, 1 Hz for the next 90 seconds, and 0.1 Hz for the remaining 900 seconds. All horizontal and vertical deformations on both sides were recorded for further analysis.

3.4.3 Indirect Tensile Strength Test

The IDT test was conducted upon completion of the creep compliance test. The specimen was loaded to failure along the vertical diametral axis with a constant rate of 50 mm/min. The deformation and load dependent on time and the peak load were recorded.

Chapter 4: Results and Discussion

4.1 Illinois Semicircular Bending Test Results

As mentioned, the IL-SCB test was performed according to the AASHTO TP 124 (2016) test method in order to investigate cracking resistance of Superpave mixtures with varying RAP and/or RAS content. Three replicate specimens for each mixture were tested, using software UTS-034, on an AMPT machine. A typical output of the test is shown in Figure 4.1.

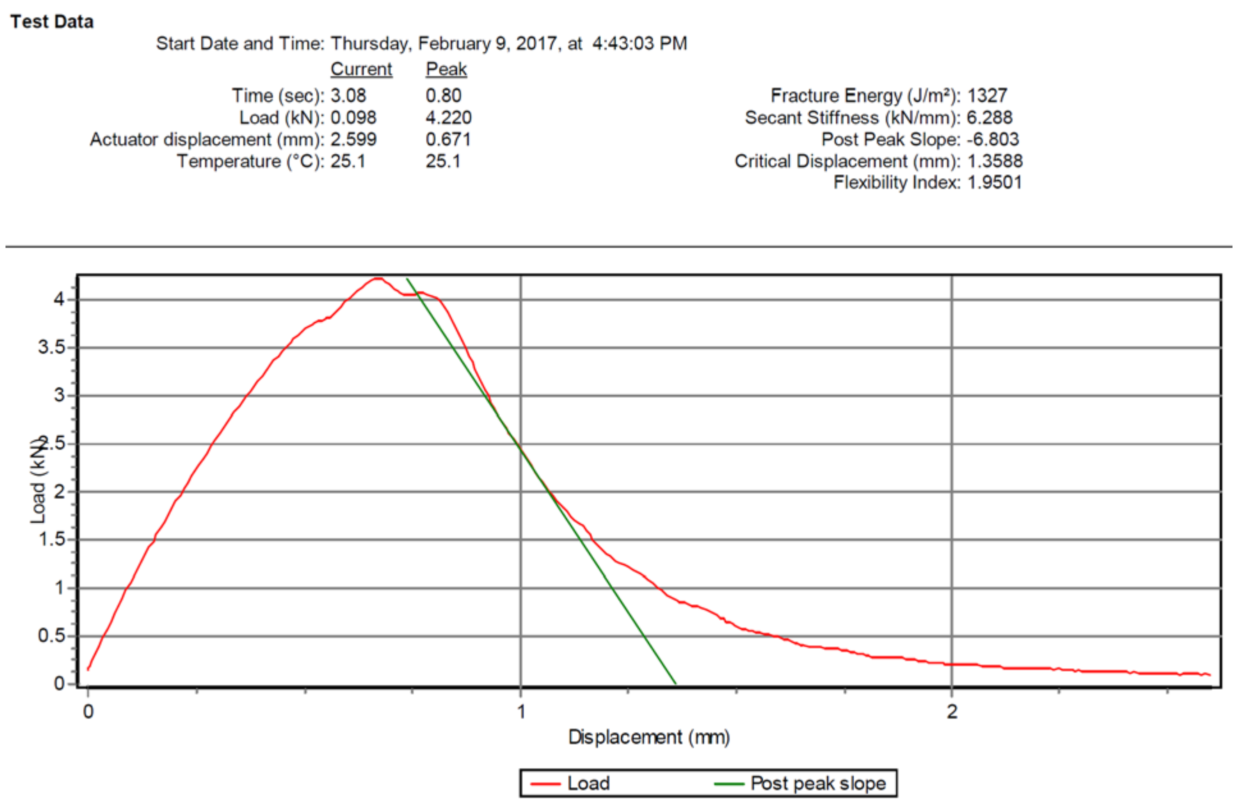


Figure 4.1: Typical Output from SCB Test Software

The peak load and FE were automatically computed by the software, and the FI was obtained by adjusting the post-peak slope drawn at an inflection point to match most of the load-displacement curve after the peak point. Test results are tabulated in Table 4.1, and Figure 4.2 and Figure 4.3 show the results graphically.

Table 4.1: SCB Test Results

Mix ID	Recycled Material Content	Asphalt Content (%)	Air Voids (%)	Peak Load (kN)	Fracture Energy (kJ/m2)	COV (%)	Flexibility Index	COV (%)
D1	10% RAP + 5% RAS	6.67	6.90	3.92	1.64	5.31	2.63	6.0
D3	25% RAP	5.60	6.65	3.25	2.07	5.35	8.67	12.5
D4	10% RAP + 5% RAS	6.30	6.94	3.90	0.93	3.97	0.93	12.3
D6	25% RAP	5.20	6.93	4.59	1.98	6.18	2.70	11.0

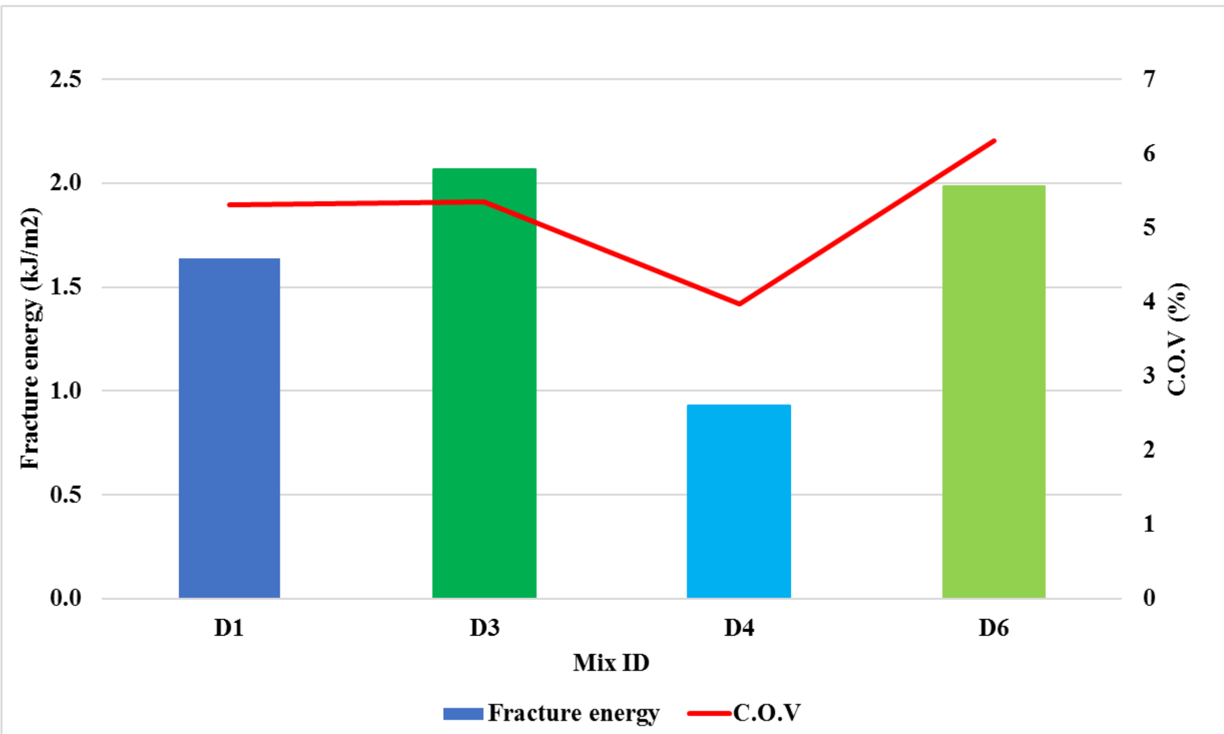


Figure 4.2: Test Results of FE and COV

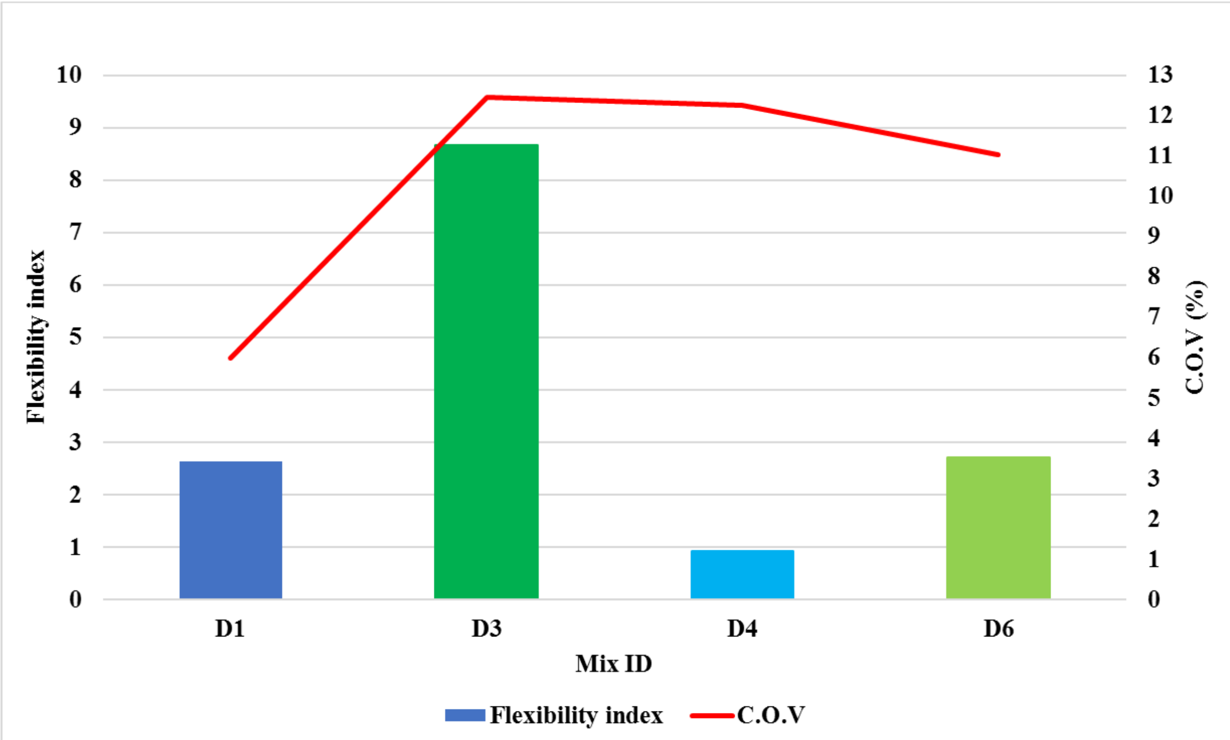


Figure 4.3: Test Results of FI and COV

Results in Figure 4.2 show that mixtures with 25% RAP and no RAS had relatively higher FE values. When comparing the mixtures with identical percentages of RAP and RAS (D1 and D4 or D3 and D6), the mixture with higher asphalt content showed higher FE values. Because FE indicates material stiffness, the mixture with higher FE is expected to have a higher stiffness value. The coefficient of variation (COV) of FE in the SCB test was less than 10%, indicating very good repeatable results.

FE value alone does not offer a definitive conclusion regarding cracking performance, so FI has been developed to determine the cracking potential of asphalt mixtures from the SCB-IL test. Results in Figure 4.3 indicate that mixtures with 25% RAP had higher FI. When comparing mixtures with identical recycled asphalt content, the mixture with higher asphalt content had higher FI. Al-Qadi et al. (2015) reported that FI values between 2.0 and 6.0 qualify a mixture as crack resistant; thus, mixtures D1 and D6 were shown to be acceptable. Mixture D4 was too stiff due to the presence of RAP, and mixture D3 had a very high FI value, 8.67, indicating the mixture was soft and would perform better. The COV of FI was higher because the FI was derived from the shape of the post-peak segment of the load-displacement curve. Based on results presented in Figure 4.3, the COV values of FI were within the range of 10%–20%.

FE and FI results showed that mixtures with 25% RAP and no RAS will have increased cracking resistance. When mixtures have identical amounts of recycled materials, the mixture with higher asphalt content is less susceptible to cracking.

4.2 Florida Indirect Tensile Strength Test Results

4.2.1 Resilient Modulus Test Results

The resilient modulus test was conducted to assess resilient characteristics of the asphalt mixtures according to ASTM D7369 (2011), with typical output illustrated in Figure 4.4.

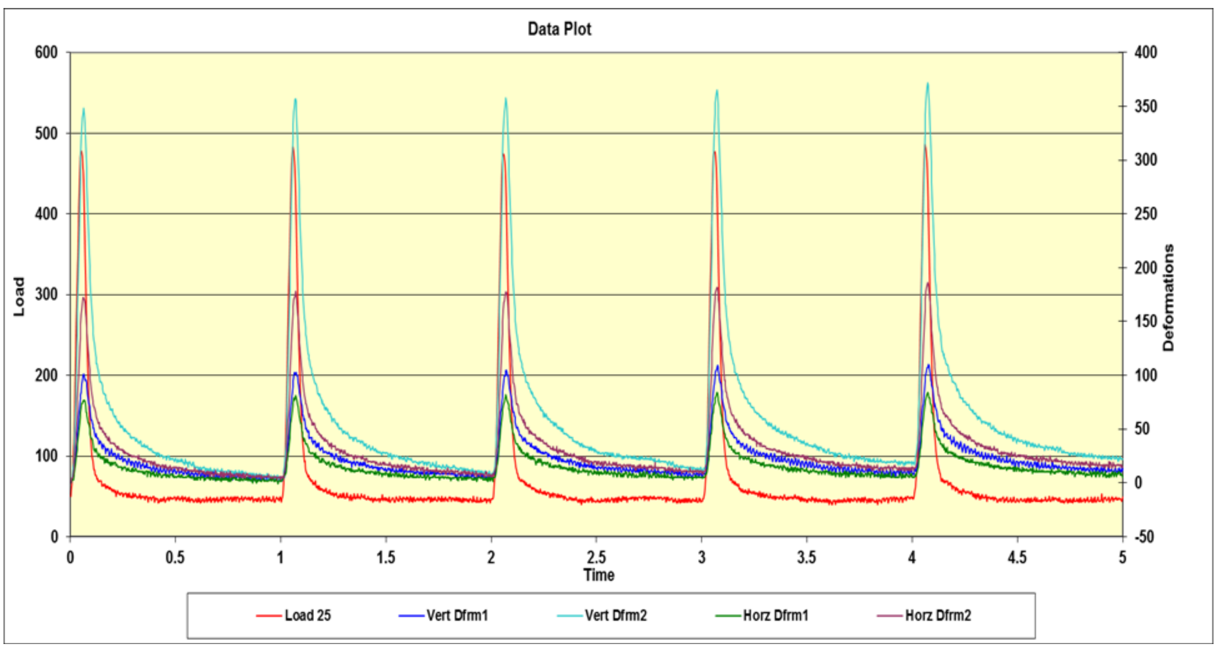


Figure 4.4: Typical Output of Resilient Modulus from IDT Test Software

Figure 4.4 shows a typical output of the resilient modulus test using the IPC asphalt tester software. Load and deformation information were collected during testing, and the instantaneous and total resilient modulus were automatically calculated by the software. Test results are presented in Table 4.2, and Figure 4.5 shows the test results for all mixtures.

Table 4.2: Resilient Modulus Test Results

Mix ID	Recycled Material Content	Asphalt Content (%)	Air Voids (%)	Instantaneous Resilient Modulus (GPa)	Total Resilient Modulus (GPa)
D1	10% RAP + 5% RAS	6.67	7.39	5.05	3.79
D3	25% RAP	5.60	6.81	2.16	1.59
D4	10% RAP + 5% RAS	6.30	7.20	4.05	3.11
D6	25% RAP	5.20	7.38	3.02	2.36

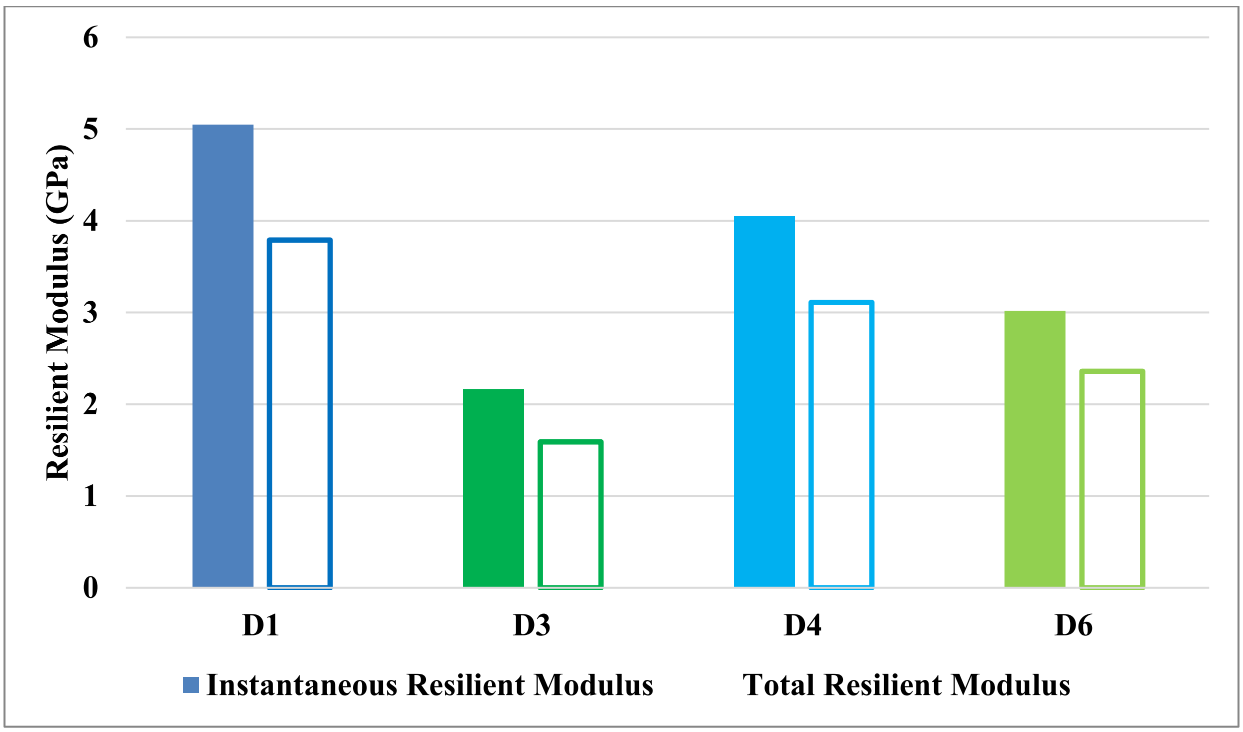


Figure 4.5: Resilient Modulus Test Results

Figure 4.5 shows that mixtures D1 and D4 with 10% RAP and 5% RAS had relatively high resilient moduli, meaning that those mixtures were stiffer than the other two mixtures. Mixture D3, which had the highest FI, also had the lowest resilient modulus.

4.2.2 Creep Compliance Test Results

The creep compliance test was performed following AASHTO T 322 (2011). Load and deformation information depending on time were recorded by the test software. Average deformations were manually calculated by averaging the deformation data sets after deducting the highest and lowest values (Du, 2010). Then, using Equation 2.4 from Chapter 2, a creep compliance curve was obtained, as shown in Figure 4.6.

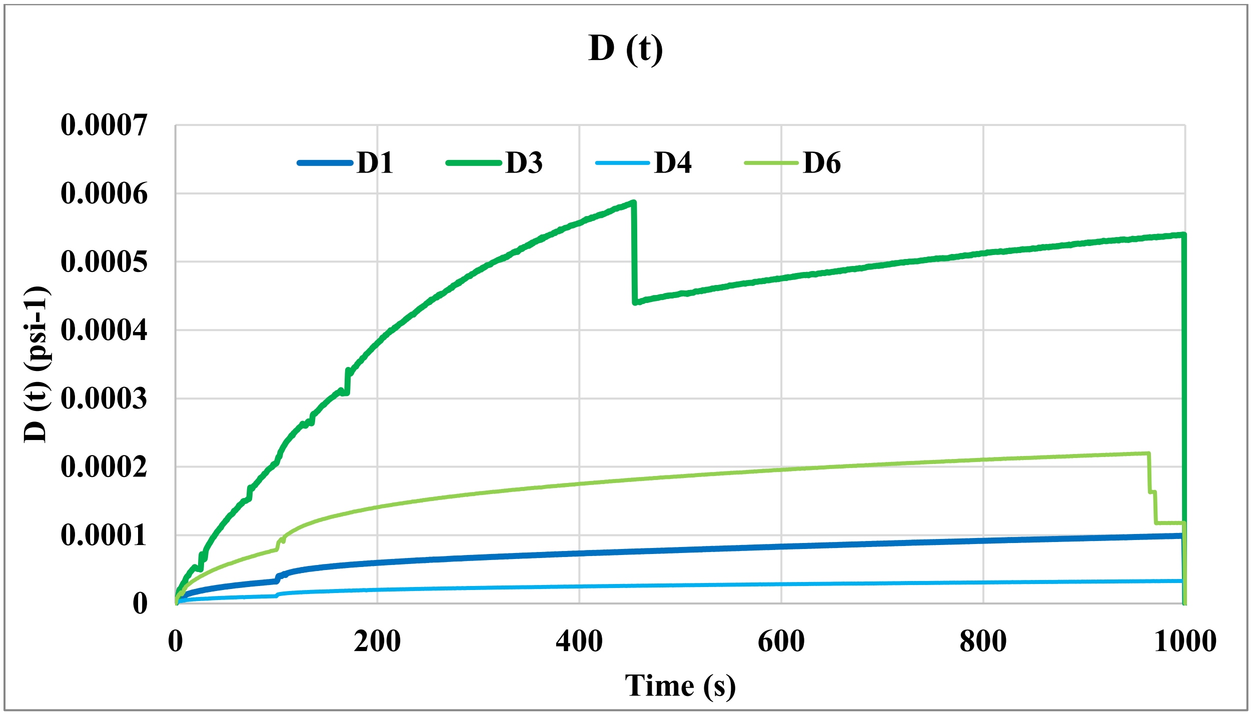


Figure 4.6: Creep Compliance Curve

As shown in Figure 4.6, mixtures with high recycled material content (D3 and D6 with 25% RAP) showed high creep compliance; mixture D3 had the highest creep compliance. The data spikes in Figure 4.6 were due to the unavoidable large particles near the LVDTs, resulting in deformation saltation. D_1 and m-value were creep compliance power law parameters and were calculated to determine the $DCSE_{min}$ values. Results are listed in Table 4.3, which shows that mixtures D3 and D6 with 25% RAP had low m-values, indicating a low damage accumulation rate of the asphalt mixture.

Table 4.3: D_1 and m-value

Mix ID	Recycled Material Content	D_1	m-value
D1	10% RAP + 5% RAS	4.21×10^{-6}	0.30
D3	25% RAP	4.36×10^{-6}	0.14
D4	10% RAP + 5% RAS	2.08×10^{-6}	0.30
D6	25% RAP	6.00×10^{-6}	0.19

4.2.3 Indirect Tensile Strength Test Results

IDT was computed using Equation 2.5, and results are shown in Table 4.4 and Figure 4.7. Figure 4.7 shows that mixture D4 with 25% RAP had the highest IDT. IDT values were used to compute the $DCSE_{min}$.

Table 4.4: Indirect Tensile Strength Test Results

Mix ID	Recycled Material Content	Asphalt Content (%)	Indirect Tensile Strength (MPa)
D1	10% RAP + 5% RAS	6.67	0.78
D3	25% RAP	5.60	0.67
D4	10% RAP + 5% RAS	6.30	0.92
D6	25% RAP	5.20	0.89

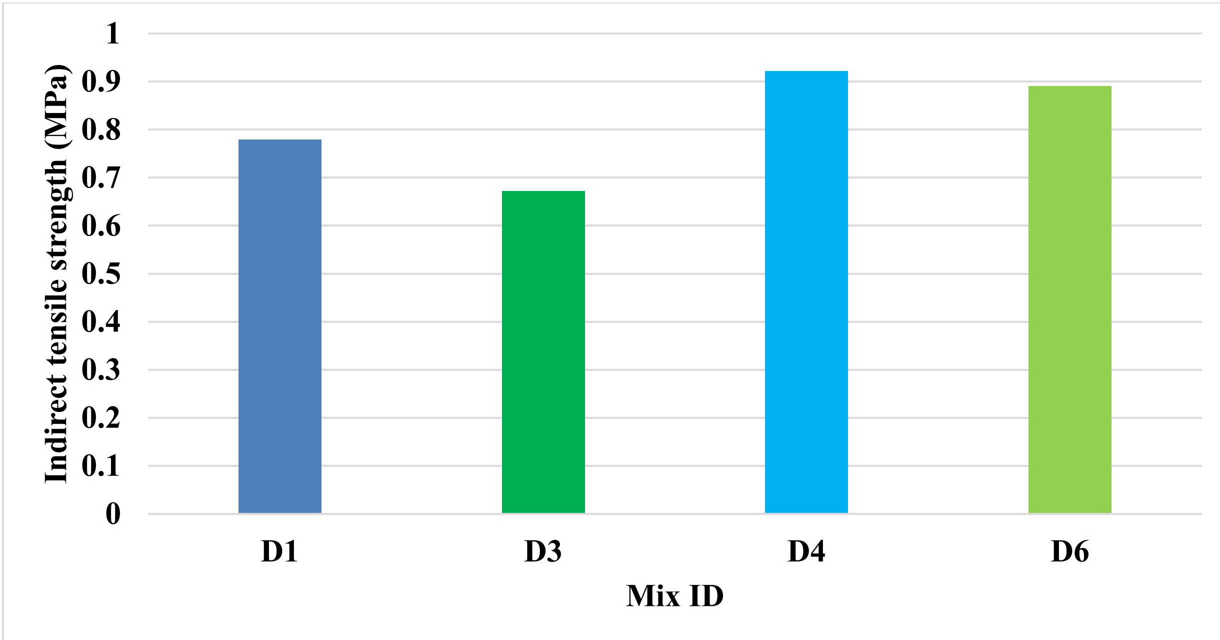


Figure 4.7: Indirect Tensile Strength Test Results

4.2.4 Fracture Energy, Elastic Energy, Dissipated Creep Strain Energy, and Energy Ratio

Using the stress-strain curve obtained from the IDT test, FE and EE were calculated using Equations 2.8 and 2.9. Figures 4.8 and 4.9 illustrate that mixtures with 25% RAP had relatively higher FE and EE. DCSE at failure was computed by subtracting EE from FE. Figure 4.10 illustrates the $DCSE_f$, showing that mixtures D3 and D6 with 25% RAP had high $DCSE_f$, indicating high fatigue-cracking resistance.

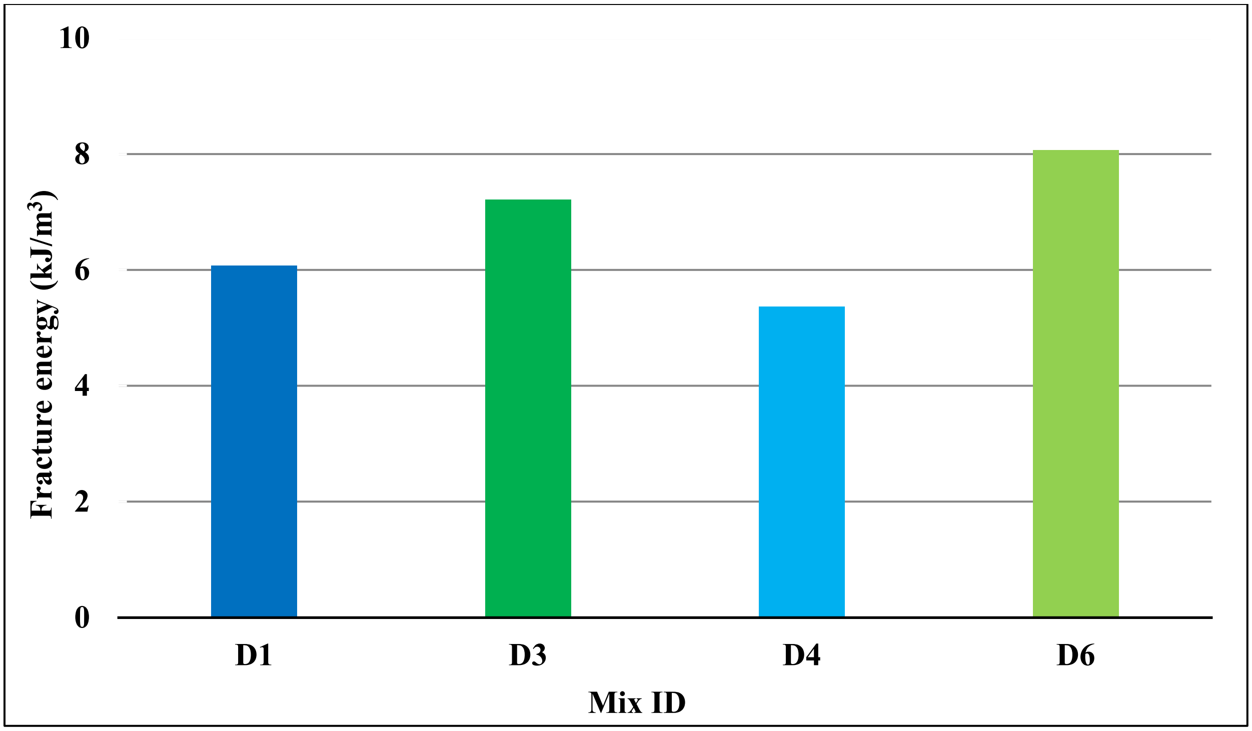


Figure 4.8: FE Results

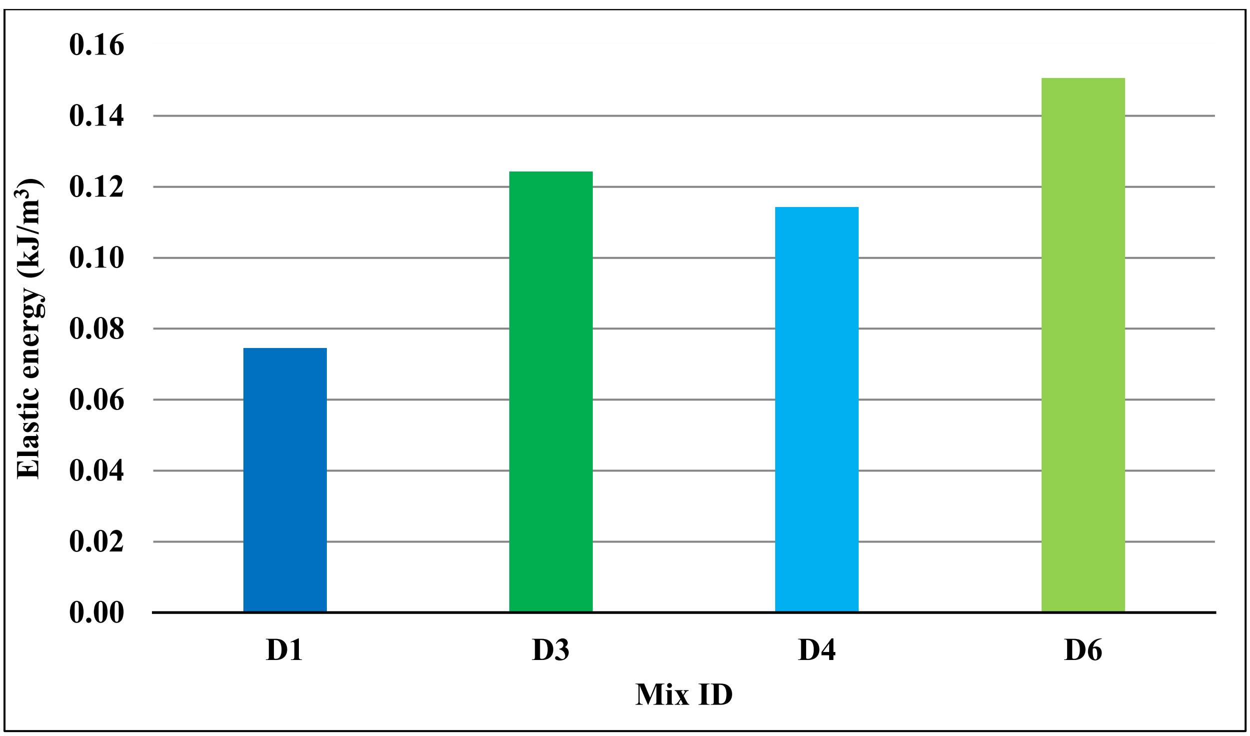


Figure 4.9: EE Results

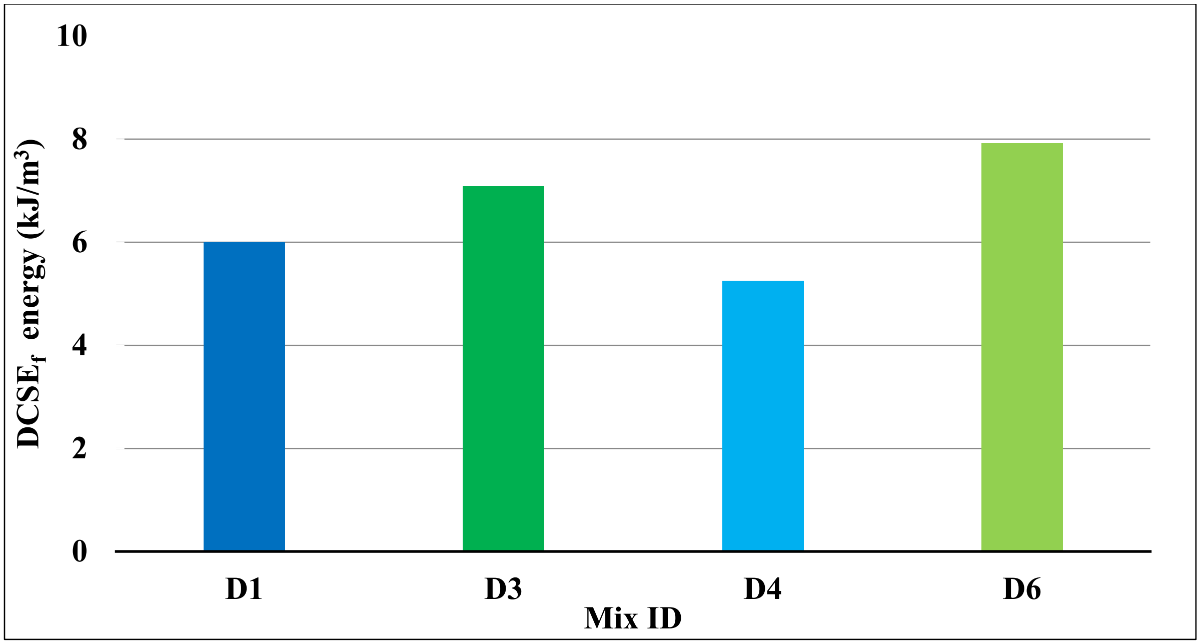


Figure 4.10: DCSE$_f$ Results

DCSE$_{min}$ was computed using Equations 2.11 and 2.12; results are shown in Figure 4.11. The DCSE$_{min}$ showed an opposite trend from the DCSE$_f$ results: Mixtures D3 and D6 with 25% RAP had relatively low DSCE$_{min}$, indicating that these two mixtures required less energy to resist damage.

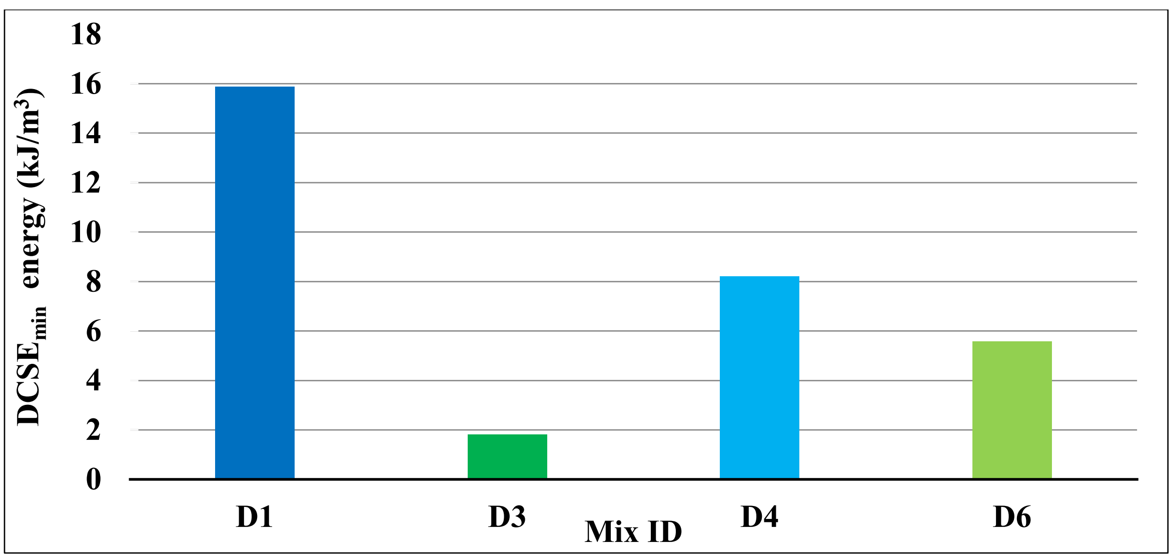


Figure 4.11: DCSE$_{min}$ Results

Finally, ER was calculated by dividing $DCSE_f$ by $DSCE_{min}$, with results presented in Figure 4.12. The ER of mixtures D3 and D6 with 25% of RAP was higher than 1, indicating that the two mixtures were less susceptible to cracking. Mixtures D1 and D4, which had 10% RAP and 5% RAS, had ERs less than 1, indicating that those mixtures would fail.

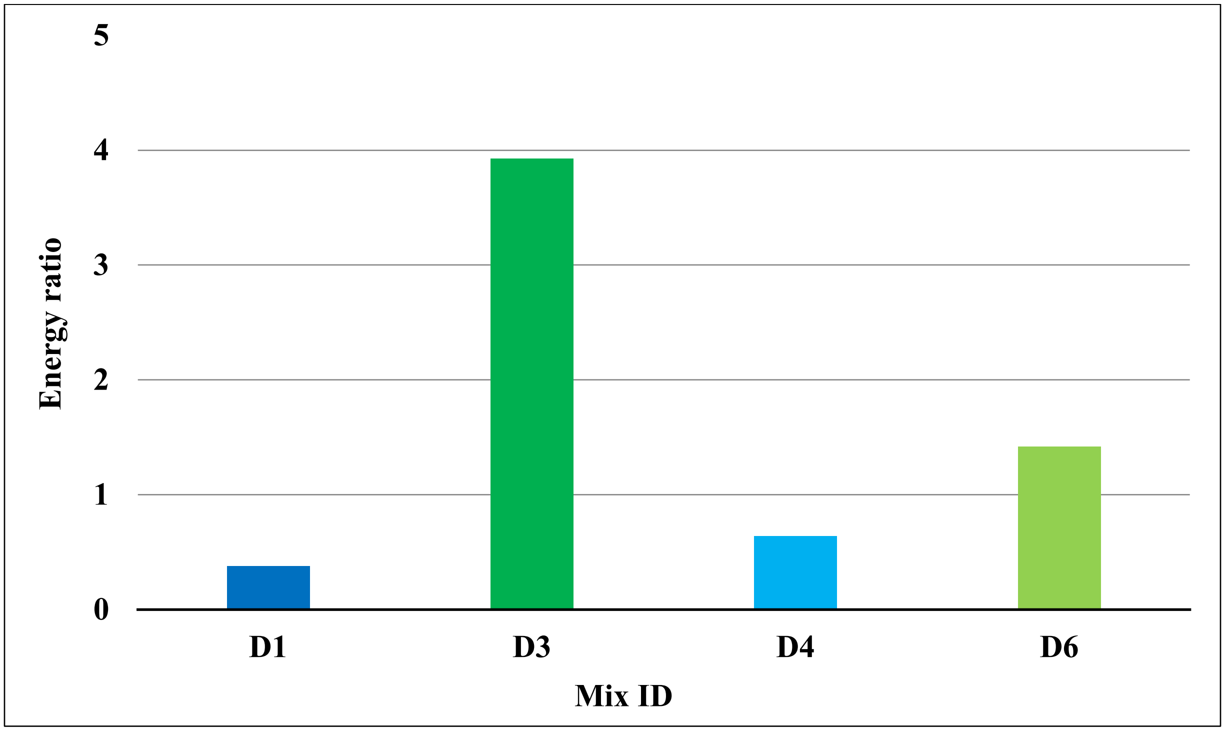


Figure 4.12: ER Results

A summary of fracture properties of the test mixes is shown in Table 4.5. In general, a threshold of ER = 1 distinguishes good and poor field performances of a mixture, and higher ERs indicate improved fracture resistance. Furthermore, FE determines cracking resistance of asphalt mixtures. According to Roque et al. (2004), a mixture with an ER greater than 1 and an FE < 0.75 could fail. Conversely, when the ER is less than 1 but the mix has an FE > 2.5, the mix should perform better in cracking. As shown in Table 4.5, the ER of mixtures with low contents of recycled material were less than 1, and no FE values were greater than 2.5, indicating that these mixtures were prone to cracking. Mixtures with high contents of recycled materials demonstrated high ERs, with values greater than 1, and FE values greater than 0.75, meaning that these two mixtures would resist cracking. Mixture D3 had the highest ER and, consequently, highest cracking resistance.

Table 4.5: Fracture Properties Results

Mix ID	Recycled Material Content	Asphalt Content (%)	Fracture Energy (kJ/m^3)	Elastic Energy (kJ/m^3)	DCSE$_f$ (kJ/m^3)	DCSE$_{min}$ (kJ/m^3)	Energy Ratio
D1	10% RAP + 5% RAS	6.67	6.07	0.07	6.00	15.87	0.38
D3	25% RAP	5.60	7.21	0.12	7.09	1.80	3.93
D4	10% RAP + 5% RAS	6.30	5.36	0.11	5.25	8.20	0.64
D6	25% RAP	5.20	8.07	0.15	7.92	5.57	1.42

4.3 Statistical Analysis

This study utilized Statistical Analysis System (SAS)® software to perform statistical analysis of SCB test. Results of the cracking tests were analyzed using a two-way, nested design, as shown in Table 4.6.

Table 4.6: Nested Design

District				
Mix	1	4	3	6
	A*	A*	B**	B**

* HMA with 10% RAP + 5% RAS
** HMA with 25% RAP + 0% RAS

The design treatments were asphalt mix (two levels: Mix A and Mix B) and districts (four levels: District 1, 3, 4, 6). The nested design was used because Mix A was available only from District 1 and District 4, and Mix B was available from District 3 and District 6. In other words, levels of factor mix were nested within levels of factor district and have one or more observations on each district (mix) combination. The following model was used:

$$y_{ijk} = \mu + \alpha_i + \beta(\alpha)_{ij} + \varepsilon_{ijk}$$ **Equation 4.1**

Where:

y_{ijk} is the response variable;

μ is the intercept;

α_i is the effect of the i^{th} level of mix, i =1,2;

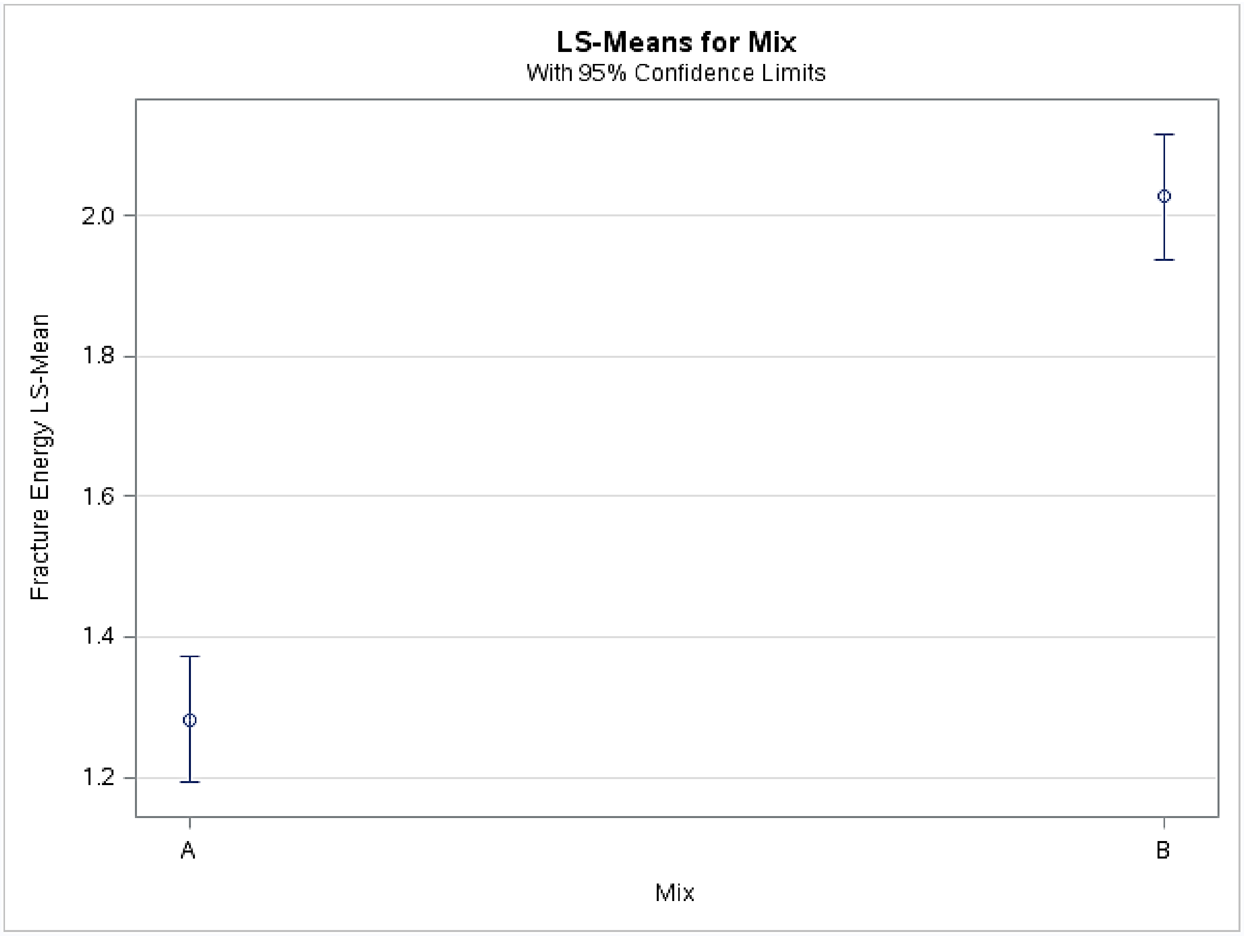
β is the effect of the j^{th} level of district, j = 1, 2, 3, 4; and

ε_{ijk} is the response (FE/FI) error for the k^{th} sample from the i^{th} mix and j^{th} district.

Instead of the analysis of variance (ANOVA) approach, a generalized linear mixed model (GLMM) was used in this analysis to compare performance differences of the two mixture types. Two mix types were randomly collected from four districts, where mixes were within the levels of district. Thus, the GLMM method accounted for the random effect of the district. A GLMM approach enables statisticians to incorporate both fixed and random effects in a model. In this analysis, α (effect of the i^{th} level of mix) brought forth the fixed affect and $\beta(\alpha)$ accounted for the random effect, where β represented the effect of the j^{th} level of district.

The goal of this analysis was to determine performance differences of Mixes A and B. Instead of arithmetic mean (simple average of the values), least square means (LSmeans) of the response variables were used to discriminate among the mixes. LSmeans, or marginal means, were adjusted for other terms in the model, such as covariates and blocking factor (Stroup, 2012).

Results of the statistical analysis with FE as the response variable are shown in Figure 4.13, illustrating significant differences between the LSmeans of the FE. Mixture B (25% RAP + 0% RAS) demonstrated higher cracking resistance (0.7438 ± 0.13 KN/mm2 FE) than Mixture A (10% RAP + 5% RAS).

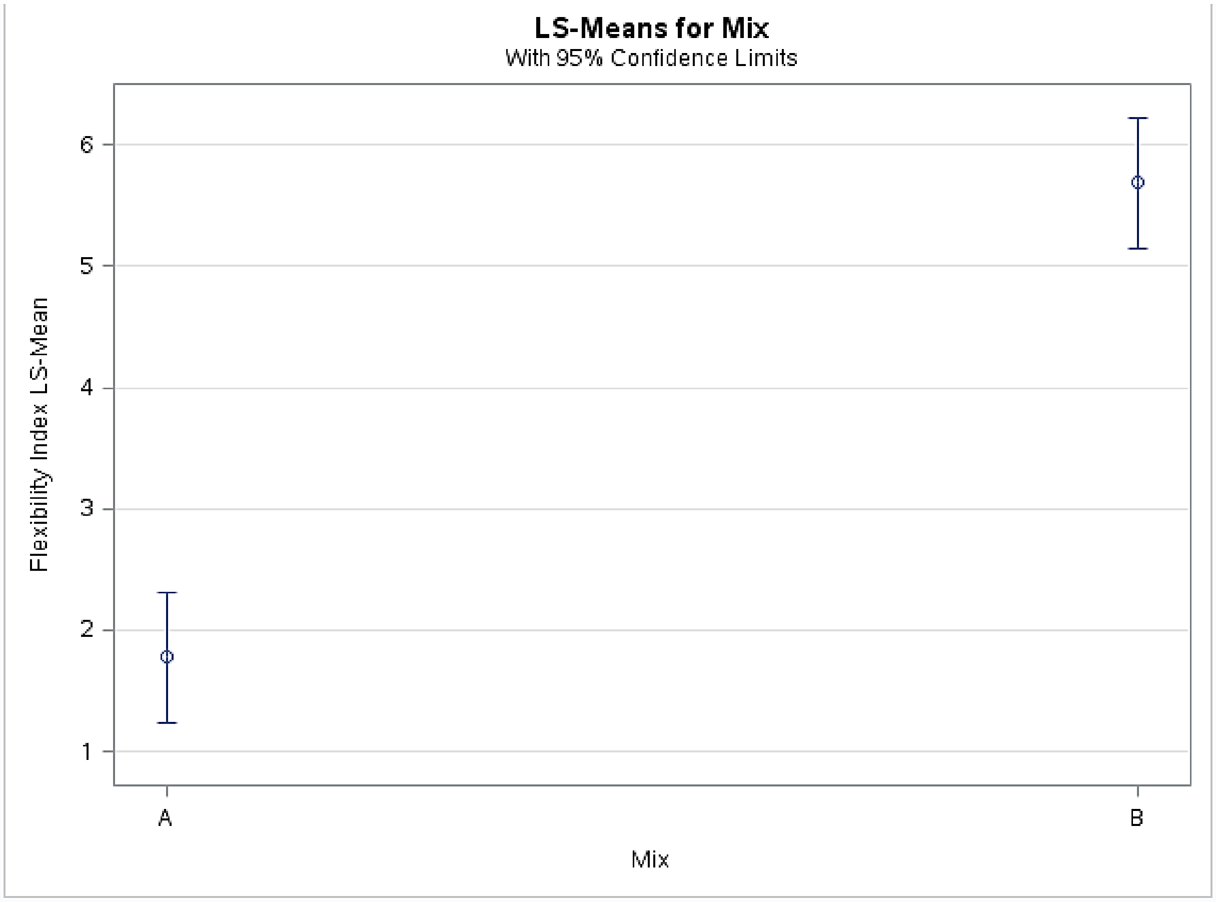


Differences of Mix Least Squares Means
Adjustment for Multiple Comparisons: Tukey

Mix	Mix	Estimate	Standard Error	DF	t Value	Pr > |t|	Adj P	Alpha	Lower	Upper	Adj Lower	Adj Upper
A	B	-0.7438	0.05492	8	-13.54	<.0001	<.0001	0.05	-0.8705	-0.6172	-0.8705	-0.6172

Figure 4.13: Statistical Analysis Results of FE

Analysis was also done using FI as the response variable, with results shown in Figure 4.14. A significant difference was observed between the LSmeans of FI of the two mixes. Mixture B (25% RAP + 0% RAS) had 3.9048±0.13 unit higher FI than Mixture A (10% RAP + 5% RAS), proving that the inclusion of RAS in the recycled mixture is detrimental to cracking resistance.



Differences of Mix Least Squares Means Adjustment for Multiple Comparisons: Tukey												
Mix	**Mix**	**Estimate**	**Standard Error**	**DF**	**t Value**	**Pr > |t|**	**Adj P**	**Alpha**	**Lower**	**Upper**	**Adj Lower**	**Adj Upper**
A	B	-3.9084	0.3286	8	-11.89	<.0001	<.0001	0.05	-4.6661	-3.1507	-4.6661	-3.1507

Figure 4.14: Statistical Analysis Results of FI

Chapter 5: Conclusions and Recommendations

5.1 Conclusions

The objective of this research was to investigate cracking resistance of asphalt mixtures with varying recycled material content. Four mixtures with various recycled contents were tested using IL-SCB and FL-IDT tests. The following conclusions were made from analysis of test results:

- IL-SCB test results indicated that mixtures with 25% RAP but no RAS have higher FE and FI, indicating increased cracking resistance. However, with the identical recycled material content, mixtures with higher percentages of asphalt are less susceptible to cracking.
- Based on FL-IDT test results, mixtures containing 25% RAP have a relatively lower resilient modulus but higher creep compliance, $DCSE_f$, as well as higher energy ratio. The former parameter indicates that the mixture with a lower resilient modulus is softest, and the latter two parameters indicate that the mixture with 25% RAP and no RAS is less susceptible to cracking.

5.2 Recommendations

Based on this study, the following recommendations are made:

- All mixtures in this study were collected from different sources, allowing the potential of some unknown variables to affect test results. Future studies should obtain mixtures from one source and develop an experimental plan with various recycled material contents to isolate main factors.
- Mixtures containing the same amount of RAP but different percentages of RAS should be tested for cracking resistance in order to evaluate the effect of RAS content on cracking performance.

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