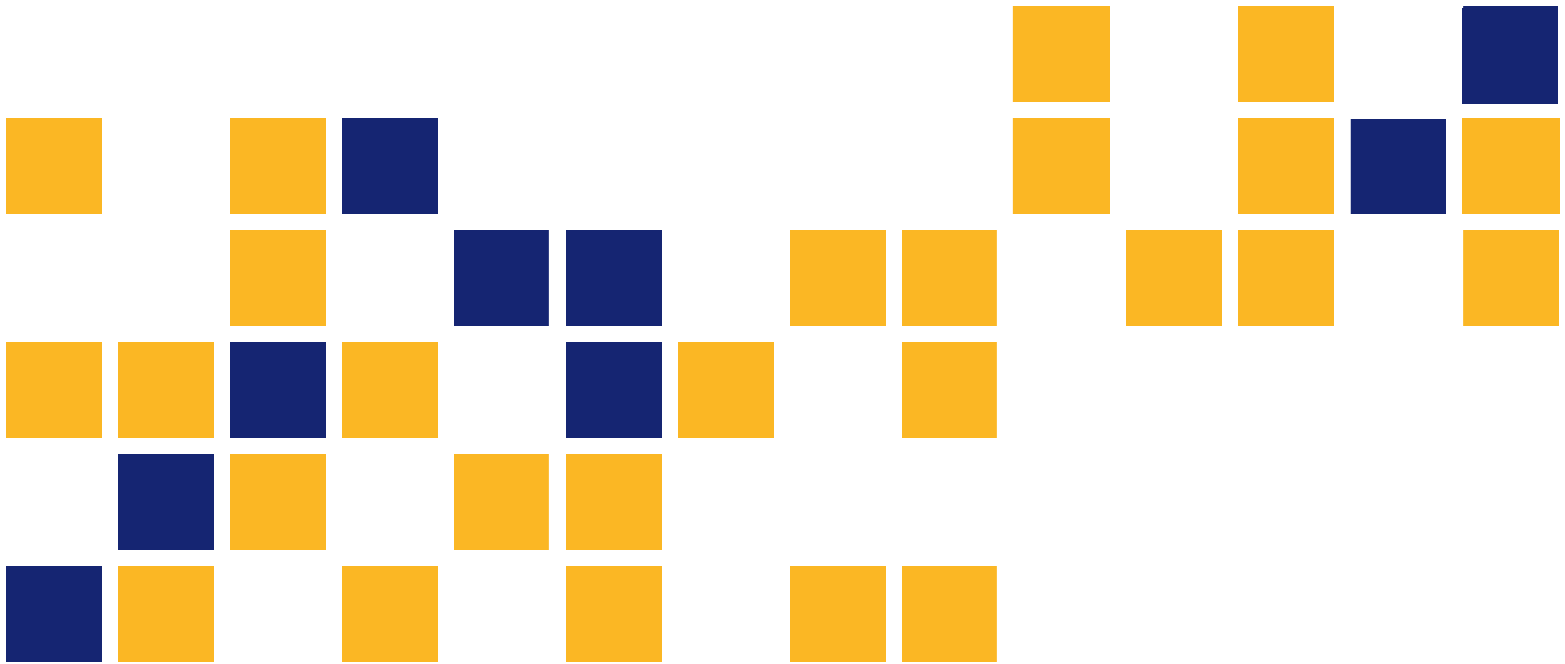


# Durable Superpave Mixes in Kansas

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<p>Approximately 89% of 11,000 miles of Kansas roads are surfaced with asphalt. Hundreds of thousands of tons of reclaimed asphalt pavement (RAP) are produced annually in the United States, including in Kansas. This bulk volume of RAP must be economically managed in order to achieve environmental friendliness. Recycling of RAP conserves natural resources and reduces landfill usage. However, many agencies have reported that increased RAP content produces drier hot-mix asphalt (HMA) mixtures than virgin mixtures that are susceptible to premature cracking.</p> <p>In this research, laboratory-produced Superpave HMA mixtures containing increased percentages (20, 30, and 40%) of RAP materials from three RAP sources (Shilling Construction Co., Konza Construction Co., and the Kansas Department of Transportation project, US-73) were studied for cracking performance. Mix designs were produced using Superpave design criteria for 12.5-mm nominal maximum aggregate size mixture. The static and repetitive Semicircular Bending (SCB) test, the Texas Overlay Tester test, the dynamic modulus test, and Simplified Viscoelastic Continuum Damage (S-VECD) tests were performed on laboratory-prepared samples. In general, cracking performance decreased with increased RAP content. The RAP from the US-73 project performed most consistently compared to other two sources of RAP. Test results were analyzed using two-way Analysis of Variance (ANOVA), proving that mixtures containing 4.5% to 4.9% binder performed the best against cracking. The RAP source was found to have more effect on cracking propensity than RAP content. Mixtures with RAP content up to 40% performed satisfactorily. Tukey's pairwise comparison method was used to compare results from all tests; S-VECD was determined to be the most appropriate test to evaluate cracking propensity of HMA mixtures.</p>			
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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

Approximately 89% of 11,000 miles of Kansas roads are surfaced with asphalt. Hundreds of thousands of tons of reclaimed asphalt pavement (RAP) are produced annually in the United States, including in Kansas. This bulk volume of RAP must be economically managed in order to achieve environmental friendliness. Recycling of RAP conserves natural resources and reduces landfill usage. However, many agencies have reported that increased RAP content produces drier hot-mix asphalt (HMA) mixtures than virgin mixtures that are susceptible to premature cracking.

In this research, laboratory-produced Superpave HMA mixtures containing increased percentages (20, 30, and 40%) of RAP materials from three RAP sources (Shilling Construction Co., Konza Construction Co., and the Kansas Department of Transportation project, US-73) were studied for cracking performance. Mix designs were produced using Superpave design criteria for 12.5-mm nominal maximum aggregate size mixture. The static and repetitive Semicircular Bending (SCB) test, the Texas Overlay Tester test, the dynamic modulus test, and Simplified Viscoelastic Continuum Damage (S-VECD) tests were performed on laboratory-prepared samples. In general, cracking performance decreased with increased RAP content. The RAP from the US-73 project performed most consistently compared to other two sources of RAP. Test results were analyzed using two-way Analysis of Variance (ANOVA), proving that mixtures containing 4.5% to 4.9% binder performed the best against cracking. The RAP source was found to have more effect on cracking propensity than RAP content. Mixtures with RAP content up to 40% performed satisfactorily. Tukey's pairwise comparison method was used to compare results from all tests; S-VECD was determined to be the most appropriate test to evaluate cracking propensity of HMA mixtures.

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# Chapter 1: Introduction

## 1.1 Background

Asphalt pavement recycling has become an integral part of pavement reconstruction and resurfacing in the United States and in many parts of the world. Scarcity of quality aggregates and increasing environmental concerns are intensifying the need for pavement recycling. Existing asphalt pavement materials are commonly removed during resurfacing, rehabilitation, and reconstruction operations. In the past, rehabilitation refuse of asphalt pavement was disposed of in landfills; however, recent principles of sustainable transportation systems have gained popularity and raised concerns regarding these landfills and their environmental impact.

Once removed and processed, pavement materials become reclaimed asphalt pavement (RAP) that contains valuable asphalt binder and aggregate, as shown in Figure 1.1. This RAP can replace expensive virgin aggregates and binders in HMA mixtures. According to the National Asphalt Pavement Association (NAPA), approximately 33 million metric tons of RAP materials are produced in the United States each year (NAPA, n.d.). The Federal Highway Administration (FHWA) recommends use of RAP for all new construction and rehabilitation work of asphalt pavements in order to achieve better or equal performance with maximum sustainability. According to FHWA, the use of RAP is economical for the intermediate and surface layers of flexible pavements in which the aged binder from RAP can replace a portion of virgin binder.



**Figure 1.1: Reclaimed Asphalt Pavement from Different Sources and with Various Gradations**

Although RAP has been in use for the last three decades, recent interest has arisen regarding use of higher quantities of RAP. By definition, mixtures with high RAP content contain more than 25% RAP by weight of the mixture (FHWA, 2009).

Initially, the use of RAP in roadwork was confined to low-volume roads, but RAP use has become increasingly more widespread. In 2007, typical hot-mix asphalt (HMA) mixture contained approximately 12% RAP. From 2007 to 2009, approximately 27 states increased the amount of RAP permitted in asphalt mixtures, and as of 2009, 23 states gained experience about utilizing high-RAP mixtures. In 2011, over 40 state highway agencies allowed more than 30% RAP; however, only 11 states reported practically using 25% RAP or more (FHWA, 2009). However, surface layer quality is of primary concern for state Departments of Transportation (DOTs). Although use of RAP in unbound layers has been practiced widely in the United States, the main factors influencing reluctance in accepting RAP usage include uncertainties in mix design, lack of information about production technology used for RAP sources, and uncertainties of resultant mixture qualities (Zaumanis & Mallick, 2015). Research conducted by McDaniel, Soleymani, Anderson, Turner, and Peterson (2000) investigated the inclusion of RAP in the Superpave mix design, resulting in the development of Procedure M 323 by the American Association of State Highway and Transportation Officials (AASHTO M 323-13, 2013). Institutionalizing RAP use, in addition to the rising price of virgin binder, has resulted in increased quantities of RAP use in new HMA mixtures.

In 2009, the North Carolina DOT conducted a survey for AASHTO in order to evaluate RAP use by state DOTs, including the Province of Ontario, Canada. According to the survey, four factors were identified that prevent DOTs from using high-RAP quantities in HMA mixtures: (1) specification limitations, (2) variability of RAP, (3) lack of availability of RAP, and (4) lack of experience with RAP. The survey also evaluated performance of pavement sections with RAP. Most agencies reported that HMA mixtures containing RAP demonstrated poorer cracking performance than pavements with no RAP, including fatigue and low-temperature cracking, but permanent deformation or rutting performance was improved (FHWA, 2009).





**Figure 1.2: Pavement Milling**

Figure 1.2 illustrates the production process of RAP. Existing pavements typically are scraped by a milling machine with carbide teeth/cutting drums, transported to plants in order to be crushed into convenient sizes and gradations, and then transported to desired locations.

## **1.2 Problem Statement**

The Kansas Department of Transportation (KDOT) is increasingly allowing recycled materials in Superpave HMA mixtures. As materials and construction costs continue to increase, KDOT and the paving industry seek to include more RAPs in asphalt pavements. However, similar to many other state highway agencies in the United States, some Kansas roads built with high-RAP mixtures have experienced premature cracking and deterioration which affects the long-term durability of these pavements. An example of premature cracking on a 1-year old pavement is shown in Figure 1.3. In general, Hamburg Wheel Tracking Device (HWTD) test results for a virgin HMA mixture with no RAP have been consistent from sub-lot to sub-lot. However, test results for HMA mixtures with high RAP percentages vary significantly, as shown in Table 1.1, in which consecutive sub-lots/lots demonstrated unpredictably different results with only 24% RAP. Another mix with 35% RAP performed better than the mix with 24% RAP. This

trend was also evident for high-RAP mixtures evaluated at Kansas State University. RAP variability is thought to be responsible for performance variability of HMA mixtures with high RAP content, including early cracking. In addition, RAP that is currently produced differs from previous generations because recycled Superpave pavements have better aggregates and binder. These RAP sources are expected to be less variable and highly valuable. However, durability of mixtures containing high RAP content still needs to be ensured.



**Figure 1.3: Low-Severity Transverse Cracking on US-83 in Thomas County, KS**

**Table 1.1: Summarized Hamburg Wheel Test Results of SR-12.5A with High RAP Content**

<b>Mix No./Lot</b>	<b>RAP (%)</b>	<b>Average Number of Passes</b>	<b>Average Rut Depth (mm)</b>
1/LOT-6A	24	16,070	11.2
1/LOT-6C	24	20,000*	19.38
2/LOT-1D	35	20,000*	3.36

\* reached maximum number of passes

### **1.3 Objectives**

The following objectives were pursued in this study:

1. Evaluate cracking resistance of Superpave mixtures with higher RAP content.
2. Evaluate the effect of RAP source or RAP quality on the cracking potential of mixtures.
3. Establish minimum asphalt binder content and maximum RAP content to ensure satisfactory cracking resistance.

### **1.4 Organization of Report**

This report is divided into five chapters, including this Introduction. Chapter 2 provides a literature review on RAP use, durability issues associated with pavements containing RAP, and cracking evaluation tests performed for this study. Chapter 3 presents laboratory mixture design and testing methods, and Chapter 4 presents test results and statistical analysis. The conclusion and recommendations for future study are summarized in Chapter 5.

# Chapter 2: Literature Review

## 2.1 Introduction

Research on RAP materials in new construction and pavement rehabilitation continues to be a topic of interest for the pavement industry. The amount of RAP that can be accepted to add in new HMA mixtures without compromising long-term pavement performance has been and continues to be extensively explored in many studies. A majority of states in the United States currently use 15% to 25% RAP in new HMA. This chapter presents a comprehensive literature review in order to assess the effects of RAP use on physical, chemical, and volumetric properties of HMA mixtures. Because RAP is a source of asphalt binder, it is expected to reduce the quantity of virgin binder used in a mixture. Benefits and challenges of using RAP as an aggregate source are also discussed in this chapter. Primary distresses on flexible pavement are rutting and cracking; this literature review also summarizes previous work associated with four cracking evaluation tests.

## 2.2 Reclaimed Asphalt Pavement

Reclaimed Asphalt Pavement (RAP) is the term given by the Federal Highway Administration (FHWA) to removed and/or reprocessed pavement materials which contain asphalt binder and aggregates. These materials are removed from existing road surfaces for reconstruction or resurfacing or to obtain access to buried utilities. To ensure better quality and stable gradation, proper crushing and screening of RAP is done. RAP is produced by removing existing asphalt pavement layers via milling or full-depth removal. Pavement milling machines typically remove up to 50-mm thicknesses in a single pass. For full-depth removal, pavements are broken into small, manageable segments and then ripped off using a rhino horn or pneumatic pavement breakers. Scraped-off road segments are transferred to RAP plants in order to undergo a series of operations, including crushing, screening, conveying, and stock piling. Hot-in-place and cold-in-place recycling are other popular methods of recycling and are typically applied with required additives (Bonaquist, 2007; Copeland, 2011).

## 2.3 Why Use RAP?

The following factors contribute to increased RAP usage in new construction and rehabilitation projects of pavements:

- **Increased cost of virgin materials:** The cost of virgin asphalt binder has increased significantly since the oil crisis in the mid-1970s (Karlsson & Isacson, 2006). Binder costs increased more than 300% by 2008. A typical asphalt mixture consists of 95% aggregate and 5% asphalt by weight, but aggregate accounts for only 30% of the total cost, while binder accounts for the other 70% (Bonte & McDaniel, 2009). However, aggregates are becoming scarce and/or unevenly distributed over the country landscape, and costs associated with processing and transporting aggregates to project locations have increased significantly due to increased energy prices.
- **Recycling of road refuse:** Each year the United States produces huge volumes of RAP. Traditionally, this refuse has been disposed of in landfills or left in stockpiles; however, chemical activity and leaching properties of RAP make it unsuitable for landfilling in large volume. Thus, large piles of RAP are scattered around. In order to reuse this material, all states use up to 25% RAP in new HMA, and the feasibility of higher RAP content is currently under study (Bonte & McDaniel, 2009). According to FHWA, 80% of RAP removed during resurfacing (approximately 73 million tons) is reused each year. Maintaining the quality of RAP is expected to increase the quantity of RAP used in new pavement construction (Shen, Amirkhanian, & Aune Miller, 2007).

## 2.4 Changes in HMA Mixture Properties Due to RAP Usage

RAP materials, before being removed from existing pavements, were subjected to thousands of wheel passes and years of weathering. Most RAPs are from pavements that were originally constructed with penetration-graded asphalt binder, not performance grade.

Performance Grade (PG) of the binder extracted from RAP is typically higher than the virgin binder required for relevant new projects. Binder performance in ductility and cohesion is typically severely altered in RAP as a result of oxidation, evaporation, exudation, and the physical hardening process (Karlsson & Isacson, 2006). Aggregates near the pavement surface sustain polishing effects of wheel passes, and binder films around aggregate particles can occasionally be worn off. Aggregates beyond the contact of wheel passes sustain changes caused by thermal gradient and variation in pressure. Chemically-active aggregates are suspected to be altered in their petrography, and a leaching effect may remove some constituents from HMA mixtures that are soluble in motor oil or water. Aggregates in RAP are also suspected to be brittle and have less structural capacity than virgin aggregates. Interlocking of aggregates and inter-particle friction are keys to maintaining load-carrying capacity in flexible pavements; polished and rounded aggregates tend to degrade these properties. Typically, blending with virgin aggregates of dominant volume suppresses this change in performance (Daniel & Lachance, 2005). However, these problems resurface if the virgin aggregate content decreases, as in the case of HMA mixtures with high RAP.

When aggregates and binder are mixed in order to obtain virgin HMA, some of the heated asphalt binder gets absorbed into the aggregate pores and the remainder of the binder creates a coating or film around the aggregates, resulting in mixture cohesion. RAP aggregates have absorption capacities and aggregate-to-binder interactions that are consistent with absorption and interaction observed in virgin mixtures. RAP typically changes volumetric and mechanistic properties of the mixture (Huang, Shu, & Tang, 2005). The degree of blending between the virgin binder and the RAP binder in mixture influences mixture properties. Although this blending is not well understood, many studies have concluded that the issue is insignificant below 25% RAP content (West, Willis, Brown, & Blow, 2014).

Daniel and Lachance (2005) studied changes in volumetric properties of HMA mixtures with increasing RAP content and compared those to that for a control HMA mixture with no RAP. The addition of RAP increased the voids in mineral aggregate (VMA) and voids filled with asphalt (VFA) of the mixtures. Increase in RAP content increased mixture stiffness but decreased

creep compliance, thereby showing that HMA mixtures with RAP are more resistant to permanent deformation and less resistant to fatigue cracking.

## **2.5 Issues Regarding RAP Use**

HMA mixtures with RAP content up to 25% have demonstrated similar results according to several researchers (Kandhal & Chakraborty, 1996; McDaniel et al., 2000; Li, Marasteanu, Williams, & Clyne, 2008; Shu, Huang, & Vukosavljevic, 2008). Most transportation agencies have reported the following issues regarding placement and performance of HMA mixture containing high quantities of RAP:

- Difficulties in attaining desired air void content and density during construction.
- Dry appearance of the mixture.
- Early cracking and raveling of the pavement surface.

Chemical agents commonly known as asphalt rejuvenating agents are currently in use or under study. Early age cracking was the focus of this study.

## **2.6 Superpave Mixture Design System**

Superpave mixture design is an outcome of the Strategic Highway Research Program (SHRP). SHRP research focused on six areas related to the road network in the United States, including an asphalt research program. Research produced a performance-based specification, or performance grade (PG), for the asphalt binder and a new mixture design system. In addition, a new compactor, Superpave Gyrotory Compactor (SGC), was used in the Superpave mixture design. Design methodology was originally planned as a performance-based HMA mix design system, and initial methodology included three levels of mix designs with increasing difficulty: Level 1, Level 2, and Level 3. Level 1 was the entry-level mixture design, but Levels 2 and 3 required performance-based mixture evaluation tests. Due to lack of suitable performance tests and levels of complexity, Levels 2 and 3 are still under research. An entry-level mixture design is currently labelled as a Superpave mixture design (Huber, 2013).

Level 1 mixture design is based on mixture parameters that define fundamental properties of the mixture, such as optimum air void content, minimum asphalt binder content, and aggregate gradation. In order to prepare specimens for volumetric analysis and performance tests, surrogate material properties were established to evaluate the performance of Superpave. Mixture performance tests are still under research in order to provide an active guideline (Huber, 2013).

## **2.7 Asphalt Pavement Durability Issues**

Durability is a definitive problem in asphalt pavement performance. Durability of a flexible pavement section indicates the ability of pavement layers to maintain structural integrity against detrimental or damaging effects caused by climate and traffic loading throughout the service life of the pavement (Nicholls, McHale, & Griffiths, 2008). According to Bonaquist (2014), a durable pavement should possess four prime properties: structural adequacy (sufficiently thick to carry traffic loading), proper drainage, proper construction, and use of proper materials for construction. The addition of recycled materials such as Reclaimed Asphalt Pavement (RAP) and Recycled Asphalt Shingles (RAS) to HMA have not yet provided definitive evidence of improved durability; instead a decrease in performance has been prominently determined in literature. As the practice of RAP material usage in HMA mixture becomes increasingly prevalent, however, uncertainties regarding durability of HMA pavements must be addressed.

### *2.7.1 Aggregate Properties*

RAP in HMA mixtures works as aggregates too. To ensure pavement durability, the aggregates must have some desirable properties (Meininger & Nichols, 1990), such as toughness and abrasion resistance in which aggregates should withstand mixing and compaction and maintain original gradation. Aggregates should also be sound and durable and resistant to degradation under traffic load during the service life of the pavement. Plastic fines interfere with the asphalt coating and potentially hamper mixture durability in regards to moisture resistance;



therefore, clay content should be minimized. Aggregates with low clay content have demonstrated good performance (Brown et al., 2009).

### *2.7.2 Binder Properties*

Appropriate Superpave Performance Grade (PG) binder selection depends on climatic conditions at the project location. Proper PG binder grade ensures mixture durability in terms of resistance to rutting, fatigue cracking, and low-temperature thermal cracking. When RAP is used in sufficient quantity, a virgin binder is selected depending on project location and effective grade of the RAP binder. The resulting binder grade (blend of virgin and aged binders) should guard against breaches in durability. NCHRP Project 9-12 (McDaniel et al., 2000) and KDOT have established guidelines to compensate for the binder grade adding a stiff element (RAP) to the mixture. Bonaquist (2011) found that a RAP binder-to-virgin binder ratio of 0.25 to 0.35 is acceptable before the binder grade must be changed. KDOT follows requirements in AASHTO M 323 (2013).

### *2.7.3 Volumetric Properties*

An NCHRP project (Bonaquist, Christensen, & Stump, 2003) studied the effect of mixture volumetric properties on performance of HMA mixtures and concluded that in-place air void content and effective binder content of the mixture are two properties that most significantly affect pavement durability (Christensen, Hanz, Velasquez, Arshadi, & Bahia, 2014). In-place air void content is controlled by construction practices. However, durability decreases with increased air void content. Pavements sections with lower air voids are less permeable, and low permeability helps reduce the rate of binder aging. Effective virgin binder content contains voids filled with mineral aggregate (VMA) minus the air void content. VMA is the controlling factor for HMA mixture durability. Christensen et al. (2014) also suggested that an increase in VMA will increase effective virgin binder content. In Kansas, 1% increase in VMA should increase effective binder by about 0.4%. Several agencies have reduced the design number of gyrations in order to increase virgin binder content. However, specifying lowering gyrations will not increase binder content in a mix. Although only a passive relationship has been found between the

number of gyrations and effective binder content, alteration of gradation is more appropriate (Shakiba et al., 2013).

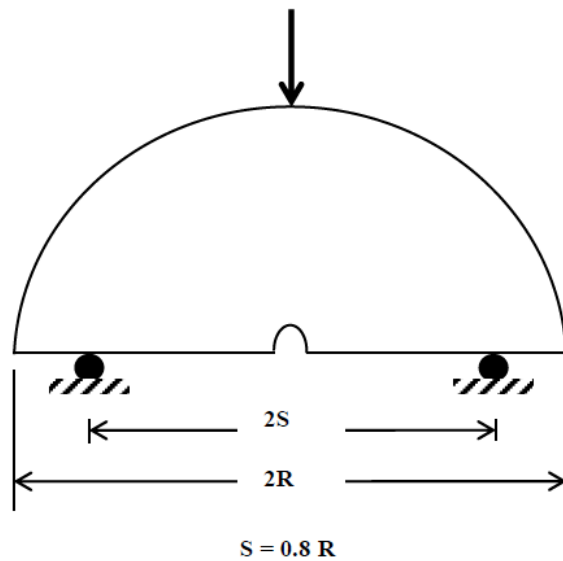
## **2.8 Past Research Evaluating Cracking Potential**

Cracking is one of the dominant distress types of flexible pavements. Two approaches are frequently followed for evaluating cracking resistance. The first approach investigates the number of load cycles sustained at a certain temperature before crack initiation under various experimental setups. The second approach integrates the degree of damage that occurs from the original undamaged sample under repetitive loading. This section describes four cracking propensity tests.

### *2.8.1 Semi-Circular Bending (SCB) Test*

The Semi-Circular Bending (SCB) test provides information regarding mixture stiffness and the energy release associated with crack formation. Although the test method is simple, several sequential steps are required in order to obtain usable parameters from the test output. Unfortunately, however, interpretation of output parameters is debatable. The test initially was designed to evaluate cracking potential and fracture mechanics of the mixture. Researchers currently tend to evaluate HMA mixture stiffness characteristics (Saadeh & Eljairi, 2011).

Chong and Kuruppu (1984) first used the SCB test to study fracture properties of rock materials. Loading configuration was strain-controlled, three-point flexural loading at a constant strain rate of 0.005 mm/s. A single-edged notch was made on the surface directly below the load line to provide a weak plane for controlled cracking. Test methods suggested by various studies to evaluate HMA mixture performance were nearly identical to that of Chong and Kuruppu. However, the strain rate and sample thickness varied in many studies. A typical setup for the SCB test at Kansas State University is shown in Figure 2.1(b).



(a)



(b)

**Figure 2.1: (a) Schematic SCB Test Setup, (b) SCB Test**

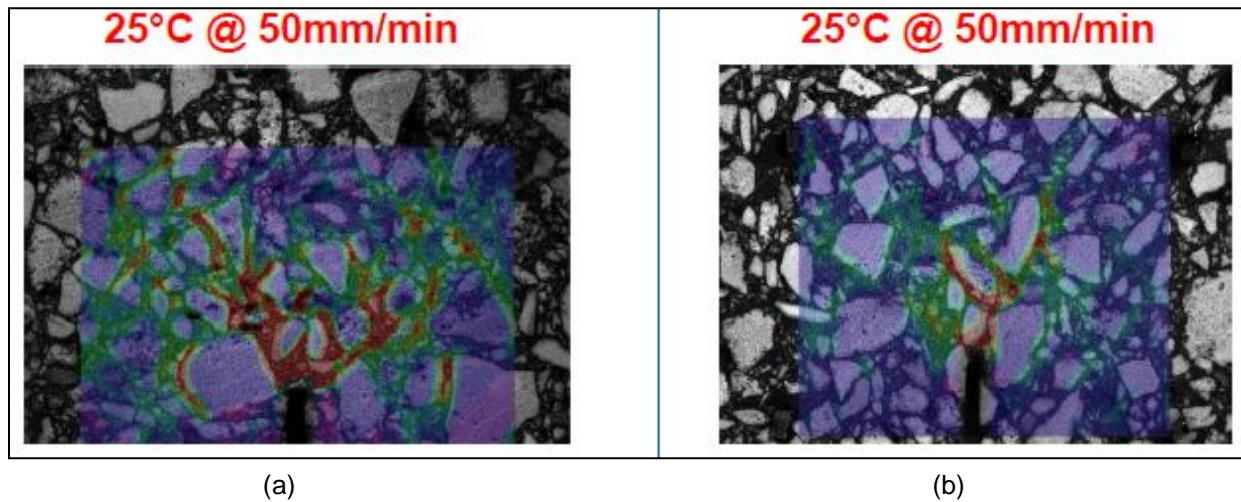
Disk-shaped samples with diameters of 100 or 150 mm are used in the SCB test. Sample thickness is typically kept within 35 to 50 mm at  $7\pm 1\%$  air voids for most test setups (Huang, Cao, & Zeng, 2009). Various loading rates in the stain-controlled test have been used, and strain rates vary from 0.01 to 0.001 mm/s in the literature (Walubita, Hoeffner, & Scullion, 2013).

Notch depth affects peak load of the sample in the SCB test (Cooper, 2014). Some researchers prefer to place the notch along the thickness plane to ensure crack initiation at the notch location. Results have shown that the deeper the notch depth, the lower the peak load. Cooper correlated this phenomenon with a decrease in ligament area that resisted failure when depth increased.

Quality standards of mixtures with high RAP content were studied using the SCB test at the University of Iowa (Lee et al., 2015). Samples were prepared with high RAP content (30, 35, and 40%), and tests were performed at two temperatures ( $-18$  and  $-30$  °C). Results showed that fracture energy (FE) decreased and stiffness increased as the test temperature decreased.

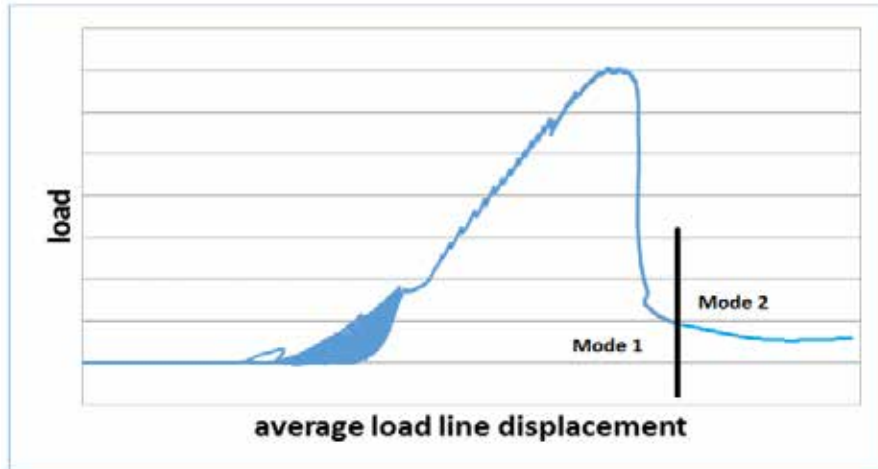
For this particular thesis,  $25$  °C was selected as the test temperature, similar to a study by Molenaar, Scarpas, Liu, and Erkens (2002). Al-Qadi, Ozer, Lambros, and Lippert (2015) also

utilized the SCB test for high-RAP HMA mixtures. Using the finite element method (FEM), they showed that the area that contributes to crack initiation reduces for HMA mixtures that contain increased amounts of RAP. Figure 2.2(a) shows the resisting zone against cracking with 0% RAP compared to 30% RAP with a narrower contributory area, as shown in Figure 2.2(b).



**Figure 2.2: FEM of SCB Stress Zone**  
Source: Al-Qadi et al. (2015)

Because stress distribution during SCB testing is not uniform and the HMA mixture is heterogeneous, simple geometry-based models cannot fully analyze SCB test output. Therefore, researchers have utilized a variety of methods in order to interpret test output. Molenaar et al. (2002) calculated tensile stress based on peak load and specimen diameter. Other output parameters, such as flexibility index, stiffness modulus, and Fracture Energy (FE), can be calculated using the load versus displacement curve obtained during SCB testing. Buss and Williams (2013) identified the load versus load line displacement curve (P-u) as typical output of the SCB test, shown in Figure 2.3. This research used test output (load versus displacement) data in order to construct Mode 1 shown in the Figure 2.3. Using curve setter in MATLAB, Mode 2 of the curve was extrapolated in order to evaluate rate of energy release.



**Figure 2.3: SCB Load versus Load Line Displacement (P-U) Curve**  
 Source: Buss and Williams (2013)

Podolsky, Buss, Williams, and Cochran (2014) studied the effects of additives in HMA at low temperatures and evaluated mixture performance at low temperatures (-24, -12, and 0 °C) using the SCB test. Results showed that when the temperature increased from -24 °C to -12 °C, the FE increased, but when the temperature increased from -12 °C to 0 °C, FE decreased. Between these two temperatures, the sample became softer and less capable of withstanding load.

Huang, Shu, and Zuo (2013) studied fatigue performance of HMA mixtures using the repetitive SCB (R-SCB) test. However, instead of constant strain, the sample was tested under constant sinusoidal load at a frequency of 5 Hz with minimum peak load as the input. The study revealed a relationship between resistance to fatigue cracking and asphalt binder properties. The SCB test was suggested as a valid test to evaluate fatigue performance of HMA mixtures with acceptable margin of error.

The SCB test has also been used to apply principles of fracture mechanics to asphalt pavement. Mathias, Adamson, and Andrews (1996) first considered asphalt mixtures as quasi-brittle materials.

Scullion, Walubita, Faruk, and Hoeffner (2012) studied repeated loading SCB test instead of monotonic loading and evaluated mixtures in terms of number of load cycles until failure.

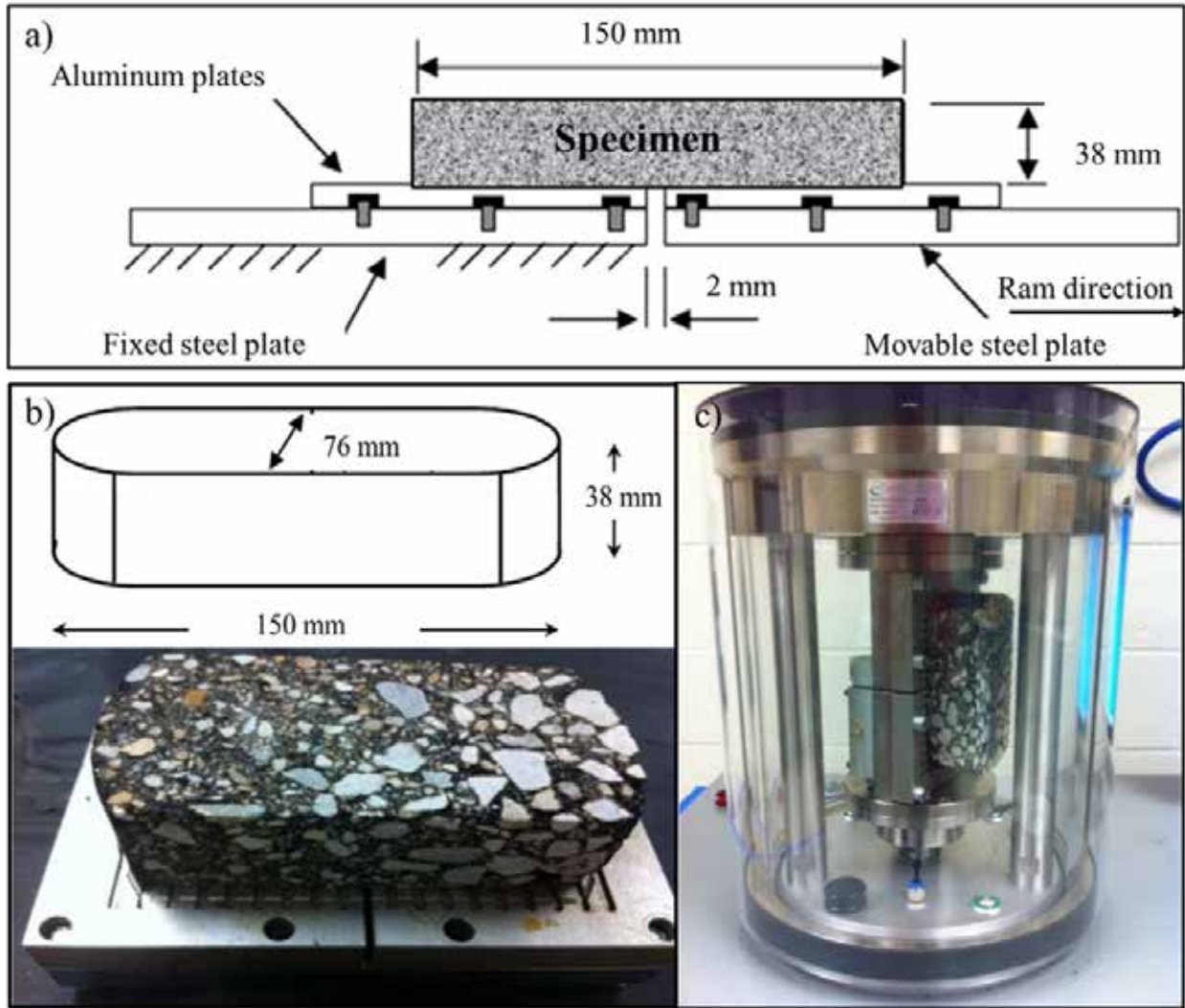
However, the released energy could not be calculated for this test, and results were reported in number of load cycles until failure. Results displayed variability beyond acceptable limits.

Al-Qadi, Abuawad, Dhasmana, Coenen, and Trepanier (2014) analyzed samples with high RAP and no RAP, proving that increased percentages of RAP decrease the area that contributes to crack resistance, thereby increasing susceptibility to cracking.

### *2.8.2 Texas Overlay Test*

The Texas Overlay Test (OT), developed and standardized by the Texas Transportation Institute, verifies cracking performance for HMA (TEX-248-F, 2014). The test determines the susceptibility of HMA mixtures to reflective cracking. In the OT test, repeated direct tension is applied to the sample glued to a flat plate with a joint in the middle. The loading mechanism simulates cracking in the underlying layer of HMA overlays (Zhou & Scullion, 2004). Schematics of a typical OT test setup and actual setup in the Asphalt Mixture Performance Tester (AMPT) at Kansas State University are illustrated in Figures 2.4(c) and 2.4(d), respectively.

The OT test is rapid and repeatable, allowing rapid identification of unsatisfactory samples. Because load is applied at a certain frequency in the test, crack initiation and crack propagation properties are able to be characterized (Zhou & Scullion, 2004). This study also discovered that the OT test evaluates reflective cracking potential of HMA mixtures, leading to the suggestion that the OT test is a promising fatigue cracking test for HMA. Theoretically, the higher the number of OT load cycles before the crack propagates through the thickness of the sample, the higher the resistance to cracking. For dense-graded Texas HMA mixtures, Zhou, Hu, and Scullion (2006) showed that HMA mixtures that sustained over 300 OT cycles demonstrated satisfactory crack resistance.



**Figure 2.4: Texas OT Setup at Kansas State University**

Li, Oh, Naik, Simate, and Walubita (2014) studied the OT test in order to characterize cracking resistance potential of approximately 30 mixture designs with various material properties and aggregate gradations. They concluded that the OT test could be used to evaluate cracking resistance of HMA mixtures. The Washington State DOT performed the OT test on five mixture designs with five replicates of each design. Variability in test results was very high, and results were inconclusive (DeVol, 2008).

The OT test was also studied with monotonic tensile load in order to evaluate the rate of FE release (Walubita et al., 2012). The load-displacement (P-u) curve was generated at a

constant loading rate and test temperature, and variability of test results was within the acceptable limit of 30%. Monotonic OT was considered to be a routine surrogate of repeated OT.

The OT test was compared to other cracking tests such as the monotonic and repeated indirect-tension test, the monotonic and repeated SCB test, the disk-shaped compaction tension test, and direct tension tests (Scullion et al., 2012). For repeated loading tests, variability was higher.

A technical report published by the Texas Transportation Institute (Scullion et al., 2012) listed sample drying method, glue quantity, number of replicates, air void content, sample age at the time of testing, and test temperature variation as key aspects of OT test repeatability and variability. As the tolerance level for variability of OT results increased up to 30% by the pavement community, the number of replicates for each type of sample decreased to 3 instead of 5. An ongoing project of the Texas Department of Transportation (TxDOT) focuses on modification of the current OT test in order to measure and characterize fatigue properties of HMA mixtures.

### *2.8.3 Dynamic Modulus Test*

Dynamic modulus has been introduced as a key input parameter in the mechanistic-empirical (ME) design method of HMA pavements. AASHTO PP 61-13 (2013) proposed a dynamic modulus test protocol that requires testing at 4, 20, and 40 °C and at loading frequencies from 0.01 to 10 Hz. In AASHTO TP 62-07 (2013), five temperatures from -10 to 54 °C and six frequencies from 0.1 to 25 Hz were required. For this study, tests were performed at 4, 21, and 37 °C with six frequencies 0.1, 0.5, 1, 5, 10, and 25 Hz in order from the lowest temperature to the highest temperature and from the highest to lowest frequencies, as is typical for these tests. Typical test setup of the dynamic modulus test is presented in Figure 2.5. The test was conducted in AMPT at Kansas State University.



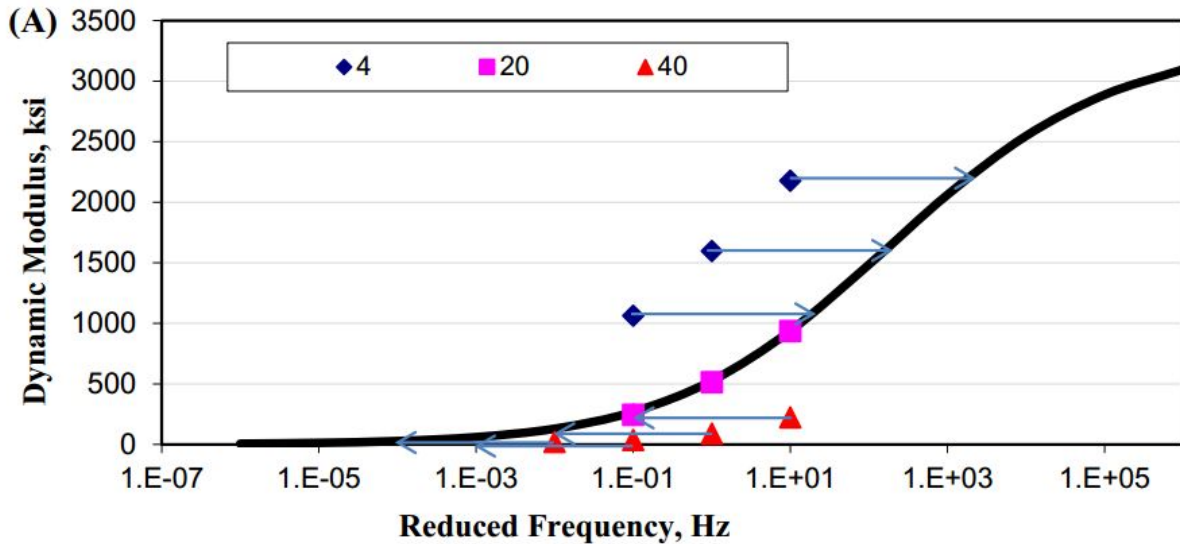


**Figure 2.5 Dynamic Modulus Test Setup**  
Source: Sabahfar (2012)

Dynamic modulus values represent stiffness of the HMA mixture. Ghabchi, Singh, and Zaman (2015) used dynamic modulus to compare warm-mix asphalt (WMA) and HMA stiffness in order to assess cracking performance. Most state DOTs that implement AASHTOWare Pavement ME design have also accepted dynamic modulus as the HMA characteristic with which design analysis can be conducted (West et al., 2014).

Dynamic modulus test outputs over a range of temperatures and frequencies are superposed in order to formulate dynamic modulus master curves. These master curves are used to predict dynamic modulus values at various points (i.e., temperature and frequency) of interest. Dynamic modulus master curves used in the AASHTOWare Pavement ME design are typically constructed using symmetric sigmoidal function. Studies have shown that lab-tested samples display a non-symmetric trend in dynamic modulus values, particularly at high temperatures when unconfined stiffness of HMA approaches zero. However, a nonsymmetric sigmoidal

function is more appropriate for explaining such phenomenon (Khosravifar, Schwartz, & Goulias, 2015). A schematic of the dynamic modulus master curve is presented in Figure 2.6.



**Figure 2.6: Dynamic Modulus Master Curve**

Source: Khosravifar et al. (2015)

Yao et al. (2013) showed that the dynamic modulus can be used to classify asphalt mixtures. This study tested samples at five temperatures (5, 20, 30, 40, and 60 °C) and at six frequencies (0.1, 0.5, 1, 5, 10, and 25 Hz). Master curves were formed using the symmetric sigmoid function, and coefficients of master curve (a, b, c, and d) were used to compare materials.

Cho, Park, and Hwang (2010) conducted rigorous laboratory testing in order to develop predictive equations for the dynamic modulus of HMA mixtures. Results showed that dynamic modulus of a mixture increased with increased loading frequency and decreased test temperature. They also concluded that air voids did not significantly influence dynamic modulus. Large nominal maximum aggregate size in the mixture resulted in higher modulus other mixtures.

Yu and Shen (2012) studied the effect of aggregate gradation and particle angularity on dynamic modulus of HMA mixtures. They discovered that the more the aggregates are packed,

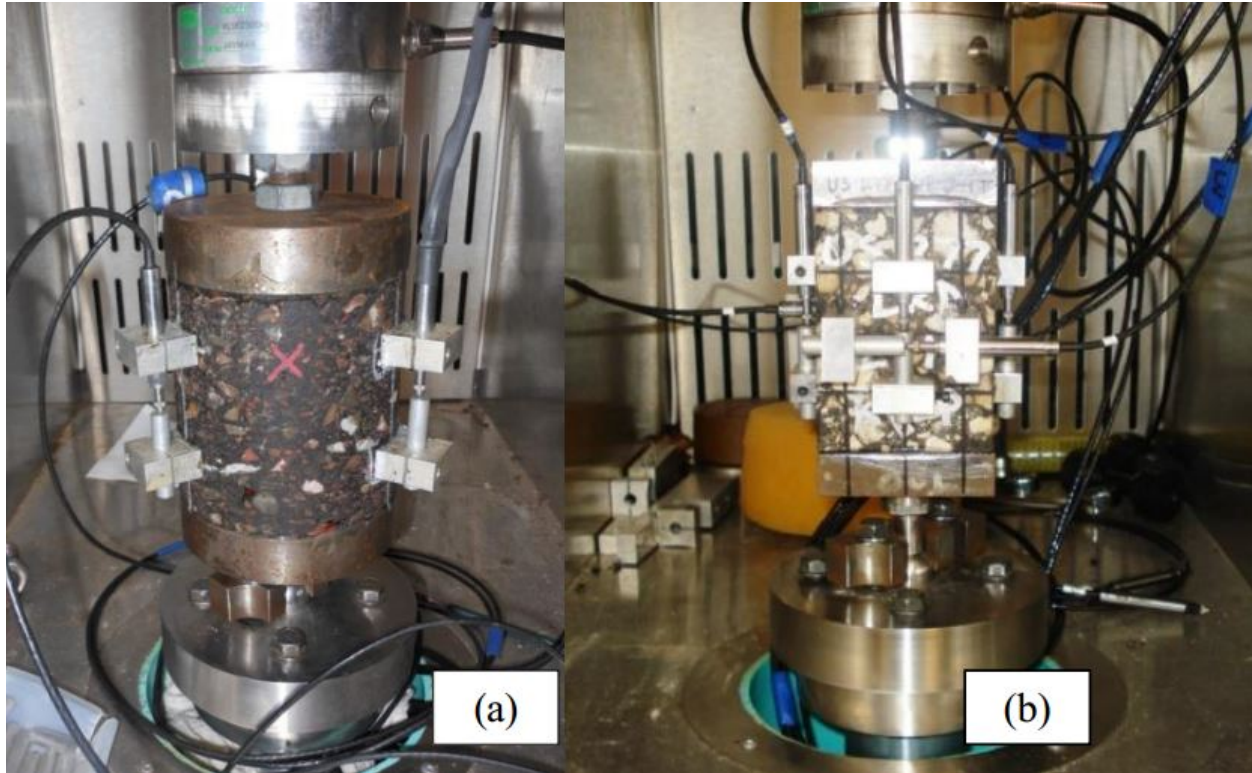
the greater the increase in dynamic modulus. Particle angularity was also found to have positive correlation.

In order to determine test temperature, researchers have selected temperature ranges suitable for the study area depending on geographic locations of projects. Sakhaeifar, Kim, and Kabir (2015) used a temperature range of -10 to 54 °C. Their study proposed a model to predict in-place dynamic modulus with a satisfactory level of accuracy.

#### *2.8.4 Evaluation of Fatigue Performance*

Fatigue performance of HMA pavements has been studied using various test setups and loading patterns. Two types of loading patterns are presented in the literature: repeated direct tension loading and repeated tension-compression loading. Tong, Lou, and Lytton (2015) used repeated direct tension loading to study fatigue crack growth in HMA samples in a dynamic mechanical analyzer. The fatigue law was verified using test results. Kim et al. (2009) used the tension-compression loading pattern to study fatigue performance of HMA mixtures. In order to study fatigue performance of HMA mixtures, Ren, Geng, An, and Wang (2015) designed a repeated uniaxial penetrating test based on composite modification of the Flow Number test and Uniaxial Penetrating test that simulates repeated vehicle load. Daniel and Kim (2002) studied fatigue characteristics of HMA samples at various strain amplitudes and frequencies at different test temperatures.

In HMA mixtures, resistance against fatigue cracking is primarily controlled by the asphalt binder; aggregate contribution to fatigue cracking resistance is low (Tan, Shan, Kim, & Underwood, 2012). Their study concluded that laboratory tests typically underestimate fatigue resistance of mixtures by not considering the healing effect of binder under real-time loading during the rest period. Extended rest periods between loading cycles could simulate real-time loading, but no index has been developed in order to quantify the amount of healing by the binder. Therefore, this issue has been addressed by the Viscoelastic Continuum Damage (VECD) theory. In order to model damage, the work potential theory takes healing effect into account. Various sample shapes and sizes for fatigue evaluation are also under experimentation. Figure 2.7 shows a cylindrical sample and a prismatic sample for field sampling (Martin et al., 2013).



**Figure 2.7: Push-pull Fatigue Test Samples: (a) Cylindrical, (b) Prismatic Field Sample**  
Source: Martin et al. (2013)

Several models have been developed in order to evaluate fatigue characteristics of HMA pavements. Modeling approaches can be broadly classified as mechanistic material models and micromechanical models. Mechanistic material models can be further classified as empirical and derived. Monismith, Secor, and Blackmer (1961) and Pell and Cooper (1975) proposed an empirical model called fatigue law. Three categories of theory-derived models are presented in the literature: dissipated creep stain energy model (Roque et al., 2010), VECD theory (Roque et al., 2010; Underwood, Baek, & Kim 2012), and viscodamage model. Numerical models have been primarily studied for micromechanical models in order to evaluate fatigue characteristics of HMA mixtures. The lattice model (Guddati, Feng, & Kim, 2002), the discrete element-based method, and two-way coupled approaches are examples of micromechanical modeling (Underwood & Kim, 2014).

The VECD test (VECD FEP++) evaluates pavement performance using FEM-based analysis and the viscoelastic continuum theory. The test was developed by North Carolina State University under the NCHRP 1-42A effort (Roque et al., 2010). The combination of the VECD test and FEM allows fracture mechanics of the compacted sample and continuum damage mechanics of the mixture to be simultaneously captured while dealing with the microcracking phenomenon (Underwood & Kim, 2014).

The VECD model was developed based on the elastic-viscoelastic correspondence principle, the temperature-time superposition principle (for dynamic modulus properties), and the work potential theory (Xie, Shen, Earnest, Li, & Jackson, 2015). The elastic-viscoelastic principle was used to explain viscoelastic behavior of HMA mixtures in the model in order to convert strain to equivalent pseudo strain. The temperature-time superposition principle was used to merge the effect of these two strains at different temperatures to certain reference temperature, and then the work potential theory (Schapery, 1990) was used to model damage growth. Equation 2.1 was derived in order to calculate pseudo strain at time step  $t$ .

$$\varepsilon^R = \frac{1}{E^R} \int_0^t E(t - \tau) \frac{d\varepsilon}{d\tau} d\tau \quad \text{Equation 2.1}$$

Where:

$\varepsilon^R$  is the pseudo strain, and

$\varepsilon$  is the actual strain.

$E^R$  is the reference modulus, and

$E(t)$  is the relaxation modulus at time step  $t$ .

In the next step, damage growth is modeled. Damage parameter  $S$  is defined as the structural change in stiffness as a result of repetitive loading in terms of pseudo stiffness,  $C$  (Equation 2.2; Kim et al., 2009).

$$C = \frac{\sigma}{\varepsilon^{R*I}} \quad \text{Equation 2.2}$$

The work potential theory has three fundamental functions:

- a. Pseudo strain energy density function

$$W^R = f(\varepsilon^R, S) = \frac{1}{2} \sigma \varepsilon^R = \frac{1}{2} (\varepsilon^R)^2 C \quad \text{Equation 2.3}$$

- b. Stress-pseudo strain relationship

$$\sigma = \frac{\partial W^R}{\partial \varepsilon^R} = C(S) \varepsilon^R \quad \text{Equation 2.4}$$

- c. Damage evolution law

$$\frac{dS}{dt} = \left( -\frac{\partial W^R}{\partial \varepsilon^R} \right)^\alpha \quad \text{Equation 2.5}$$

Where:

$W^R$  is pseudo strain energy density, and  
 $\alpha$  is the damage evolution rate.

Cumulative damage accumulated due to loading for each time step can be evaluated using Equation 2.6, resulting in development of damage characteristics curves (C-S) for each mixture.

$$S_{N+1} = S_N + \left[ -\frac{DMR}{2} (C_N - C_{N-1}) (\varepsilon^R)^2 \right]^{\frac{\alpha}{\alpha+1}} \Delta \xi^{\frac{1}{\alpha+1}} (K_1)^{\frac{1}{\alpha+1}} \quad \text{Equation 2.6}$$

$$C = 1 - C_{11} S^{C_{12}} \quad \text{Equation 2.7}$$

Where:

$DMR$  is dynamic modulus ratio,  
 $\Delta \xi$  is the reduced time interval, and  
 $K_1$  is constant accounted for cyclic data.

$$DMR = \frac{|E^*|_{Fingerprint}}{|E^*|_{LVE}} \quad \text{Equation 2.8}$$

Depending on predicted damage characteristics, fatigue life prediction can be done for mixtures.

In this study, a simplified VECD test was utilized in order to evaluate fatigue performance of HMA mixture with RAP. Because prediction of effective stress versus strain equations and growth of macrostrain are typically the bases for evaluating viscoelastic properties of HMA mixtures (Lee, Mun, & Kim, 2011), dynamic modulus and damage characteristics were used for the S-VECD model in this study. The S-VECD model considered stiffness of a damaged body as a body with lower stiffness; the amount of associated damage was related to the effective area. Damage characteristic curves were direct indications of fatigue resistance of a material at each time step.

Increase in RAP content has been reported to lead to low fatigue resistance, but a decrease in fatigue life has not been found to decrease significantly. Sheikhmotevali and Ameri (2014) concluded that HMA mixtures with up to 50% RAP can perform well. Evaluation of crack initiation in the fatigue test is typically considered to be the stopping point of the test. However, researchers have had difficulty identifying microcracks as soon as they appear. Therefore, Sheikhmotevali and Ameri conducted a study to detect cracks using laser technology in order to provide an automated stopping point.

## **2.9 Summary**

Many agencies have reported that the addition of recycled materials decreases levels of cracking performance and durability. Ongoing studies seek to identify acceptable levels of decrease in performance. Rutting performance has been proven to improve with the addition of recycled materials. Researchers are attempting to suggest specifications based on mixture performance and mixture recipe. Various tests have been used to characterize cracking resistance of HMA mixtures, and various fatigue models have been proposed in order to define cracking resistance of these mixtures.

## Chapter 3: Methodology

### 3.1 Experimental Design

Three RAP sources were used for this project in order to study the effect of RAP source and increasing RAP content on HMA mixtures. The sources were Shilling Construction Company, Konza Construction Company, and a KDOT project on US-73 near Kansas City. For each RAP source, three Superpave mixture designs with increasing RAP content (20, 30, and 40%) were prepared. Sabahfar (2012) developed the mixture designs for the first two RAP sources (Shilling and Konza RAP). Mix design for the project on US-73 was developed in this study. All mixtures had 12.5-mm nominal maximum aggregate size (NMAS). These mixtures when containing RAP are known as SR-12.5 in KDOT mix design specifications. Virgin aggregates used in all mix designs were kept same, and cracking potential tests were performed on all mixtures. Sabahfar previously used the dynamic modulus test to study fatigue cracking and permanent deformation of mixes containing Shilling and Konza RAP. Aziz (2013) evaluated the cracking potential of those mixtures using the SCB test and OT test. In this study, the S-VECD test was used to test mixtures with all three RAP sources in order to evaluate fatigue cracking potential. Mixtures with RAP from US-73 were also tested for cracking potential using the SCB, OT, and dynamic modulus tests.

### 3.2 Aggregate and RAP Gradation

Mixture aggregate gradations for this study were done by blending five virgin aggregates and the RAP. The following virgin aggregates were obtained from Shilling Construction Company: CS-1 (crushed limestone with coarse gradation), CS-1A (crushed limestone with fine gradation), MSD-1 (manufactured sand), CG-5 (crushed gravel), and SSG (natural/river sand). Table 3.1 lists gradations of each aggregate and RAP sources. Figure 3.1 shows gradations on semi-log scale and Figure 3.2 shows blending of nine mixtures on 0.45-power chart. Although all RAP sources demonstrated substantial amounts of fines, some mixtures were deficient in the dust-to-binder ratio. Therefore, in order to meet specifications, 1% limestone dust was added to the aggregate blend. Table 3.2 summarizes aggregate proportions for each SR-12.5A mixture.



**Table 3.1: Aggregates and RAP Gradations**

Material	CS-1	CS-1A	MSD-1	CG-5	SSG	Shilling RAP (1)	Konza RAP (2)	US-73 RAP (3)
Sieve Size	% Passing							
¾	100	100	100	100	100	100	100	100
½	59	100	100	100	100	98	96	99
⅜	20	100	100	100	100	94	92	96
#4	2	29	99	96	95	80	78	80
#8	2	6	63	77	77	64	64	57
#16	2	2	36	49	53	47	48	39
#30	2	1	22	30	31	33	35	28
#50	2	1	13	18	12	20	21	19
#100	2	1	9	11	4	13	15	15
#200	2	1	8	9	4	10	12	12

**Table 3.2: Proportion of Individual Aggregates in the Mix Design**

Aggregate Type	Aggregate Proportion (%)								
	Shilling RAP (1)			Konza RAP (2)			US-73 (3)		
RAP	20	30	40	20	30	40	20	30	40
CS-1	20	16	12	20	16	12	15	14	12
CS-1A	12	15	13	12	15	13	17	15	13
MSD-1	12	13	13	12	13	13	18	15	13
CG-5	16	12	12	16	12	12	12	10	9
SSG	20	14	10	20	14	10	18	16	13

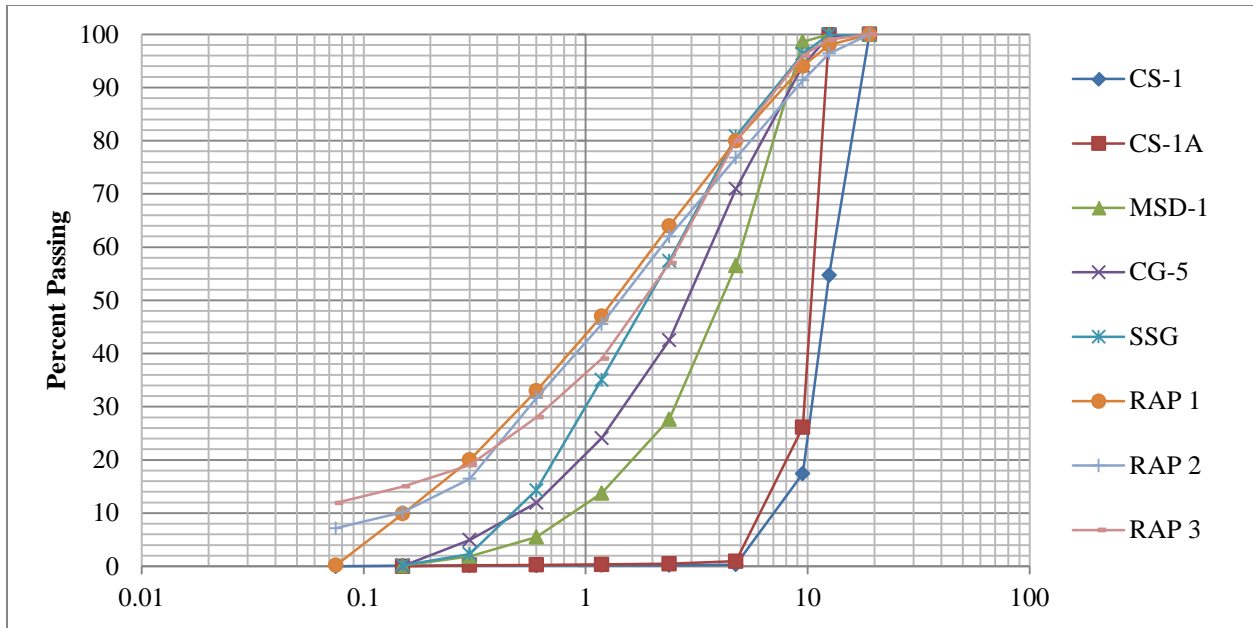
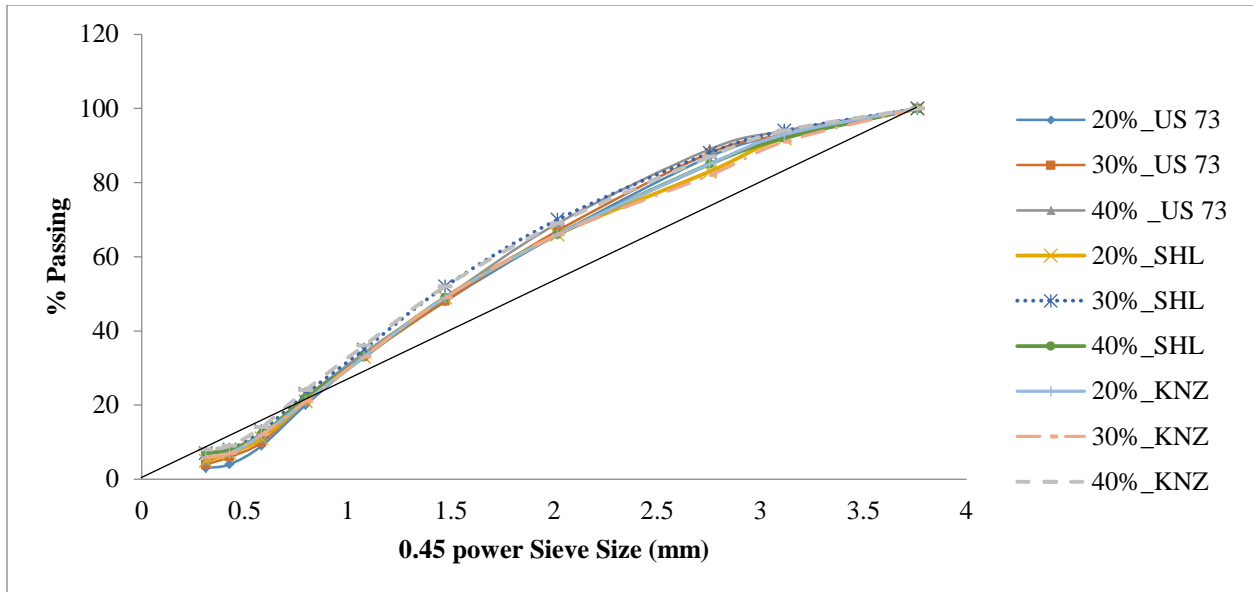


Figure 3.1: Aggregate Gradations

Table 3.3: Aggregate Blending of Mixtures

Sieve Size (mm)	20% RAP			30% RAP			40% RAP			KDOT Requirements
	Shilling RAP	Konza RAP	US-73 RAP	Shilling RAP	Konza RAP	US-73 RAP	Shilling RAP	Konza RAP	US-73 RAP	
19	0	0	0	0	0	0	0	0	0	0
12.5	8	7	7	6	9	7	8	6	6	0-10
9.5	17	15	13	12	18	12	15	13	11	10 Min
4.75	34	34	34	30	34	33	34	31	31	
2.36	51	51	52	48	51	52	51	48	51	42-61
1.18	67	67	67	65	67	67	67	64	66	
0.6	79	79	80	77	79	79	78	76	78	
0.3	89	88	91	87	88	90	88	86	89	
0.15	93	93	96	92	93	94	92	91	93	
0.075	95	94	97	93	94	96	93	92	95	90-98



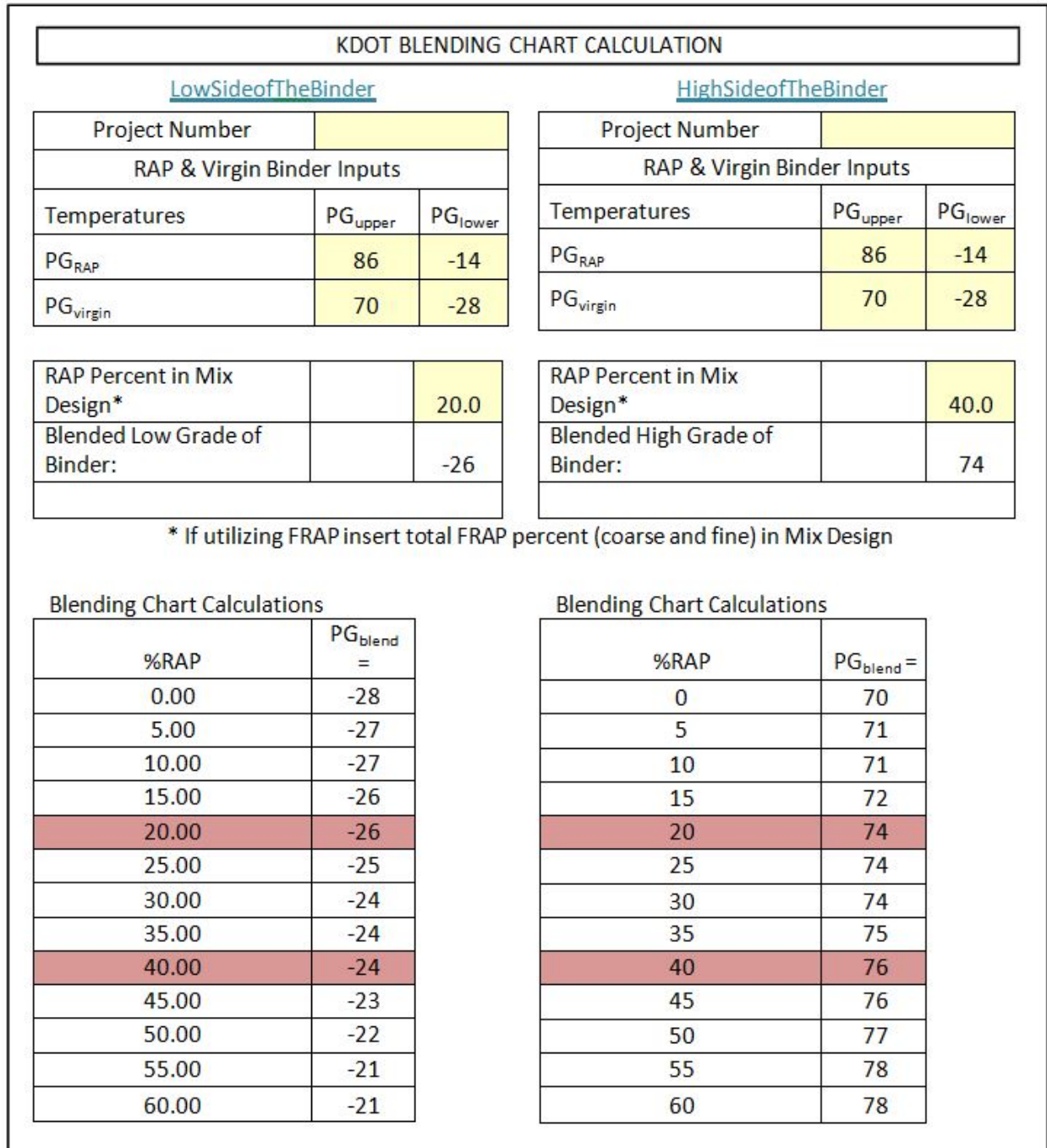
**Figure 3.2: 0.45-Power Sieve Size Distribution Curve**

### 3.3 RAP and Virgin Binder Grade

Performance Grade (PG) of the virgin binder was selected in accordance with AASHTO M 323 (2013) guidelines. Binder in the RAP typically becomes stiff due to years of weathering. Therefore, if RAP is used in a mixture, the virgin binder grade must account for the aged binder contribution. According to current guidelines (AASHTO M 323), no adjustment in virgin binder grade is required for mixtures with less than 15% RAP. Because this study utilized mixtures with higher percentages of RAP, softer binder had to be used. Guidelines for binder modification are summarized in Table 3.4.

**Table 3.4 Binder Grade Modification Recommendations for RAP Additions**

Recommended virgin asphalt binder grade	%RAP
No change in binder selection	<15
Select virgin binder one grade softer than normal	15-25
Follow recommendations from blending chart	>25



**Figure 3.3: PG Virgin Binder Selection Using KDOT Blending Chart**

### 3.4 Laboratory Mix Design

For the first two sources of RAP, virgin binder PG was selected as 70-28 by Sabahfar (2012). For the third RAP source, US-73, PG 70-28 was also selected. KDOT determined

effective PG of the RAP binder to be 86-14. Based on the KDOT blending chart, the low sides of PG limits were -26 and -24, and upper limits were 74 and 76 for 20% and 40% RAP content, respectively, as shown in Figure 3.3. Therefore, a high side of 70 and a low side of -28 were selected for mixtures containing US-73 RAP.

As mentioned, in this study nine SR-12.5A mixture designs were established for three RAP sources (Shilling, Konza, and US-73) and three RAP contents (20%, 30%, and 40%). Sabahfar (2012) performed the first six mixture designs, and the other three designs were developed in this study. Mixture designations used throughout this document are shown in Table 3.5.

**Table 3.5: Identification of Mixture Designs**

Mixture Identification	RAP Source	RAP Content (%)	Comment
1	Shilling Co.	20	Sabahfar (2012)
2		30	
3		40	
4	Konza Construction	20	
5		30	
6		40	
7	US-73	20	This study
8		30	
9		40	

All mixtures met the volumetric property and other requirements for SR-12.5A mixture specifications in Kansas. The design asphalt content was selected based on volumetric property

criteria at 4% air voids at the design level number of gyrations ( $N_{des}$ ) of 75. Steps followed for the mix design process are described in the following sections.

### 3.4.1 Mixing

Before mixing the aggregates, RAP, and virgin binder, all aggregates were dried on individual trays overnight at  $110 \pm 5$  °C in order to remove excess moisture. Aggregates, RAP, and virgin binder were heated separately before mixing. The aggregates and virgin binder were heated in order to attain recommended mixing temperatures of 153 to 160 °C. RAP was heated to 48 °C in order to avoid further aging of the accompanying binder. Mixing was done using a lab-scale paddle mixer, as shown in Figure 3.4.



**Figure 3.4: Paddle Mixer Used in the Study**

### 3.4.2 Sample Preparation for Volumetric Analysis

#### 3.4.2.1 Samples for $G_{mm}$

Theoretical maximum specific gravity ( $G_{mm}$ ) of mixtures was determined using Kansas Test Method KT-39 (2014). Loose mixture was separated into small pieces in such a way that no lump was greater than 5 to 6 mm. They were then aged in a temperature-controlled oven for 2 hours at 130 to 138 °C (compaction temperature range). Mixtures were stirred after 1 hour for homogenous conditioning. The required sample size for this test for the 12.5-mm NMA mixture was 1,500 gm.

### 3.4.2.2 Samples for $G_{mb}$

The Kansas Test Method KT-15 (2014), Procedure III, was used to prepare compacted samples to measure the bulk specific gravity of the compacted mixture. Mixtures were heated at recommended compaction temperatures (130 to 138 °C) for 2 hours. The mold and the top and bottom plates of the SGC were also heated at the same temperature for approximately 45 minutes. Samples were compacted using SGC at  $4\pm 0.2\%$  air voids at  $N_{des}$  of 75 to 115 mm height.

### 3.4.3 Test Procedures

#### 3.4.3.1 $G_{mm}$ Test Procedure

Once the loose mixture cooled after mixing, it was broken into small pieces, weighed, and poured into a calibrated flask in order to determine  $G_{mm}$ . Weights of the flask with and without the sample were also taken in the air. The flask was half-filled with water at  $25\pm 0.5$  °C, and a vacuum of  $3.6\pm 0.4$  kPa was applied in order to remove excess air trapped inside the loose mixture. Full vacuum was applied in 30 seconds and maintained for  $14\pm 0.5$  min. A vibrator plate under the flask agitated the sample to remove air, and then the flask was suspended in a water bath at  $25\pm 0.5$  °C for  $10\pm 1$  minutes. The weight of the flask with sample under water was measured. Figure 3.5 shows the steps for measuring  $G_{mm}$ .

Equation 3.1 was used to calculate the theoretical maximum specific gravity ( $G_{mm}$ ).

$$G_{mm} = \frac{A}{A-C}$$

**Equation 3.1**

Where:

A is mass of dry sample in air (gm), and

C is mass of sample in water at 25°C (gm).



(a)

(b)



(c)

**Figure 3.5: (a) Loose HMA Mixture in Calibrated Conical Flask, (b) Vacuum Application on Loose Mixture to Remove Air, and (c) Submerged Mass Measurement of Loose Mix**

Source: Aziz (2013)

#### 3.4.3.2 $G_{mb}$ Test Procedure

In this test, the dry mass of the sample in air was measured first, and then the sample was submerged in water for  $4 \pm 1$  minutes in order to measure the mass in water. Water temperature was maintained at  $25 \pm 0.5$  °C. The sample was removed from the water, rolled in a damp towel,



and the mass of the sample at the saturated surface dry (SSD) condition was determined.  $G_{mb}$  was calculated using Equation 3.2.

$$G_{mb} = \frac{A}{(B-C)} \quad \text{Equation 3.2}$$

Where:

A is mass of dry sample in air (gm),

B is mass of SSD sample (gm), and

C is mass of saturated sample in water at 25 °C.

### 3.4.4 Volumetric Properties

Using  $G_{mm}$  and  $G_{mb}$  values and specimen heights at the initial, final, and design number of gyrations, volumetric properties were calculated for the mixtures, as presented in Table 3.6. As shown in the table, the mixtures met all volumetric and other KDOT requirements for 12.5-mm NMAS Superpave mixture.

**Table 3.6: Volumetric Properties of Mixtures**

Mix Design	% Air Voids at Ndes			%VMA			%VFA			Dust-to-binder Ratio			%Gmm at Nini			%Gmm at Ndes		
	R1	R2	R3	R1	R2	R3	R1	R2	R3	R1	R2	R3	R1	R2	R3	R1	R2	R3
<b>20% RAP</b>	3.9	4.0	3.9	14.1	14.0	16.2	71.6	71.5	68.4	0.60	0.61	0.65	88.5	88.5	83.5	96.0	96.0	95.0
<b>30% RAP</b>	4.0	3.9	3.9	14.0	14.1	15.9	71.3	71.3	67.3	0.60	0.62	0.65	88.0	88.0	84.8	96.0	96.0	94.0
<b>40% RAP</b>	4.0	4.0	4.1	14.2	14.1	14.8	71.9	71.9	71.5	0.70	0.61	0.68	87.8	87.8	87.6	96.0	96.0	95.0
<b>KDOT Superpave volumetric mix design specifications</b>				<b>Minimum 14</b>			<b>65-78</b>			<b>0.6-1.2</b>			<b>Maximum 90.5</b>			<b>Maximum 98.0</b>		

Note: R1 = Shilling RAP; R2 = Konza RAP; R3 = US-73 RAP

### 3.4.5 Design Asphalt Binder Content

When the trial mixture satisfied the volumetric properties, the corresponding binder content was taken as the design binder content. However, several trials were necessary before this design binder content was obtained. Table 3.7 lists total binder content, virgin binder content, and RAP binder contents for each mixture.

**Table 3.7: Binder Proportions (Total Binder, Virgin Binder, and RAP Binder Content)**

Mix Design	Shilling RAP			Konza RAP			US-73 RAP		
	Total Asphalt Content (%)	Virgin Asphalt added (%)	Asphalt Content from RAP (%)	Total Asphalt Content (%)	Virgin Asphalt added (%)	Asphalt Content from RAP (%)	Total Asphalt Content (%)	Virgin Asphalt added (%)	Asphalt Content from RAP (%)
20% RAP	4.7	3.6	1.1	4.3	3.5	0.8	4.8	3.5	1.3
30% RAP	4.8	3.1	1.7	4.4	3.2	1.2	4.6	3.2	1.4
40% RAP	4.3	2.1	2.2	4.1	2.5	1.6	4.5	2.7	1.8

### 3.5 Moisture Susceptibility Test

The moisture susceptibility test was performed according to the Kansas Test Method KT-56 (2014), *Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage*, which measures changes in tensile strength resulting from effects of saturation, freezing, and accelerated water conditioning of SGC-compacted mixtures. KT-56 is a slightly modified version of AASHTO T 283 (2007) and is also known as the modified Lottman test in Kansas. Samples were compacted to  $7 \pm 0.5\%$  air voids using SGC. Diameters of the compacted samples were 150 mm and heights of the samples were  $95 \pm 5$  mm; six specimens were compacted and tested for each mixture design. After compaction and air void determination, three specimens of comparable air voids were selected for testing in dry conditions, and three specimens were selected for testing after being exposed to one full freeze-thaw cycle and hot-water soak. The latter set of samples was also subjected to partial vacuum saturation between 70 and 80% of the volume of air before freezing. This was done by immersing the samples in a vacuum container filled with water pressurized at 33 to 87 kPa pressure in order to remove air and partially saturate

the sample. After correct saturation level was attained, three samples were wrapped in plastic film and sealed in an airtight bag with 10 mL free water. Samples were then placed in a freezer for at least 16 hours at  $18\pm 3$  °C temperature. After completion of the freezing cycle, the samples were thawed in a hot water bath for  $24\pm 1$  hours at  $60\pm 1$  °C. The samples were cooled to room temperature in a water bath of  $25\pm 1$  °C temperature for 2 hours and then tested for indirect tensile strength. Figure 3.6 shows the test stages.



**Figure 3.6: Moisture Susceptibility Test Steps**  
Source: Sabahfar (2012)

The first set of samples was stored sealed in plastic wrap in dry conditions. Before testing, these samples were also kept in the water bath for 2 hours at 25±1 °C. Both samples were tested at a loading rate of 50mm/minute until failure. Peak loads were recorded, and tensile strength of each sample was calculated using Equation 3.3 (Hossain, Maag, & Fager, 2010).

$$S = \frac{2P}{\pi tD} \quad \text{Equation 3.3}$$

Where:

$S$  is tensile strength (kPa),

$P$  is maximum load (N),

$t$  is specimen thickness (mm), and

$D$  is specimen diameter (mm).

Tensile strength ratio (TSR) was calculated according to Equation 3.4.

$$\text{TSR} = \frac{S_2}{S_1} \times 100 (\%) \quad \text{Equation 3.4}$$

Where:

$S_1$  is average tensile strength of unconditioned samples, and

$S_2$  is average tensile strength of conditioned samples.

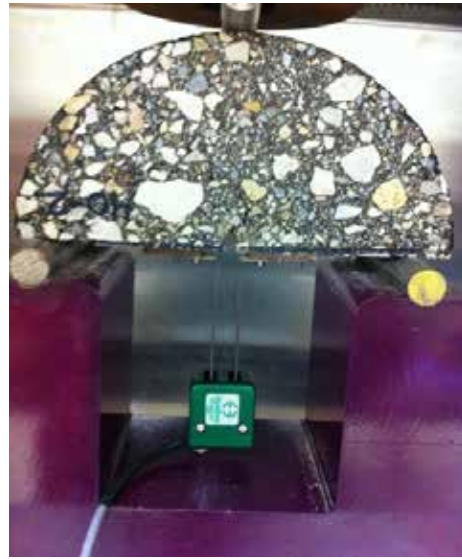
### 3.6 Semi-Circular Bending Test

The SCB test was performed in two modes for this study: static and repetitive. The static mode evaluated cracking resistance of HMA mixtures under monotonic load. The R-SCB test was performed in order to investigate fatigue cracking behavior of mixtures using strength parameters of mixtures derived from static SCB (S-SCB) tests. Both tests were performed in the Universal Testing Machine (UTM-25).

#### 3.6.1 Static Semi-Circular Bending Test

The S-SCB test uses a three-point bending load. The test was performed under a monotonic load at a static load rate of 30mm/minute at 25 °C. Samples were compacted using SGC in order to produce 150-mm diameter and 100±5-mm-high cylindrical samples with 7±1% air voids. Test samples were sawn into two 50-mm-thick disks and then cut into halves in order

to produce semicircular samples of 150-mm length of the bottom surface and  $50\pm 5$ -mm width. A triangular notch of 2.5- to 5-mm depth was fabricated at the middle of the base of the sample. Three replicates were tested for each mixture. Samples were conditioned at the test temperature for 2 hours in a temperature-controlled chamber. The test setup is shown in Figure 3.7.



**Figure 3.7: SCB Test Setup**

The three-point bending setup was custom-made for the test. Roller supports were cleaned with lubricant oil before each test in order to avoid stress concentration near the support. The load line of the sample was aligned with the top of the notch for controlled crack initiation, and loading rate of 30 mm/minute was determined based on previous researchers (Walubita et al., 2012). Once a crack appeared and a rapid change in the stress-strain curve occurred, the test was stopped manually.

After completion of the test, peak load, and load-load line deflection (P-u) curve were obtained. From the P-u curve, Mode I of the fracture process (Figure 3.8) was obtained. In order to identify Mode II, Matrix Laboratory (MATLAB) was used to simulate the trend in the curve. The area under the entire curve was obtained by MATLAB and taken as the fracture energy (FE; Equation 3.5). Fracture stiffness parameter was also calculated using Equation 3.6.

$$G_f = \frac{W_f}{A_{lig}} \quad \text{Equation 3.5}$$

Where:

$G_f$  is FE (J/m<sup>2</sup>), and

$A_{lig}$  is area under the load-line (m<sup>2</sup>).

$$S = c \left(\frac{L}{D}\right) \frac{P}{td} \quad \text{Equation 3.6}$$

Where:

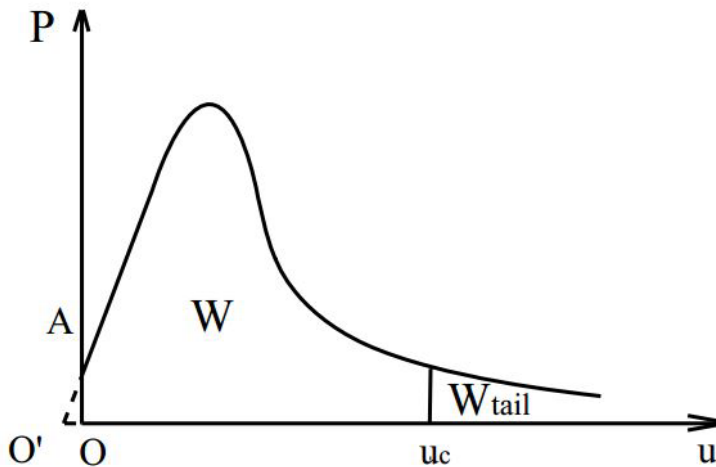
$S$  is stiffness modulus,

$c$  is dimensionless function of relative spacing between supports,

$L/D$  is length-to-diameter ratio,

$P$  is load (KN), and

$t$  is thickness of the sample (mm).



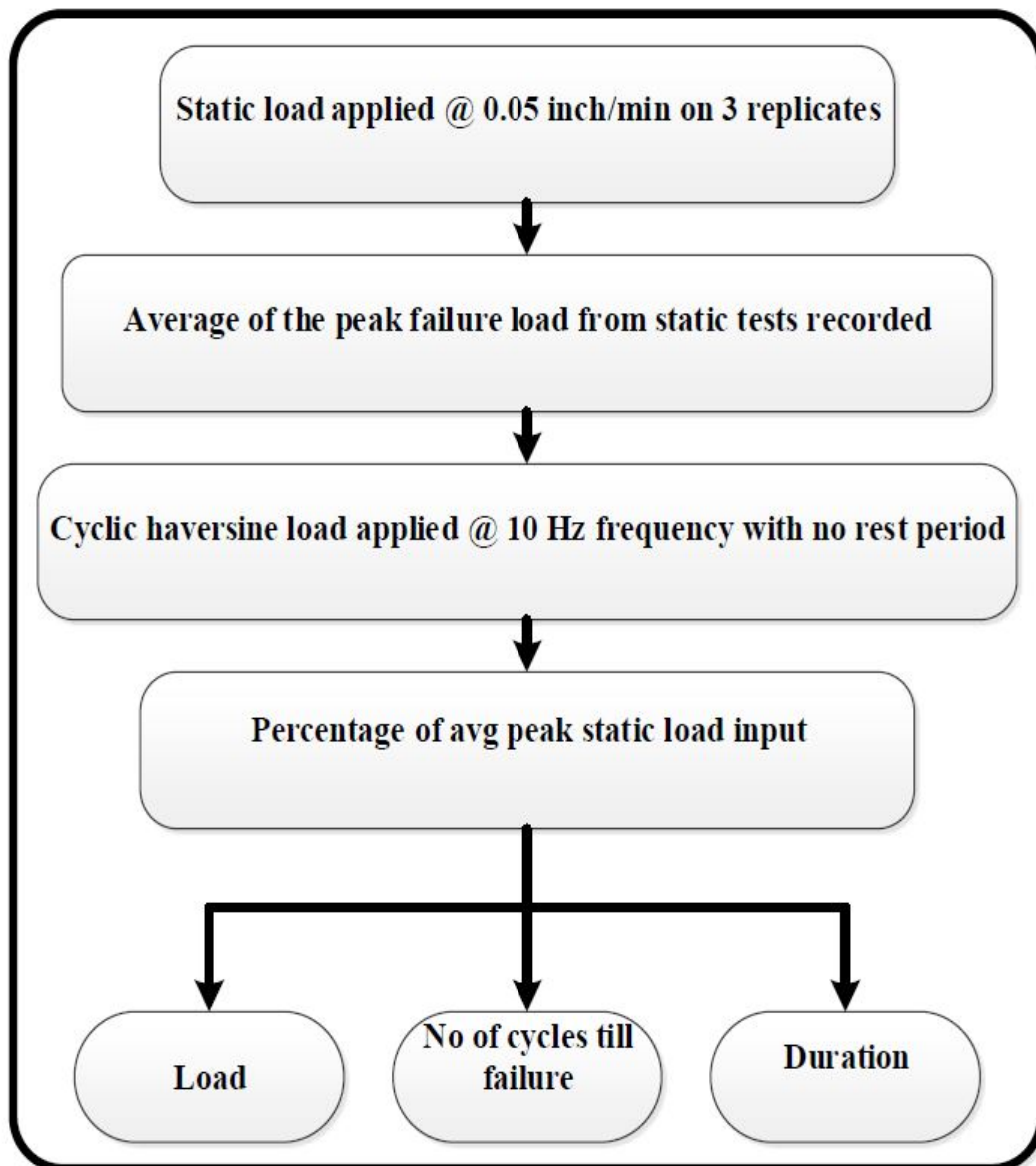
**Figure 3.8: Typical Load-Displacement Curve for SCB Test**

Source: Cooper (2014)

### 3.6.2 Repetitive Semicircular Bending Test

The R-SCB test, performed in order to evaluate fatigue cracking potential of the mixtures, used the average peak load obtained from the S-SCB test. For each mix, four percentages of peak loads (30, 40, 50, and 60%) were used as input values. For each load

percentage, three samples were tested under repetitive loading. R-SCB test procedures were similar to the S-SCB test in terms of sample preparation and test temperature, but the test mode was switched from the strain-controlled mode to the stress-controlled mode. A loading frequency of 10 Hz was used with no rest period, as suggested by Walubita et al. (2010). Earlier mentioned study also suggested that 50% of the peak load should be used to evaluate fatigue performance of HMA mixtures. Therefore, test results from the 50% peak load were used for further analysis.



**Figure 3.9: Flow Chart of R-SCB Test**  
Source: Aziz (2013)

The R-SCB test was stopped manually after full propagation of a crack along the load-line, and the number of cycles before failure was recorded. A flow chart of the test is shown in Figure 3.9. The crack mouth opening was also monitored during this test using an Epsilon clip-on gauge attached on the bottom surface of the specimen (as shown in Figure 3.7). This device measured the gradual increase in crack dimensions in order to provide a guideline for test standardization.

### 3.7 Texas Overlay Test

The Texas Overlay Test is performed on the HMA mixtures to evaluate the reflective cracking resistance. The test was developed by the Texas Department of Transportation (TxDOT) and is covered by the TxDOT Standard Test Procedure TEX-248-F (2014).

Test samples for the OT test required fabrication of cylindrical compacted samples with diameters of 150 mm and heights of  $115 \pm 5$  mm at  $7 \pm 1\%$  air voids using an SGC. OT test samples were trimmed, as shown in Figure 3.10. Final test samples were 150 mm long, 76 mm wide, and 3.8 mm high. Three replicates of each mix were tested.

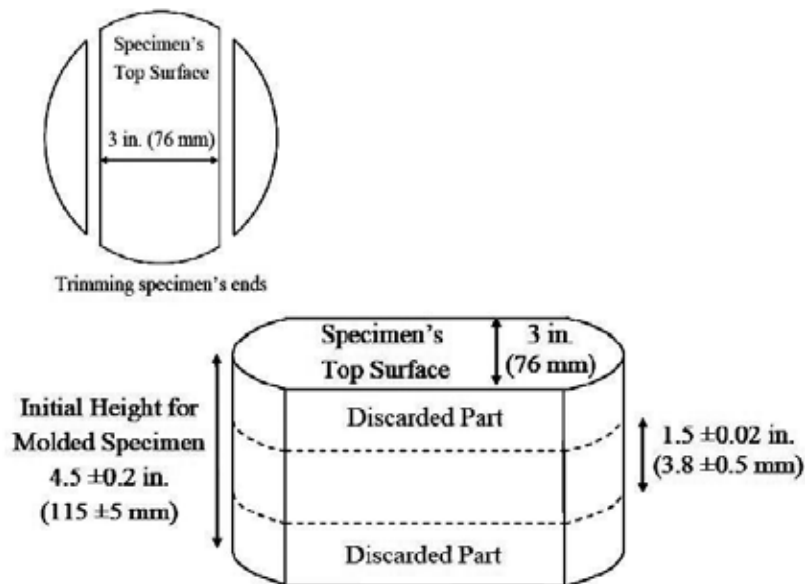


Figure 3.10: Sample Fabrication (TEX-248-F)



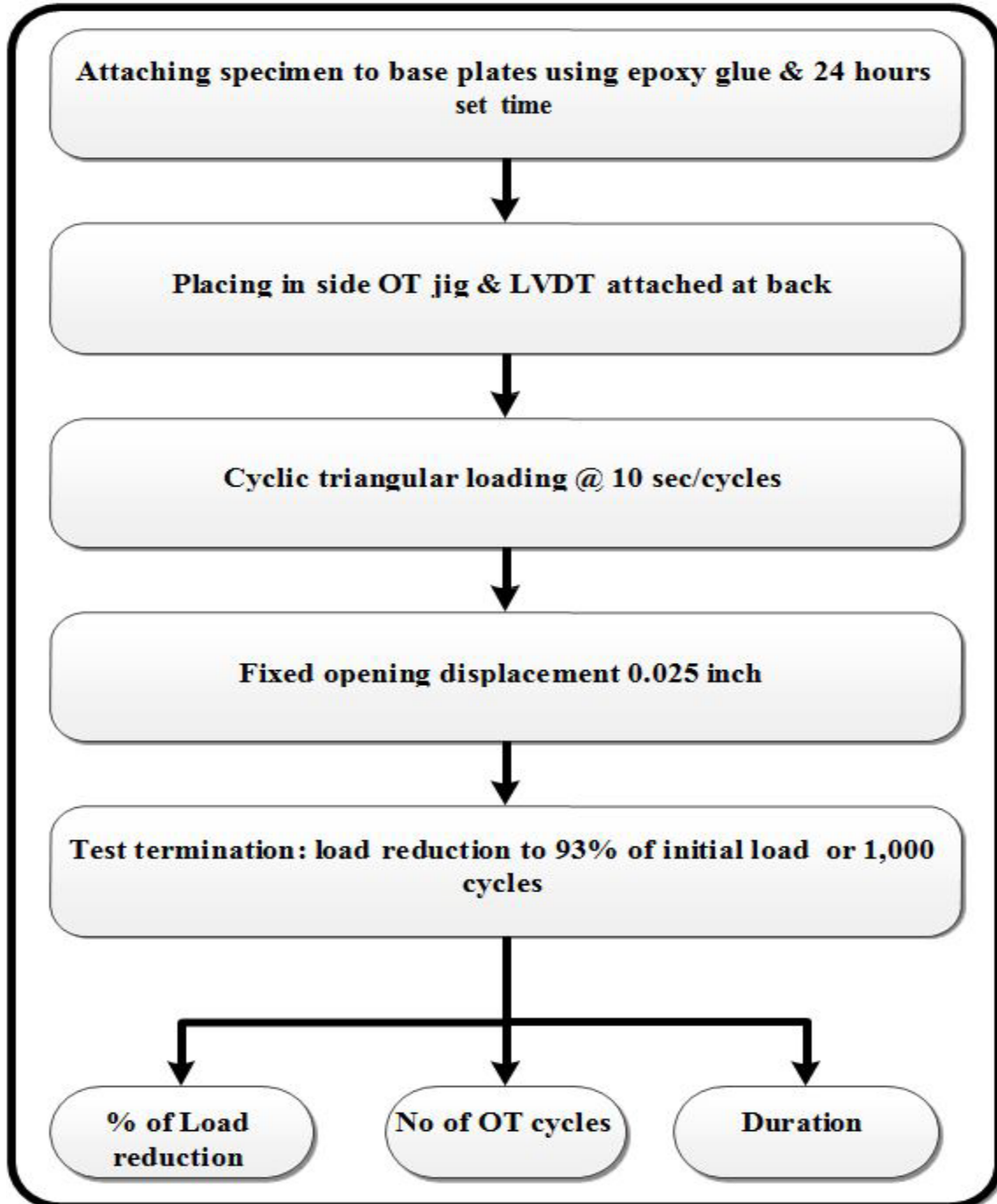
Epoxy glue was used to affix trimmed samples to two metal plates which had gaps between them. Figure 3.11 shows a glued sample to the OT test plates that had gap of 4.2 mm between the plates. A heavy weight rested over the newly glued sample, and the glue was allowed to cure for 24 hours. The test was then performed using the AMPT in which the sample was attached to the base plate. One Linear Variable Differential Transformer (LVDT) was also attached to the sample. Figure 3.12 shows the test in the AMPT environment.



**Figure 3.11: Glued OT Sample**



**Figure 3.12: Sample Mounted Inside AMPT**



**Figure 3.13: Flow Chart of OT Test**

Source: Aziz (2013)

The test temperature was 25 °C according to TEX-248-F test procedure. Samples were preconditioned in a temperature-controlled chamber for 2 hours at that temperature. The loading rate was 10 seconds per cycle (5 seconds loading and 5 seconds unloading). The load pulse

configuration was cyclic triangular displacement-controlled waveform at the standard maximum opening of 4.2 mm. During loading, repetitive movement of only one plate along the length of the sample resulted in tensile stress directly at the center of the sample. The test was terminated when the sample passed 1,000 OT cycles or at a load reduction of 93%, whichever came first. TxDOT has set a minimum of 300 OT cycles in order for an HMA mixture to be acceptable. Recorded parameters at the completion of the test were the number of OT cycles, applied load, and displacement along the load line. Figure 3.13 shows a flow chart of the OT test process.

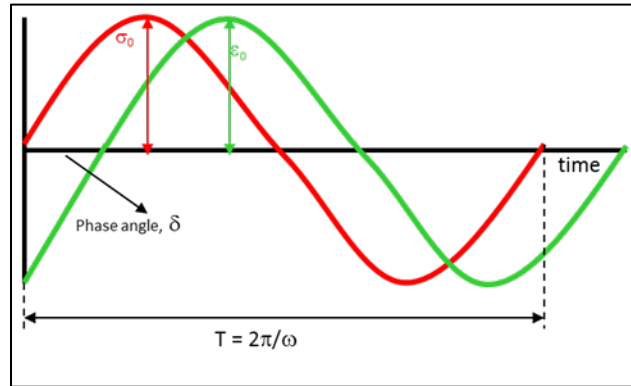
### 3.8 Dynamic Modulus Test

The dynamic modulus test is typically performed on HMA mixtures in order to measure stiffness. In this study, dynamic modulus tests were conducted according to AASHTO TP 62-07 (2013). Cylindrical samples with diameters of 150 mm and heights of  $172 \pm 5$  mm were compacted in an SGC to an appropriate level of air voids to ensure  $7 \pm 1\%$  air voids in the cored test sample. According to AASHTO TP 62-07, cored samples for the test must have diameters of 100 to 104 mm and heights of 150 mm. Three LVDTs were used to measure deformation at three locations, as shown in Figure 3.14.



**Figure 3.14: Dynamic Modulus Test Sample Inside AMPT**

In this test, a compressive sinusoidal load, shown in Figure 3.15, was applied at various temperatures and loading frequencies. Dynamic modulus was calculated by dividing peak-to-peak stress by peak-to-peak strain according to stress-strain developed from this test.



**Figure 3.15: Sinusoidal Stress-Strain Response Curve in the Dynamic Modulus Test**

Dynamic Modulus tests were performed at three temperatures: 4, 21, and 37 °C. Temperatures of -10 °C and 54 °C were also suggested in the guideline, but they could not be used because LVDTs did not perform accurately at temperatures lower than zero and epoxy began melting at temperatures higher than zero (Sabahfar, 2012). The test was performed for six frequencies (0.1, 0.5, 1, 5, 10, and 25 Hz) between axial strains of 50 to 150 microstrains. Samples were preconditioned at 4 °C overnight and tested for six frequencies. Then the same sample was conditioned at 21 °C for 1 hour and tested at six frequencies. The same sample was again conditioned for 2 hours at 37 °C and tested. Three replicates of each mix were tested.

After conditioning at the desired temperature, the sample was placed inside the AMPT. Because the loading mode was compression, screws were not necessary for mounting. Proper alignment of the loading head with the sample was achieved, and thin Neoprene pads were used to avoid end friction. The test was terminated after testing the sample at each frequency, beginning from the lowest one. Figure 3.16 shows a flow chart of the test process.

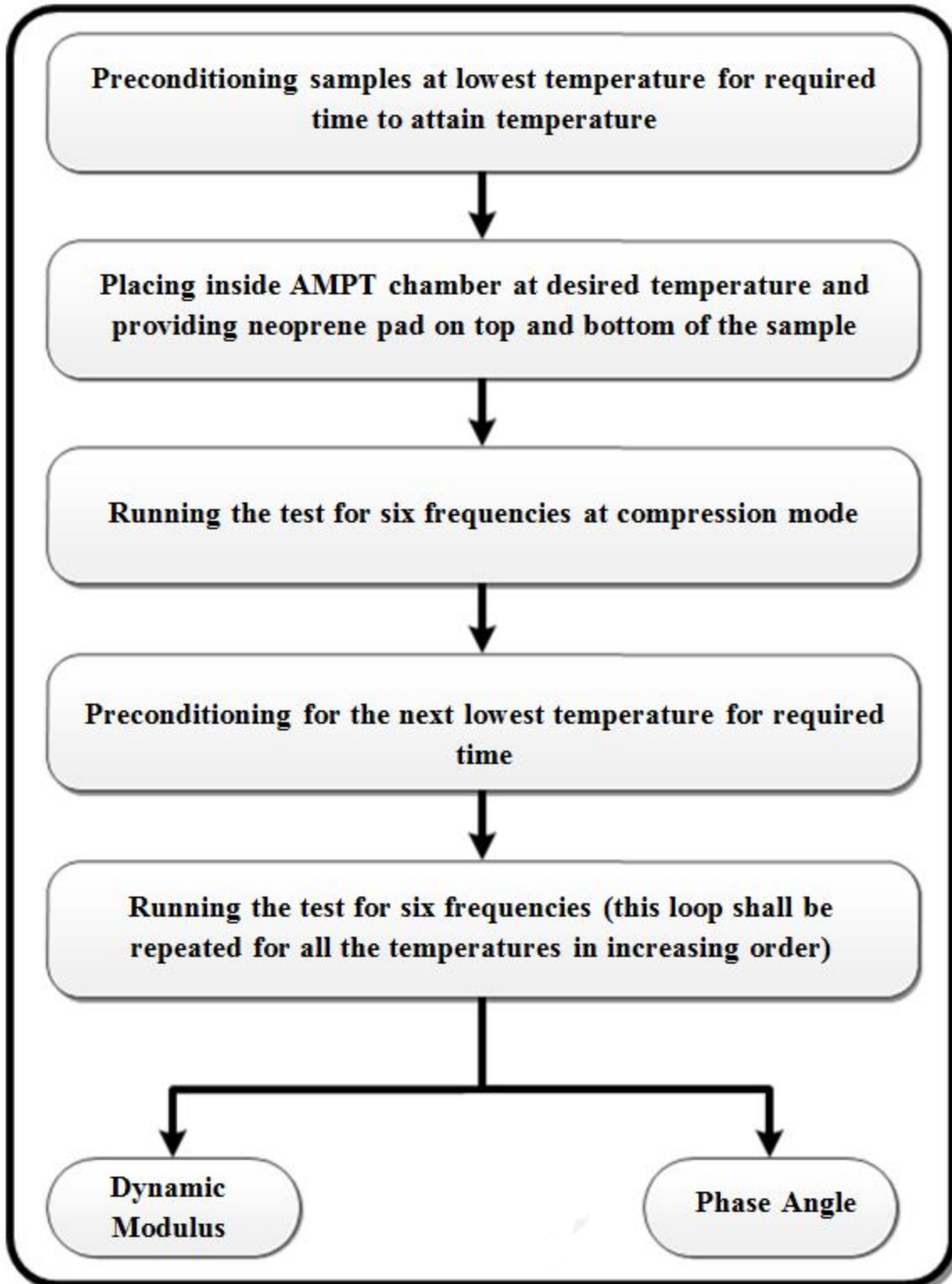


Figure 3.16: Flow Chart of the Dynamic Modulus Test

### 3.9 Simplified Viscoelastic Continuum Damage Test

VECD theory, a mechanistic approach to study fatigue performance of HMA mixture, depends more on theoretical considerations than phenomenological approaches like in the beam fatigue test. Phenomenological approaches tend to show large margin of errors, but the Simplified Viscoelastic Continuum Damage (S-VECD) approach is gaining gradual acceptance because it is capable of predicting fatigue performance of HMA mixes with errors in tolerable limits.

Sample preparation in the S-VECD test was similar to sample preparation for the dynamic modulus test. Samples were compacted in an SGC to heights of 180 mm and diameters of 150 mm at the predetermined air voids to ensure  $7\pm 0.5\%$  air voids in the trimmed samples that were cored out of the compacted samples. The trimmed test samples had diameters of 100 to 104 mm and heights of 127 to 132 mm. Figure 3.17 shows a cored sample. Height and diameter measurement were taken at five places for each sample.



Figure 3.17: S-VECD Sample Coring



Because the sample is loaded repeatedly in pull-pull mode during the test, strong glue was required to ensure that failure did not occur along the glue or near the end. A customized gluing jig was used to apply Devcon 10110, also known as steel putty, in order to attach the top and bottom plates to the sample, as illustrated in Figure 3.18. However, the steel putty had to cure for at least 4 hours in order to achieve appropriate adherence.

Three LVDTs were attached to the specimen in order to measure deformation. Epoxy was used to attach the sample to three sets of clips with 75-mm gauge length located 120° from each other.



**Figure 3.18: S-VECD Sample Gluing**

The applicable test temperature for these tests was suggested in AASHTO TP 107-14 (2014). The temperature for the test in this study was selected based on 98% reliability, and the PG was determined from the LTPPBind at the location of interest. The PG was PG 64-22 for this project. AASHTO TP 107-14 required that the lower end of the binder grade in absolute value to be subtracted from the upper grade and to be divided by 2. Then 3 °C had to be subtracted from

this result. Maximum value is suggested not to exceed 21 °C. If the value after calculation comes higher than 21 °C, 21 °C should be used. In this study, the test temperature was selected as 18 °C depending on binder grade PG 64-22. Test samples were conditioned for two to four hours in a temperature-conditioning chamber to achieve the test temperature.



**Figure 3.19: S-VECD Sample Mounted Inside AMPT**

In this study, the S-VECD test was performed using AMPT. The sample was tightly attached to the top and bottom plates of the machine using screws, as shown in Figure 3.19. The test was performed with three sequential steps described in the following paragraphs.

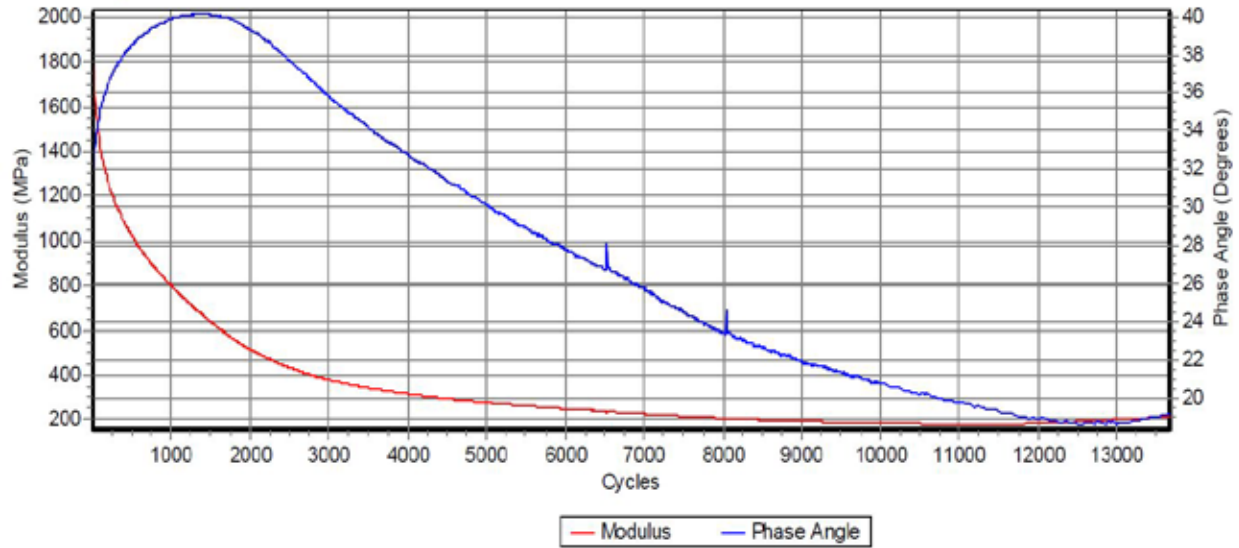
- Application of zero loads: After the sample was glued to the top and bottom plates and placed in AMPT, the temperature-controlled chamber was activated. Application of zero loads and proper attachment of the sample ensured that no confining pressure and no torsional stress occurred.



- Dynamic modulus fingerprint test: This test step, performed for 55 tension-compression loading cycles at 10 Hz frequency, provided an initial estimation of dynamic modulus of the sample. After completion of the fingerprint test, a 20-minute rest period was provided so the samples could recover from the loading effect.
- Direct tension fatigue test: After the rest period, a repeated-load fatigue test was performed using the direct tension mode. The test evaluated deterioration in dynamic modulus of the sample due to fatigue loading cycles. The test was conducted for 50 to 70 peak-to-peak microstrains at 10 Hz frequency. AASHTO TP 107-14 (2014) suggested 300 microstrains as the strain level for the initial sample. Strain levels of the next two samples were decided based on the number of fatigue cycles found for the first sample. Table 3.8 presents guidelines for deciding consecutive initial microstrains for two other replicates of the same mixture. Figure 3.20 shows a diagram of a typical dynamic modulus versus number of fatigue cycles.

**Table 3.8: Microstrain Level Selections for Replicates 2 and 3 in S-VECD test**

Case	$\epsilon_2$	$\epsilon_3$
$500 < N_{f1} < 1000$	$\epsilon_1 - 100$	$\epsilon_1 - 150$
$1,000 < N_{f1} < 5,000$	$\epsilon_1 - 50$	$\epsilon_1 - 100$
$5,000 < N_{f1} < 20,000$	$\epsilon_1 + 50$	$\epsilon_1 - 50$
$20,000 < N_{f1} < 100,000$	$\epsilon_1 + 100$	$\epsilon_1 + 50$
$100,000 < N_{f1}$	$\epsilon_1 + 150$	$\epsilon_1 + 100$



**Figure 3.20: Typical Numbers of Fatigue Cycles in S-VECD Test**

The test was stopped when microcracks began to appear. Change in the pattern of diminishing dynamic modulus curve was also an indication of sample failure. Figure 3.21 shows a completely-cracked sample and a sample after appearance of microcracks. A flow chart for this step is presented in Figure 3.22.



**Figure 3.21: (a) Fully-Cracked S-VECD Sample, (b) S-VECD Sample with Micro-Cracks**

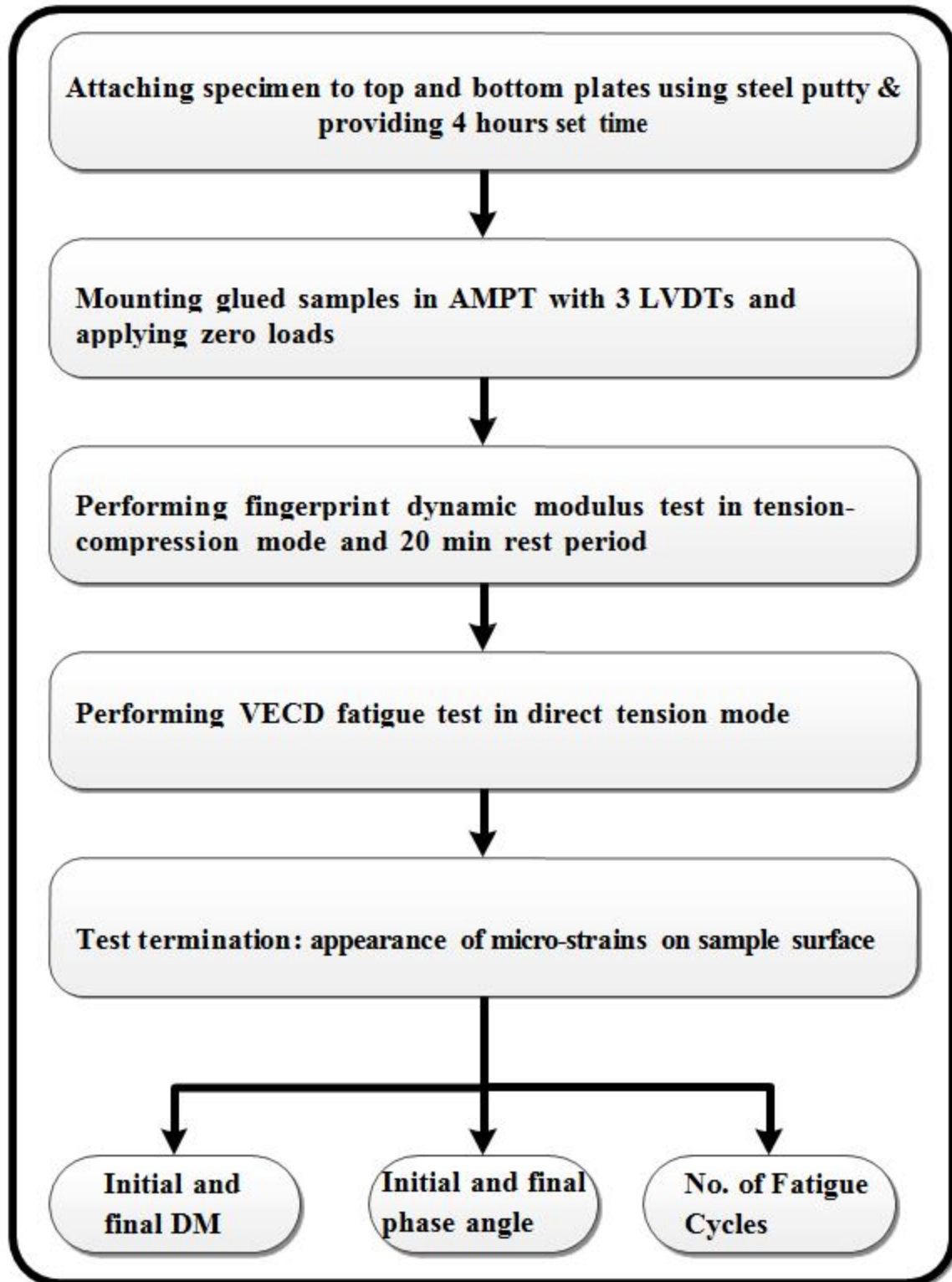


Figure 3.22: Step-by-Step Operations of S-VECD Test

Fatigue performance prediction was done using the software Alpha-F (Xie et al., 2015). Test results were used to simulate the condition of a damaged sample and its changes in dynamic modulus properties in order to attain final level of damage. Equation 3.7 (exponential model) and Equation 3.8 (power model) were calibrated for each mix using results from three replicates, and mixtures were compared to each other based on parameter values in these equations.

$$C = e^{aS^b} \quad \text{Equation 3.7}$$

$$\text{or, } C = 1 - yS^z \quad \text{Equation 3.8}$$

Where:

S is cumulative damage to attain first fatigue crack-line;

C is pseudo secant modulus at the time of crack initiation;

a, b are fitting coefficients for exponential model; and

y, z are fitting coefficients for the power model.

In this study, the power model was used to compare fatigue performance of various mixtures. The average number of cycles to fatigue failure in this test was also determined using the software.

## Chapter 4: Results and Analysis

### 4.1 Moisture Susceptibility Test Results

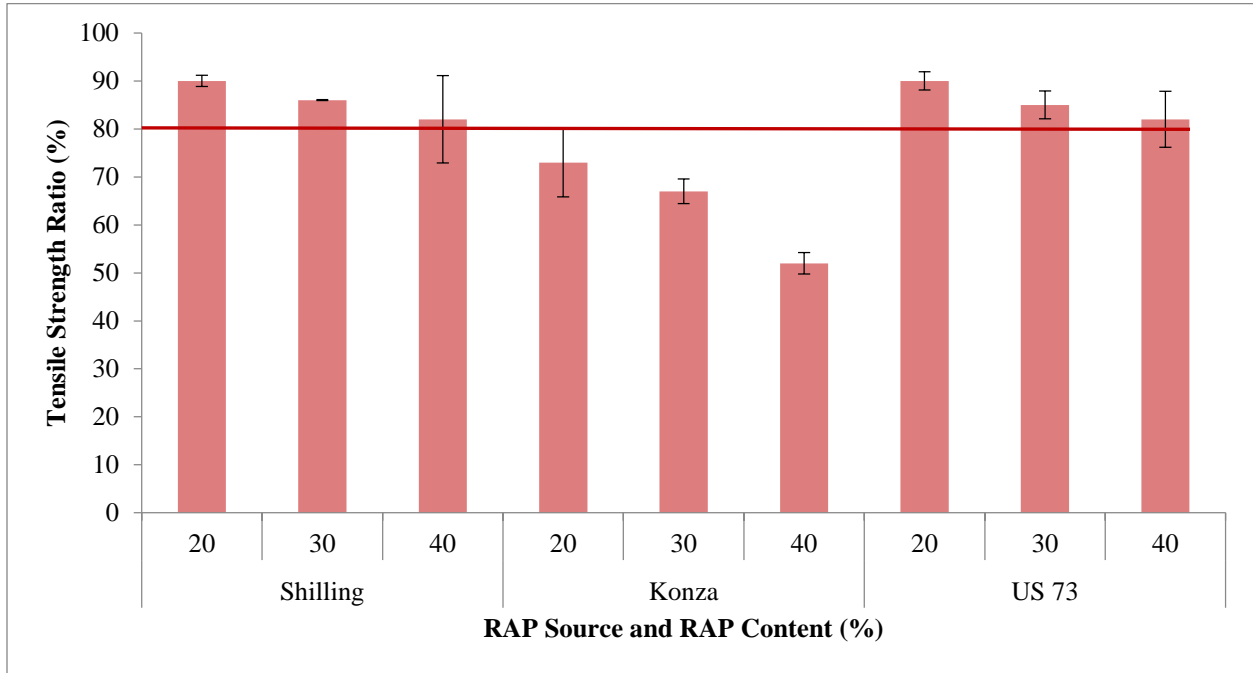
The moisture susceptibility test was performed according to the Kansas Standard Test Method KT-56 (2014) in order to assess moisture susceptibility of HMA mixtures with high RAP content. Table 4.1 tabulates results of the test. Three replicates for each conditioned and unconditioned state were tested in indirect tension. The table also shows the Tensile Strength Ratio (TSR) of the conditioned samples to the unconditioned samples. Sabahfar (2012) performed tests for the Shilling and Konza RAP sources, and tests for the US-73 RAP source were conducted in this study.

**Table 4.1: Moisture Susceptibility Test Results**

RAP Source	RAP Content (%)	Air Voids (%)		Average Tensile Strength (lbs.)		Tensile Strength Ratio (%)
		Conditioned	Unconditioned	Conditioned	Unconditioned	
Shilling	20	7.2	7.2	3,390	4,430	90
	30	6.8	6.7	4,257	4,964	86
	40	6.6	6.6	4,425	5,391	82
Konza	20	6.9	6.9	3,249	4,473	73
	30	6.8	6.8	3,307	4,906	67
	40	7.0	6.9	3,255	6,235	52
US-73	20	7.1	7.1	4,289	4,766	90
	30	7.3	7.2	4,101	4,825	85
	40	7.0	7.1	4,007	4,887	82

Figure 4.1 illustrates TSR values for mixtures in this study. For all sources, TSR values decreased as the RAP content in the mixture increased. Mixtures with Shilling and US-73 RAP met minimum TSR requirements of KDOT (80%) even at the highest RAP content (40%), but mixtures with the Konza RAP did not meet minimum requirements of KDOT. Data in Table 4.1 show that these mixtures had very indirect tensile strength in the unconditioned state. Conditioning led to very low indirect tensile strength of these mixtures; consequently, TSR

values were lower. The binder in the Konza RAP was highly aged. The freeze cycle in KT-56 was presumed to have ruptured the asphalt films during the freeze cycle of conditioning, leading to low indirect tensile strength after conditioning.



**Figure 4.1: TSR Test Results**

## 4.2 Semi-Circular Bending Test Results

The SCB test was performed in order to investigate cracking resistance of Superpave mixtures with varying RAP content. The S-SCB test was performed to evaluate the rate of energy release in order to initiate fracture under increasing load. In the R-SCB test, repeated load was applied on the samples in order to simulate recurring wheel load. Aziz (2013) performed S-SCB and R-SCB tests for mixtures with Shilling and Konza RAP. In this study, these tests were repeated for the US-73 RAP source.

### 4.2.1 Static SCB Test Results

Average FE and stiffness value of each Superpave mixture are tabulated in Table 4.2. For each mixture, three replicates were tested. Bending strain and stress represented by displacement and load, respectively, at the termination of the test somewhat indicated ductility of HMA

mixtures (Aziz, 2013). The coefficient of variation (COV) of test results for each mixture is indicated in parentheses under the corresponding FE values in Table 4.2. These values indicate fairly good repeatability of the test.

**Table 4.2: SCB Test Results**

RAP Source	RAP Content (%)	Air Voids (%)	Peak Load (KN)	Average Fracture Energy (J/m <sup>2</sup> )*	Average Stiffness (MPa)
Shilling	20	7.3	2.55	2,098.7 (8.6)	20,491
	30	7.2	3.15	2,064.9 (10.4)	29,440
	40	7.3	3.88	2,173.8 (8.1)	58,594
Konza	20	7.2	2.48	1,632.2 (6.0)	26,804
	30	7.1	1.72	1,769.4 (5.2)	27,407
	40	7.2	3.12	1,884.0 (18.6)	33,843
US-73	20	7.3	1.51	1,161.2 (15.2)	11,124
	30	7.1	1.87	1,098.6 (8.2)	18,313
	40	7.2	2.04	1,701.1 (15.6)	18,466

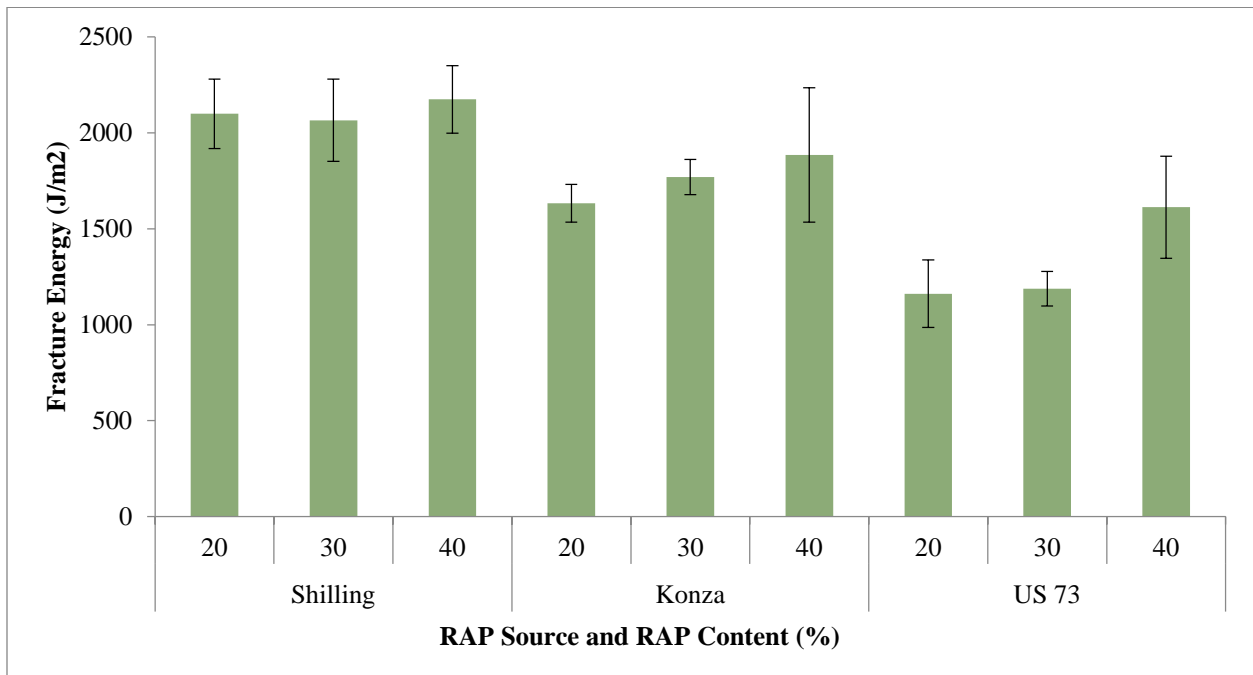
\*COV shown in parentheses.

According to Table 4.2, for mixtures with Shilling RAP, FE for mixtures with 20% RAP and 30% RAP are not numerically similar values, but for 40% RAP, FE value is abruptly higher. Area under the load-deflection curve at a strain-controlled monotonic loading setup is an indication of material stiffness (Royslance, 2001). As mentioned, a mixture containing 40% RAP is expected to have a higher stiffness value. Using only this FE value, no definitive conclusion regarding cracking performance of viscoelastic HMA mixtures could be reached. Cracking performance is associated with ductility of a mixture under loading before initiating the crack. For viscoelastic materials, fracture mode is cup-and-cone fracture. If a material releases energy in a higher rate in order to reach fracture, the material is comparatively brittle and thus weaker in cracking (Anderson, 1995).

Stiffness values of mixtures presented in Table 4.2 also increased as RAP content increased. Again, mixtures containing 20% and 30% RAP did not show a large difference in

stiffness values, but the mixture with 40% RAP had much higher stiffness value, as shown in Figure 4.2. As illustrated in the figure, this trend in FE values, in which increased RAP content increased the FE, was also observed for mixtures with other RAP sources, leading to the release of energy at a higher rate. Stiffness values also increased with increasing RAP content, as shown in Figure 4.3. Mixtures containing RAP from US-73 had the lowest FE and stiffness values. Figure 4.3 also shows that mixtures with Shilling RAP had the highest stiffness values.

In Figure 4.4, rates of energy released per minute for mixtures are compared. For Shilling and US-73 RAP sources, an increasing trend of energy release rate was observed, indicating that the mixtures became brittle with increasing RAP content. As from unit ligament area, per minute energy release rate was higher for mixtures containing 40% Shilling RAP or 40% US-73 RAP. The samples failed at much faster rates and were more susceptible to cracking than mixtures containing 20% and 30% RAP.



**Figure 4.2: S-SCB Test FE Results**



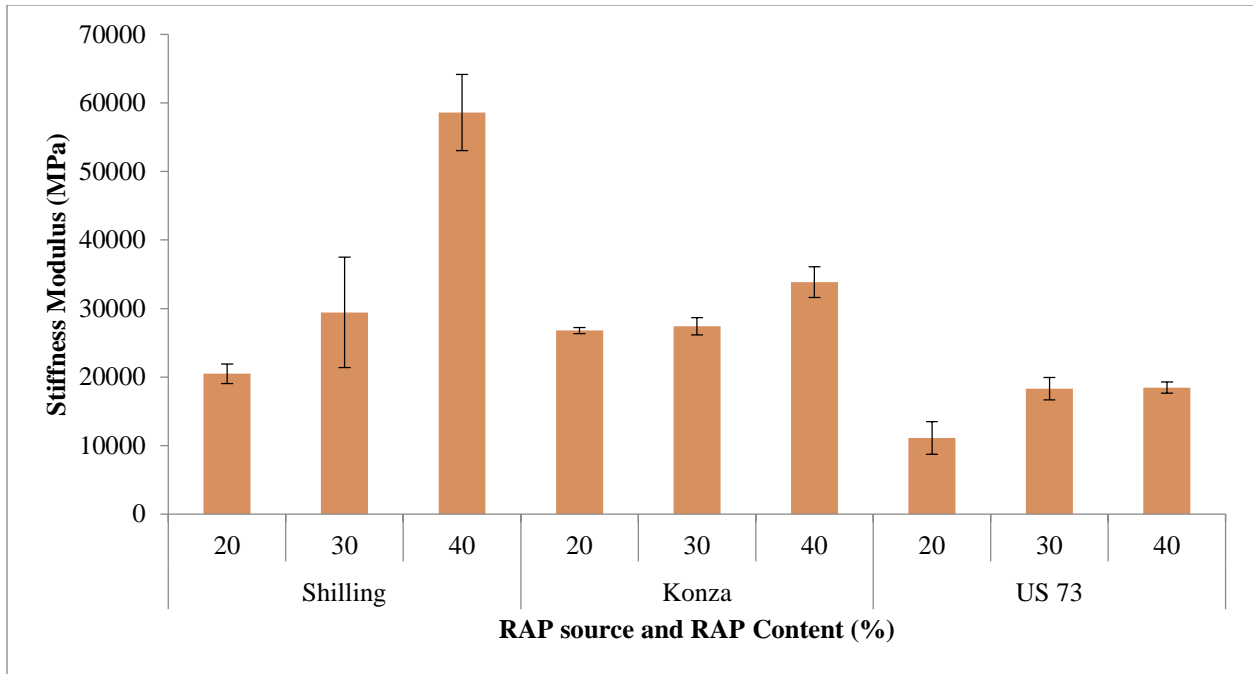


Figure 4.3: S-SCB Stiffness Results

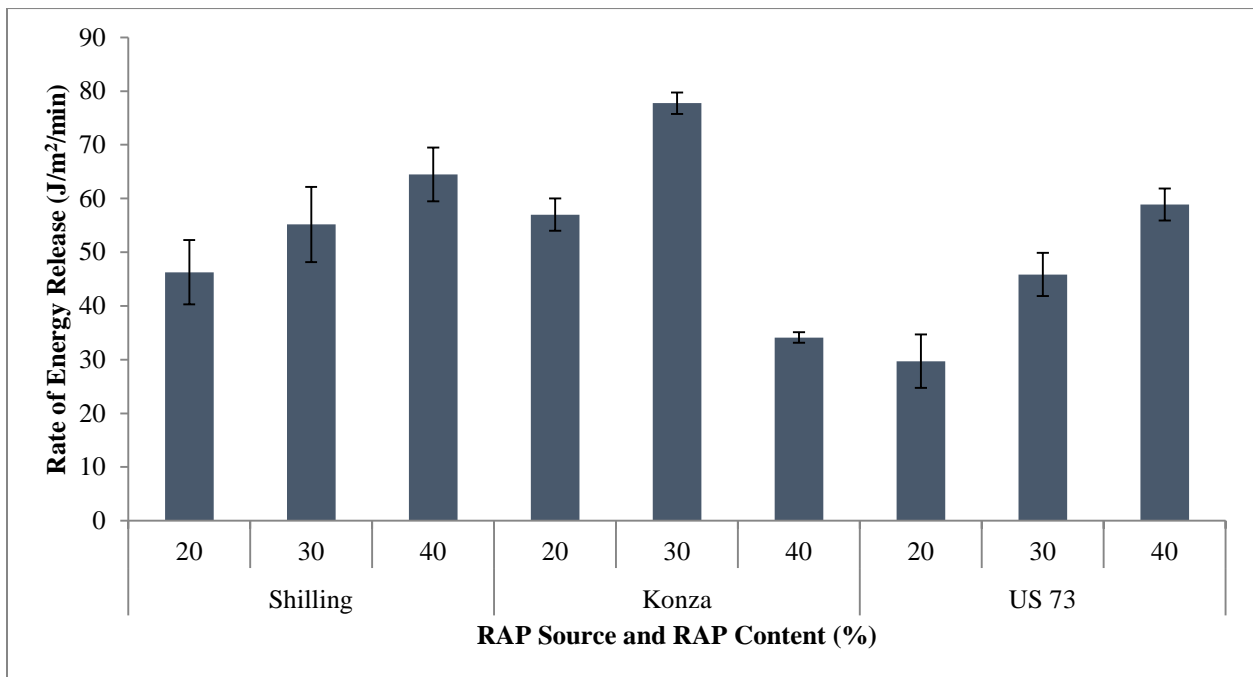


Figure 4.4: Rate of Energy Release Comparison

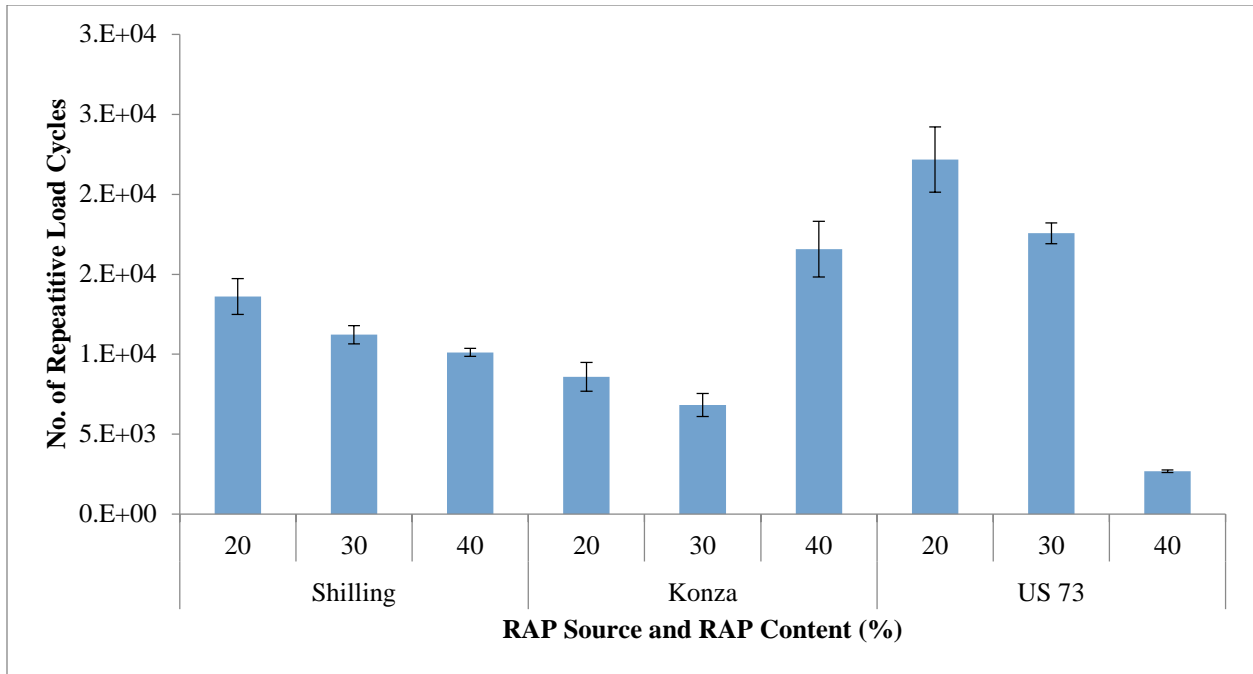
However, according to the rate of FE release, cracking resistance of this mixture was the lowest. Mixtures containing RAP from US-73 had the lowest stiffness values and the highest cracking resistance. Therefore, high stiffness of HMA is not necessarily a safeguard against cracking.

#### 4.2.2 Repetitive SCB Test

The R-SCB test was performed using four input loads (30, 40, 50, and 60%) derived from the initial peak load obtained in the S-SCB test. Previous research has indicated that a load value of 50% peak load most appropriately simulates fatigue performance; therefore, the R-SCB Test was used for this study. Table 4.3 lists the average number of load cycles for each mixture for 50% of the peak load input in the R-SCB test. Figure 4.5 shows the results graphically. The number of load cycles was measured until cracks propagated throughout the total thickness of the specimen under repetitive loading. Therefore, the higher the number of load cycles until failure, the higher the cracking resistance.

**Table 4.3: R-SCB Test Results**

RAP Source	RAP Content (%)	Initial Peak Load (kN)	Input Load (kN)	Average No. of Load Cycles	COV (%)
Shilling	20	2.55	1.28	13,607	8.2
	30	3.15	1.58	11,227	5.1
	40	3.88	1.94	10,113	2.5
Konza	20	2.48	1.24	8,593	10.4
	30	1.72	0.86	6,830	10.6
	40	3.12	1.56	16,567	10.5
US-73	20	1.51	0.76	22,170	9.2
	30	1.87	0.94	17,568	3.7
	40	2.04	1.02	2,680	3.0



**Figure 4.5: Number of Load Cycles of R-SCB Test**

Results in Table 4.3 and Figure 4.5 indicate that increasing RAP content reduced cracking resistance of mixtures with Shilling and US-73 RAP. However, an anomaly was observed for the Konza RAP source: mixtures containing 20% and 30% RAP from this source showed almost an equal number of load cycles to fracture, but the mixture with 40% RAP displayed the maximum number of load cycles. According to Figure 4.5, the inference can be made that the mixtures with US-73 RAP showed better cracking resistance than the other two sources. However, the US-73 source was from a highway project, whereas the other two RAPs were obtained from the processed stockpile of contractors at RAP plants.

### 4.3 Texas Overlay Tester Test Results

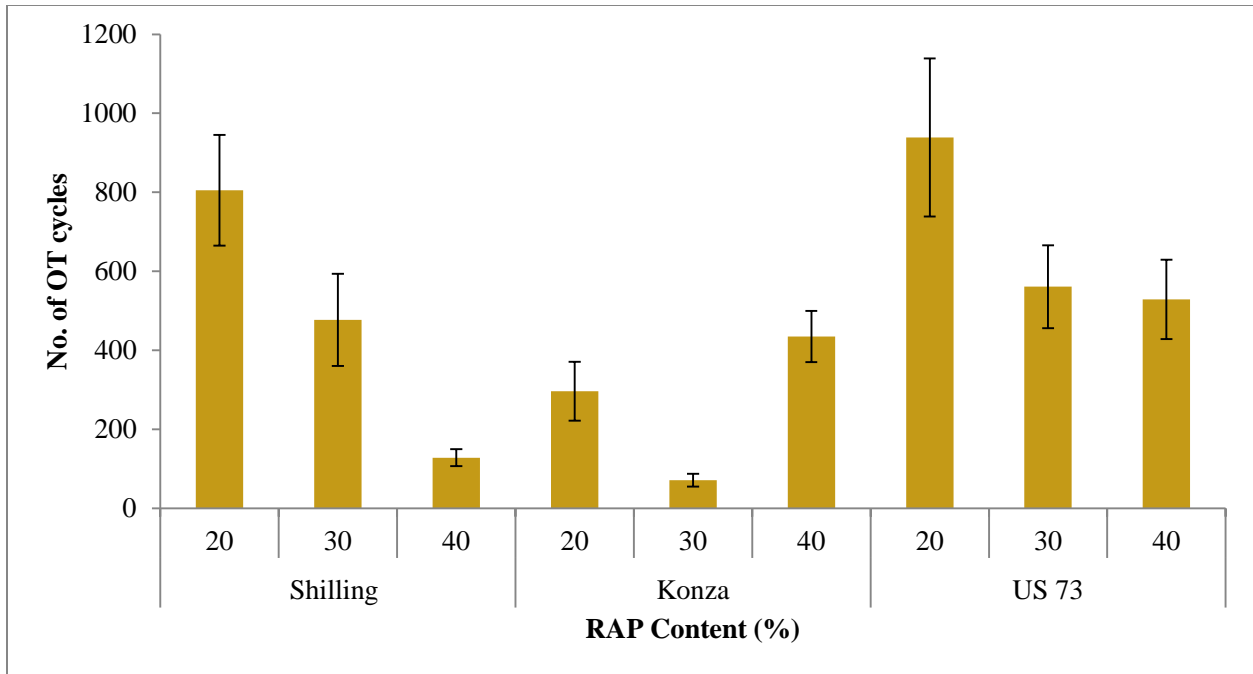
The Texas OT test was performed in order to evaluate reflective cracking resistance of HMA mixtures with high RAP content. Triplicate specimens of mixtures with three RAP contents from three RAP sources were tested according to TEX-248-F (2014) test method. Table 4.4 lists peak loads in the load-displacement curve. The basis of performance comparison of mixtures is the number of OT load cycles before failure. Results for the nine mixtures are

summarized in Table 4.4 and presented in Figure 4.6. Samples were compacted to  $7\pm 1\%$  air voids. Peak load in the load-displacement curve were also recorded for corresponding mixtures. The COV among each replicate block of the test is presented in parentheses for the respective mixture data. This test showed much higher COVs than the R-SCB tests.

**Table 4.4: Texas OT Test Results**

RAP Source	RAP Content (%)	Air Void (%)	Average Initial Peak Load (KN)	Average No. of OT Cycles to Failure ( $N_{OT}$ )*	Duration (min)
<b>Shilling</b>	20	7.2	2.35	805 (17.4)	134
	30	7.3	2.9	477 (24.5)	80
	40	7.1	3.28	128 (16.9)	21
<b>Konza</b>	20	7.1	2.66	296 (25.2)	49
	30	7.3	2.1	71 (22.7)	12
	40	7.1	3.09	435 (14.9)	73
<b>US-73</b>	20	7.2	1.5	939 (21.3)	97.7
	30	6.9	0.7	561 (18.7)	93
	40	7.1	0.5	529 (19.0)	47.3

\*COV shown in parentheses.



**Figure 4.6: Number of Cycles in Texas OT Test**

OT results confirmed findings from R-SCB tests. Mixtures with RAP from Shilling and US-73 performed the best. In both cases, the number of load cycles to failure (attainment of 1,000 OT cycles or 93% reduction of initial peak load) declined with increasing percentages of RAP. For both sources, mixtures with 40% RAP showed significantly lower cracking resistance. Therefore, high RAP content appears to make HMA mixtures more susceptible to reflective cracking. TxDOT requires a minimum number of 300 OT cycles for an acceptable HMA mixture. In this study, the mixture with 40% RAP from Shilling did not meet that requirement. This was also observed for mixtures with 20% and 30% RAP from Konza, with the exception of the mixture with 40% RAP from Konza which showed acceptable OT load cycles.

#### **4.4 Dynamic Modulus Test Results**

Dynamic modulus test results described stress-strain behavior of nonlinear viscoelastic materials such as HMA. The dynamic modulus test was performed at three temperatures (4, 21, and 37 °C) and six loading frequencies (0.1, 0.5, 1, 5, 10, and 25 Hz) in compression loading mode in an Asphalt Mixture Performance Tester (AMPT) according to AASHTO TP 62-07 (2013). After completion of the test, AMPT provided outputs for the dynamic modulus and phase

angle for six frequencies at constant temperature. A typical AMPT output file is presented in Figures 4.7 and 4.8.

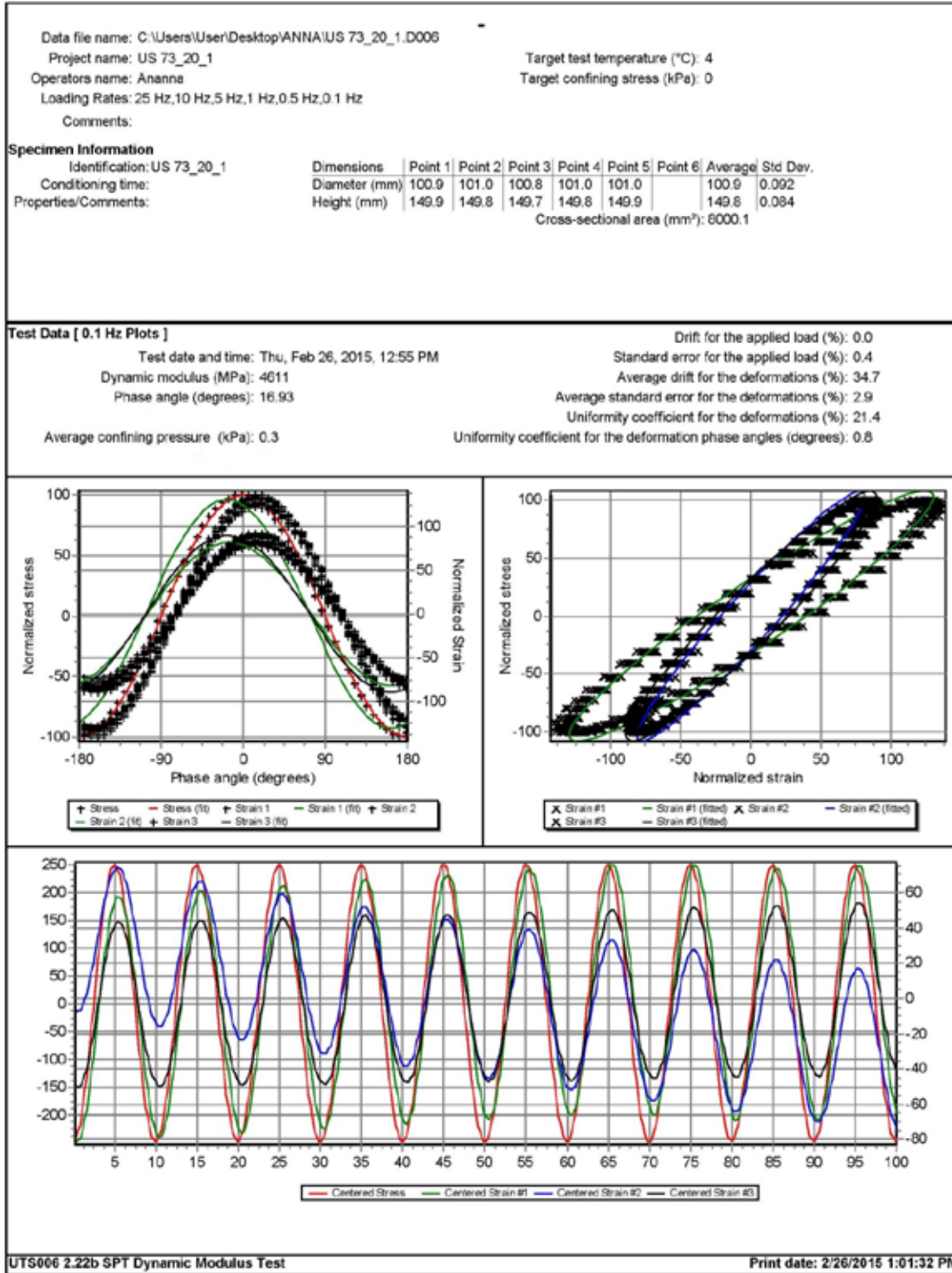
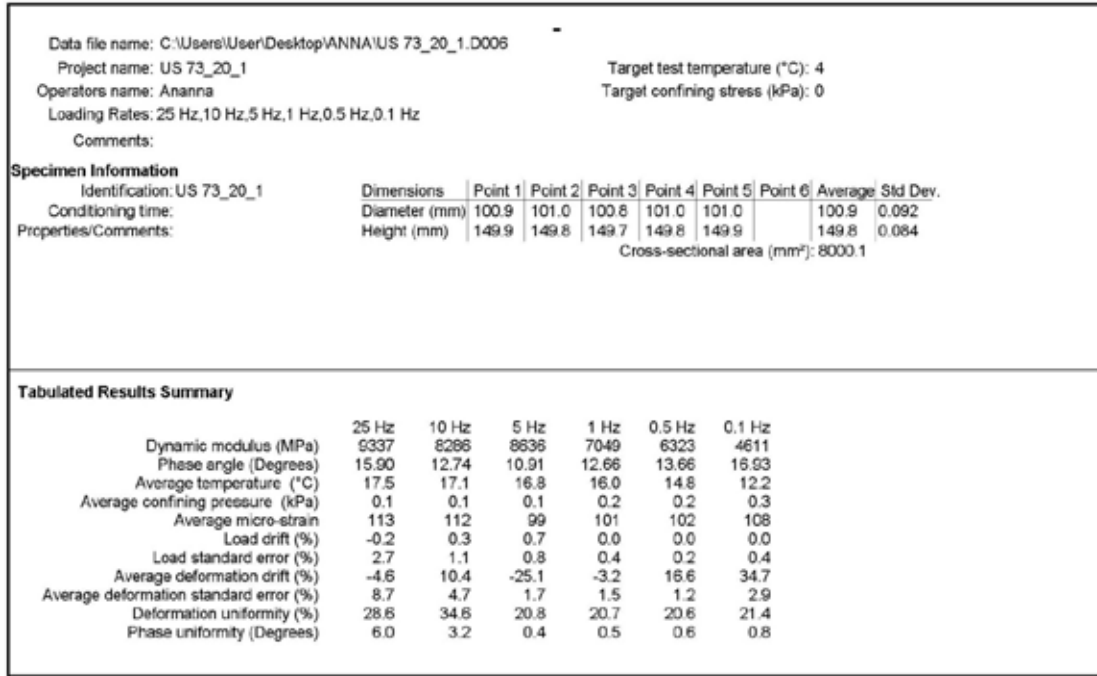
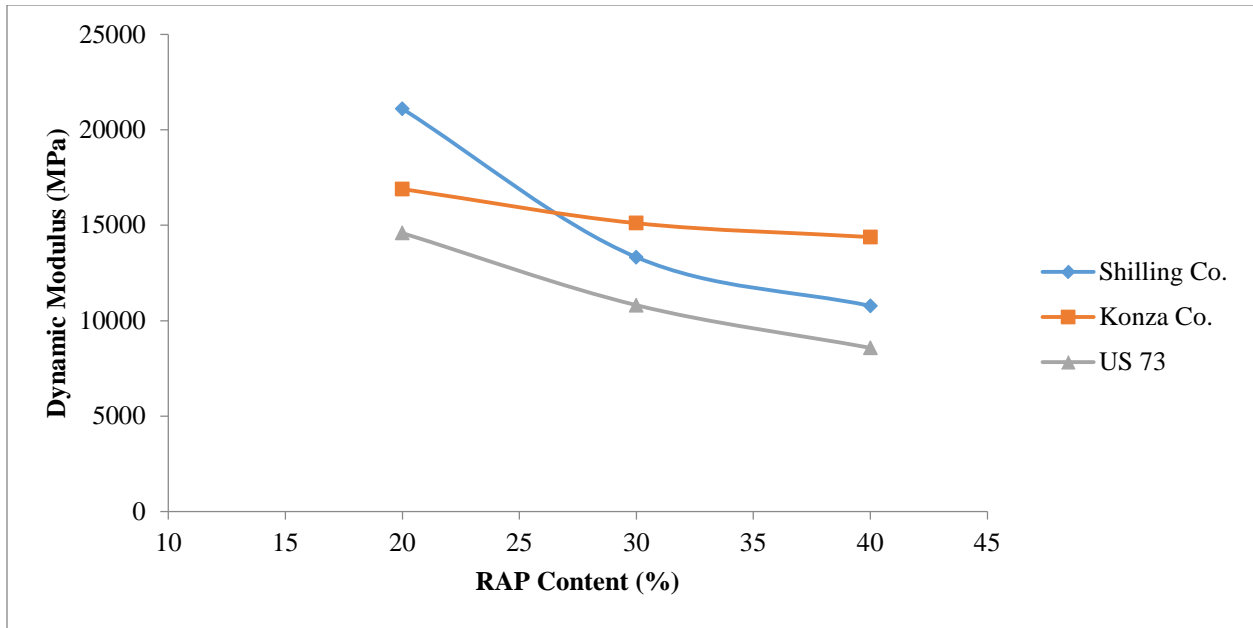


Figure 4.7: Typical Dynamic Modulus Test Output (Screen 1)

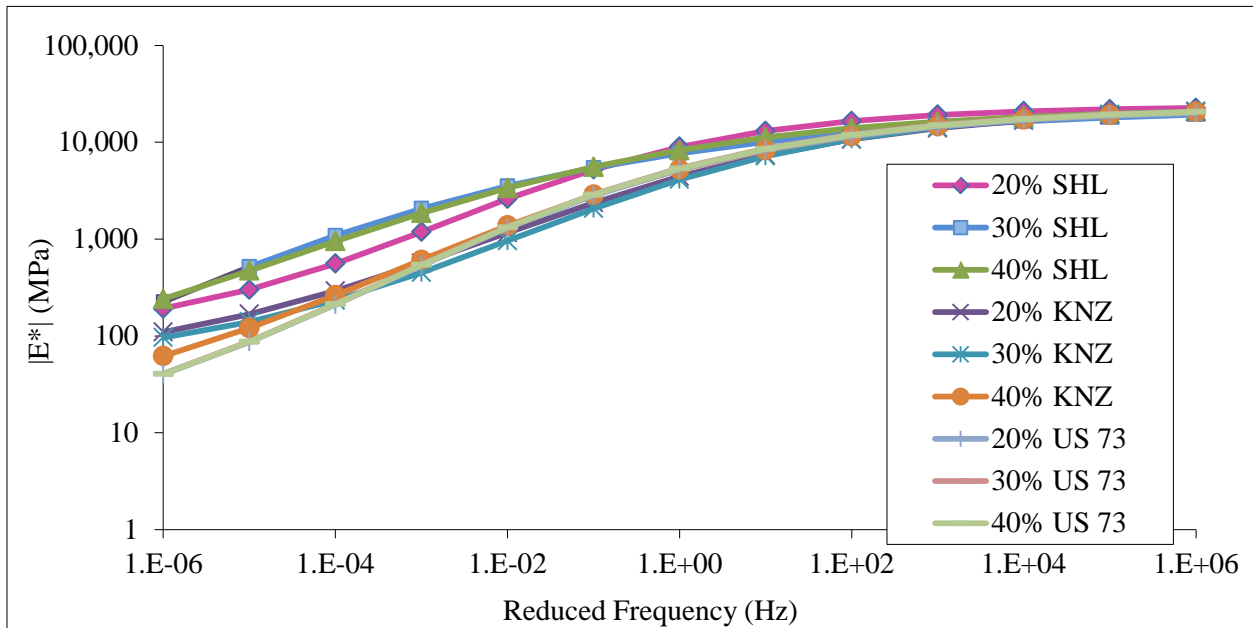


**Figure 4.8: Typical Dynamic Modulus Test Output (Screen 2)**

HMA mixture performance (rutting and fatigue cracking) can be compared based on dynamic modulus  $|E^*|$  and phase angle ( $\phi$ ) values (Bhasin, Button, & Chowdhury, 2004). As loading frequency increases, the dynamic modulus value typically increases. At low temperatures, dynamic modulus of the same mixture is usually greater than that at high temperatures. In this study, dynamic modulus value at 10 Hz frequency was used to compare mixtures. This frequency was selected because it most closely corresponded to highway speeds of approximately 64 km per hour. Figure 4.9 illustrates dynamic modulus values of mixtures at 4 °C. The figure shows that dynamic modulus decreased with increasing RAP content; however, the rate of decrease was not identical for all RAP sources. Although the rates were similar for mixtures with Shilling and US-73 RAP, the mixture with Konza RAP showed a much lower rate of change of dynamic modulus with RAP content.



**Figure 4.9: Dynamic Modulus Value Comparison of RAP Sources**



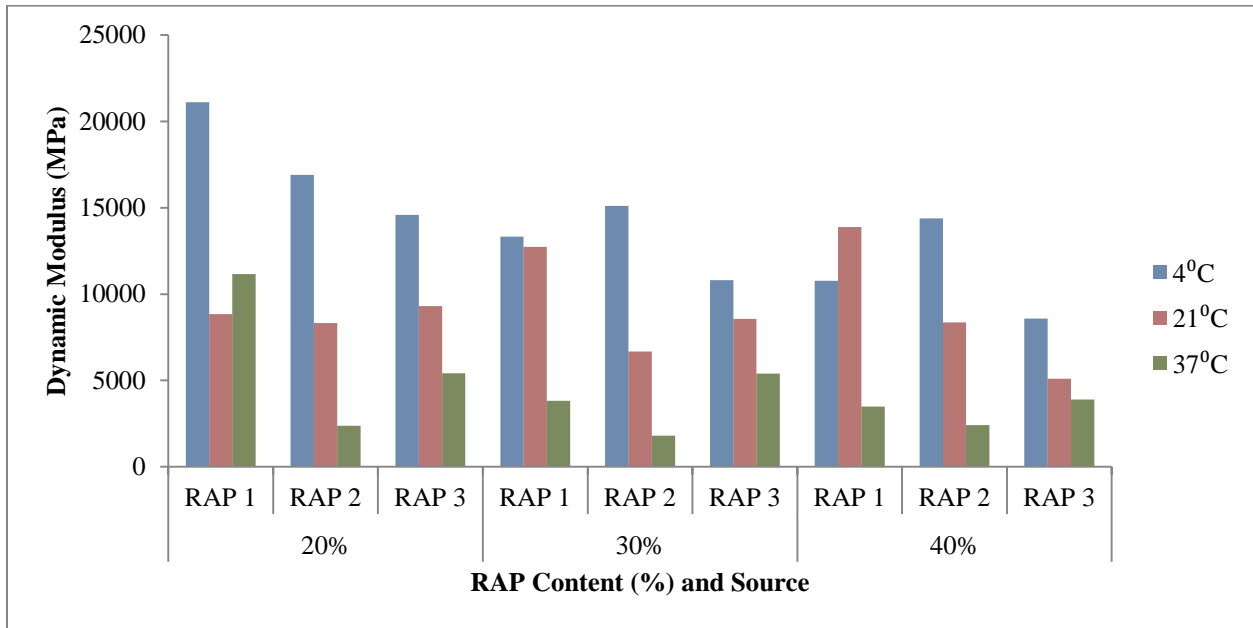
**Figure 4.10: Dynamic Modulus Master Curves**

Dynamic modulus master curves were drawn for all temperatures and frequencies, as shown in Figure 4.10. The figure indicates that dynamic modulus values increased with increasing loading frequency. Master curves are helpful in predicting dynamic modulus values at



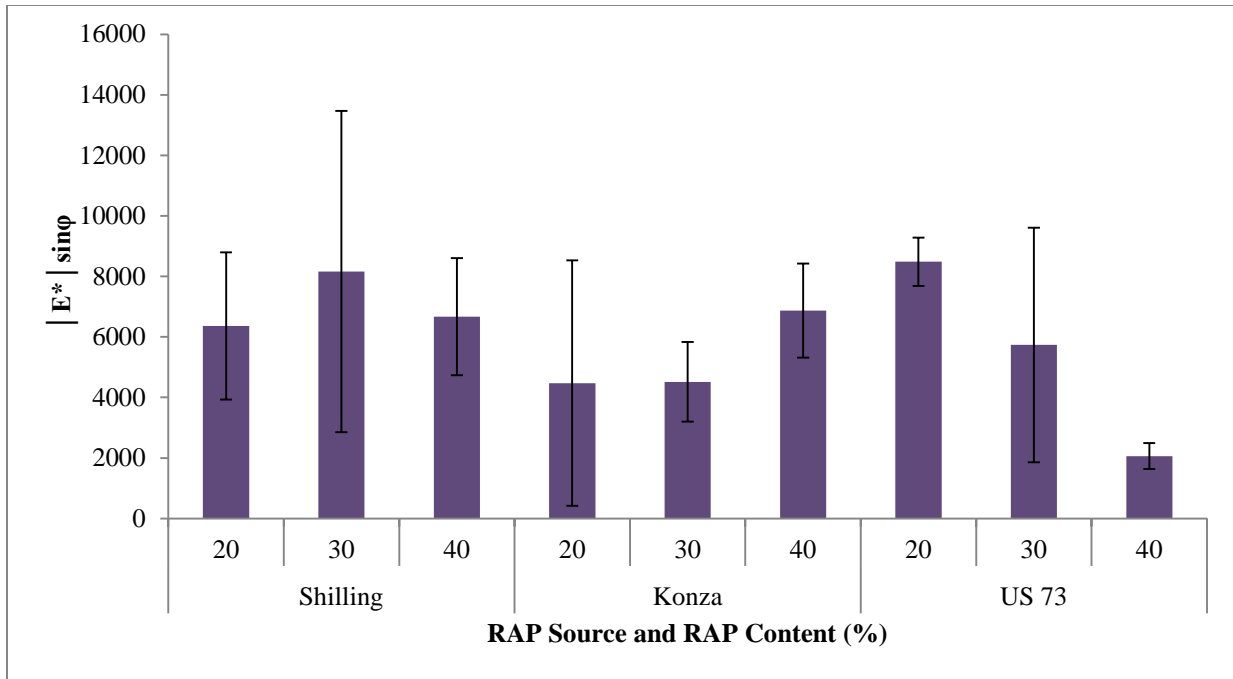
the project’s design temperature and vehicle classification associated with the project during the analysis period.

Figure 4.11 illustrates dynamic modulus values at 10 Hz loading frequency as compared to dynamic modulus values with changing temperatures. For most mixtures, dynamic modulus values decreased with increasing temperature, except for two mixtures: the mixture with 20% Shilling RAP and the mixture with 20% US-73 RAP. The source of this discrepancy is unknown.



**Figure 4.11: Comparison of Dynamic Modulus Values at Various Temperatures**

The cracking factor ( $|E^*| \sin\phi$ ) is an indication of improved fatigue performance (Figure 4.12). The lower the cracking factor, the better the fatigue cracking performance. Results of this test tended to contradict results observed from R-SCB and OT tests. Mixtures with Konza RAP tended to show higher fatigue cracking resistance than the other mixtures.



**Figure 4.12: Cracking Factors Comparison for RAP Percentages**

#### 4.5 Simplified Viscoelastic Continuum Damage Test Results

The S-VECD test was performed in order to evaluate fatigue performance of HMA mixtures. SGC-compacted samples were cored and cut in order to produce samples according to AASHTO TP 105-13 (2013). Triplicate samples were then subjected to the pull-pull or direct tension fatigue test. Initial output was the number of fatigue cycles before failure of the specimen, as presented in Table 4.5. S-VECD results showed that fatigue cracking resistance of mixtures with RAP decreased with increasing RAP for all RAP sources, including the Konza RAP.

**Table 4.5: S-VECD Test Results**

RAP Source	RAP Content (%)	Air Void (%)	No. of Fatigue Cycles	COV (%)
Shilling	20	7.0	7,567	14.2
	30	6.7	3,619	5.3
	40	7.0	1,771	5.2
Konza	20	7.1	6,788	4.6
	30	7.3	4,705	14.8
	40	7.4	960	7.9
US-73	20	7.1	13,705	4.4
	30	6.6	11,281	3.0
	40	7.0	5,598	7.4

#### 4.5.1 Damage Characteristic Curve

A damage characteristic curve using test results from the S-VECD test for an HMA mixture is generally developed in order to study mixture resistance to damage. Mixtures are then compared based on parameter values. In this study, the following power model was investigated in order to compare fatigue performance of various mixtures:

$$C = 1 - yS^z \quad \text{Equation 4.1}$$

Where:

S is cumulative damage to attain first fatigue crack line,

C is pseudo secant modulus at the time of crack initiation, and

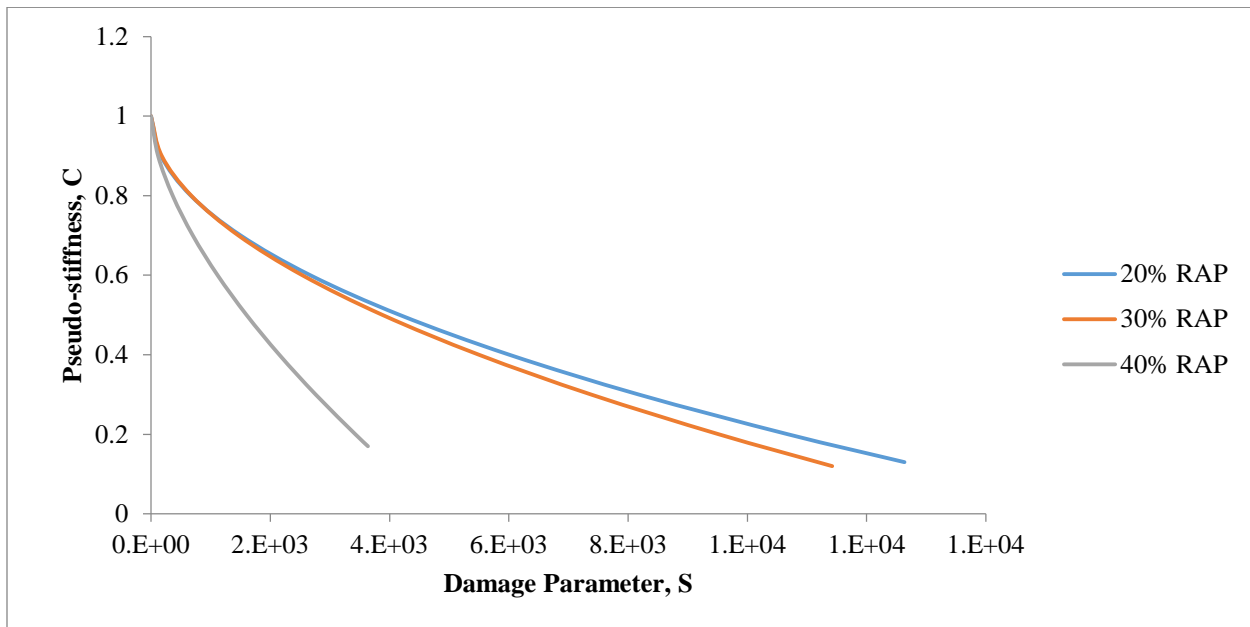
y, z are the fitting coefficients.

For a given normalized stiffness (C), a high damage parameter (S) value is indicative of increased resistance to damage (AASHTO TP 107-14, 2014). Results of three replicate tests for each mixture were used to fit the power model (Equation 3.8) of fatigue characteristics using the software Alpha-Fatigue in order to produce damage characteristic curves. Table 4.6 lists the fitting coefficients; y and z and pseudo strains at the failure are also listed.

**Table 4.6: S-VECD Calibration Equation Coefficient Values**

RAP Source	RAP Content (%)	y	z	Pseudo Strain at Failure ( $\mu\epsilon$ )
Shilling	20	7.74E-03	5.00E-01	0.12
	30	6.58E-03	5.24E-01	0.13
	40	5.23E-03	6.18E-01	0.17
Konza	20	8.72E-03	4.19E-01	0.18
	30	1.12E-02	4.76E-01	0.20
	40	9.63E-03	5.22E-01	0.22
US-73	20	5.85E-03	4.89E-01	0.10
	30	5.19E-03	4.81E-01	0.12
	40	5.47E-03	4.78E-01	0.15

Damage characteristic curves were developed using pseudo-stiffness values at failure for the range of 1 to the end value (Xie et al., 2015). Figure 4.13 illustrates damage characteristic curves for mixtures containing Shilling RAP.

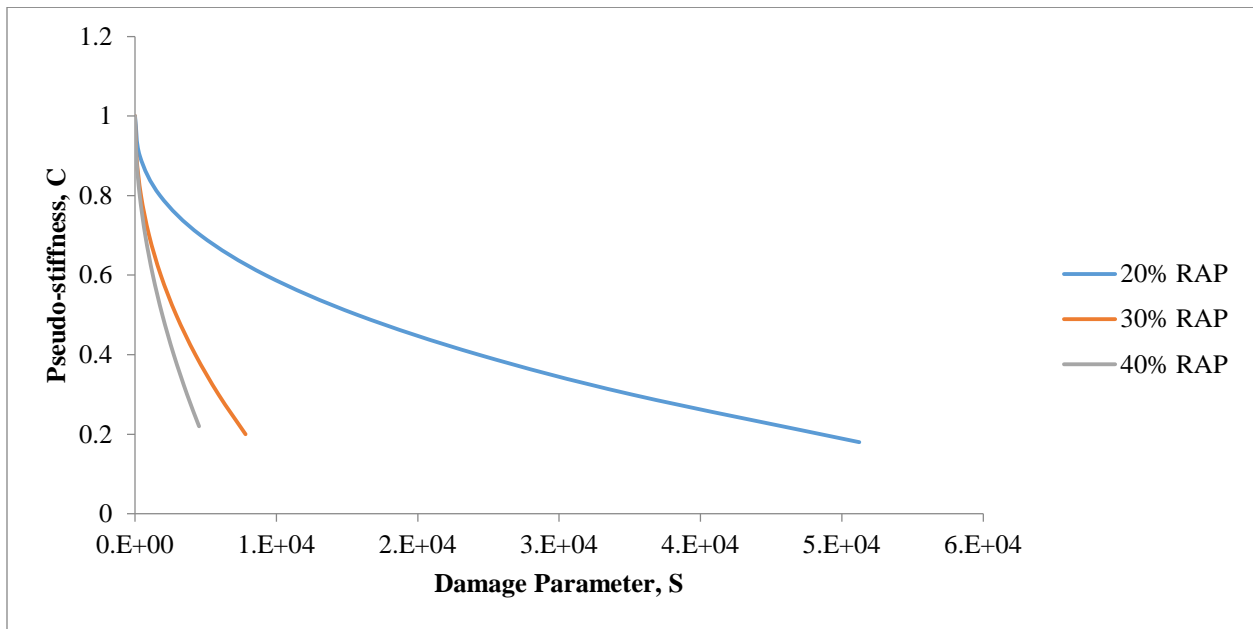


**Figure 4.13: Stiffness versus Damage Curves (Shilling RAP)**

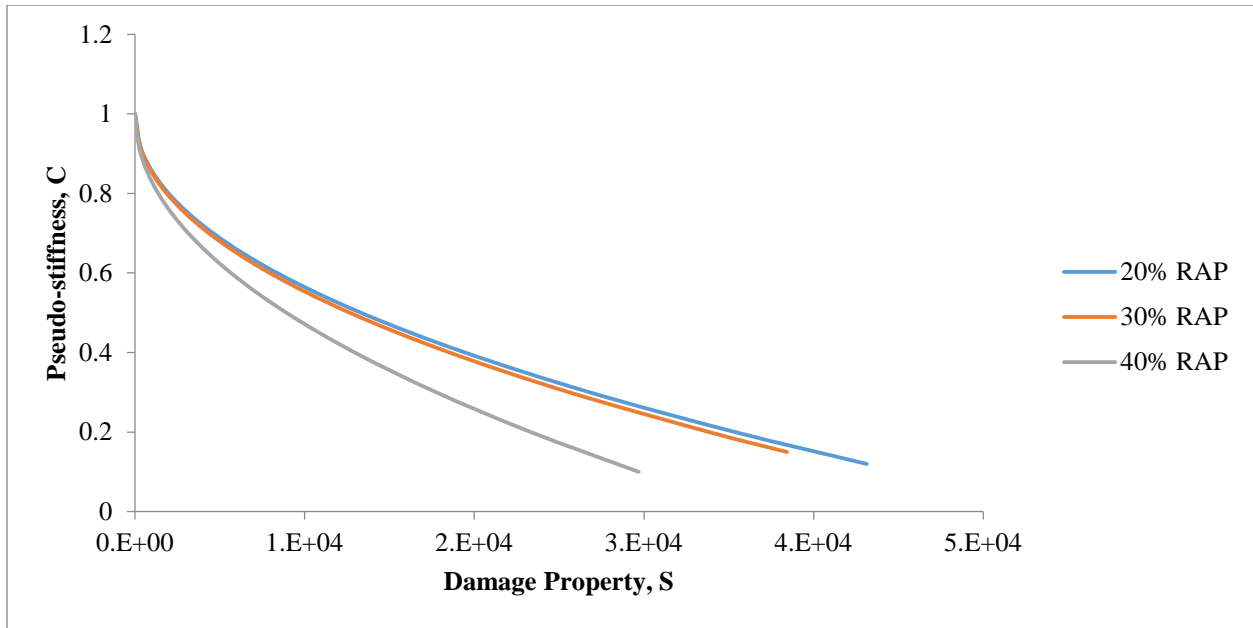
Figure 4.13 demonstrates that mixtures containing 20% RAP showed damage value at any given normalized value of pseudo strain as compared to mixtures containing 30% and 40% RAP. However, the mixture with 30% RAP showed a higher damage parameter value than the

mixture containing 40% RAP. Although mixtures with 20% and 30% RAP displayed almost similar fatigue characteristics, the mixture with 40% RAP failed abruptly with a large difference in damage compared to the other mixtures. According to Table 4.5, Shilling RAP samples with low resistance to fatigue cracking had high values of end stiffness, indicating that stiff samples are less satisfactory for fatigue performance.

Figure 4.14 illustrates fatigue characteristic curves for mixtures containing RAPs from the Konza Company. The mixture containing 20% RAP showed a substantially higher value of the damage parameter than the other two mixtures. Mixtures with 30% and 40% RAP also demonstrated that increased RAP content decreased fatigue performance of the mixture. However, mixtures with US-73 RAP showed consistent fatigue behavior, as illustrated in Figure 4.15. Mixture performance did not change substantially in terms of damage parameter values for 20% and 30% RAP; however, the mixture with 40% RAP showed different damage parameter values.



**Figure 4.14: Stiffness versus Damage Curves (Konza RAP)**



**Figure 4.15: Stiffness versus Damage Curves (US-73 RAP)**

#### 4.5.2 Fatigue Life Prediction

Damage characteristic curves are essential for evaluation of damage resistance and resistance to fatigue cracking. In order to interpret characteristics for real-time constructed pavement, life prediction of HMA mixture is necessary. Using the same software, fatigue life of the mixture can be determined using Equation 4.1 for known strain level. The test was performed for three replicates. The first replicate was subjected to 300 microstrains ( $\mu\epsilon$ ), and other two were decided according to the AASHTO guideline but never exceeding 450  $\mu\epsilon$ .

$$N_f = K_1 \left( \frac{1}{\epsilon_t} \right)^{K_2} (|E^*|)^{K_3} \quad \text{Equation 4.2}$$

Where:

$N_f$  is number of wheel passes,

$\epsilon_t$  is strain level ( $\mu\epsilon$ ),

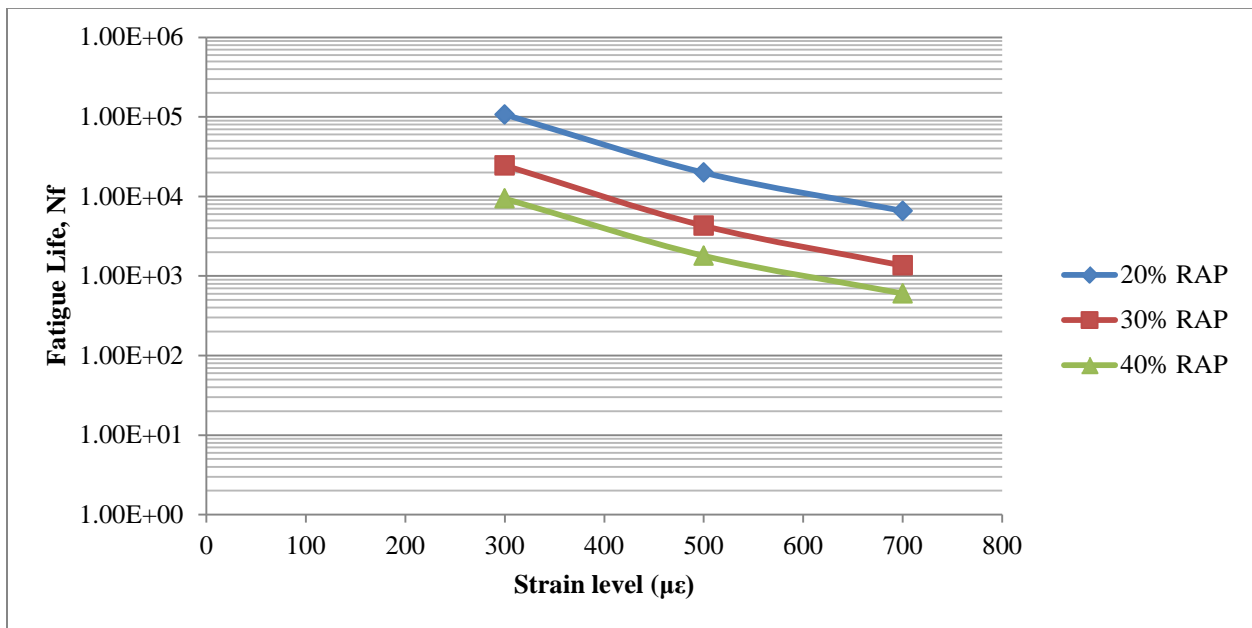
$|E^*|$  is dynamic modulus of the mixture at any given temperature and 10 Hz load frequency (MPa), and

$K_1, K_2, K_3$  are fitting coefficients.

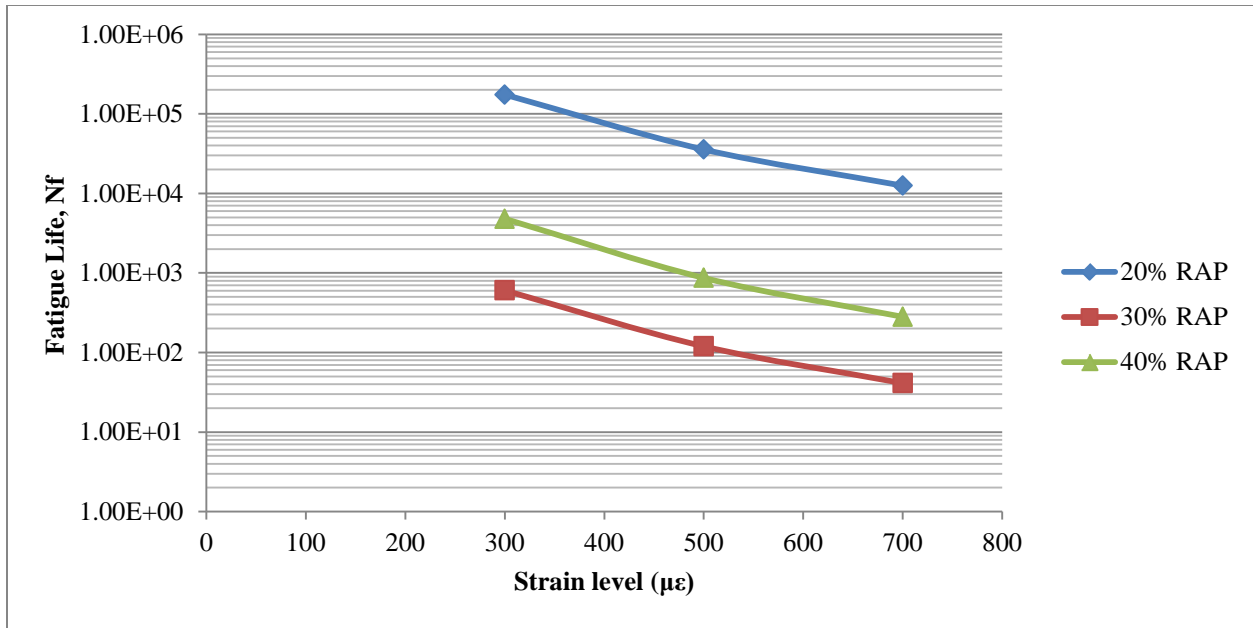
Equation 4.1 was calibrated using the least-squared distance method from data of three replicates. Using coefficient values, certain dynamic modulus, and associated strain level, remaining fatigue life of the mixture was predicted in terms of number of wheel passes.

In Figure 4.16, remaining fatigue life versus strain level is plotted for mixtures with Shilling RAP. Fatigue life is plotted on the log axis. At any strain level, the mixture with 20% RAP content showed highest fatigue life, but fatigue life decreased as the proportion of RAP increased in the mixture. With the increase in strain level, for the same mixture, fatigue life decreased for all three mixtures.

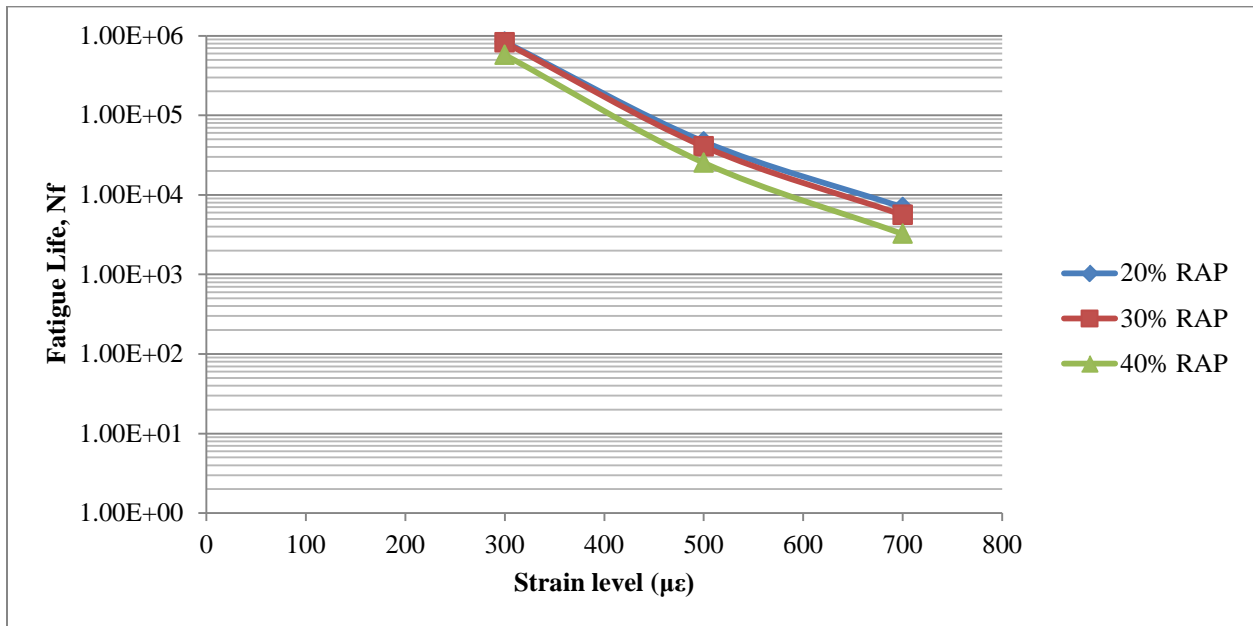
For mixtures containing Konza RAP and US-73 RAP, the hypothesis that mixtures with increasing amounts of RAP reduces fatigue life was verified. Fatigue life prediction curves are presented in Figures 4.17 and 4.18.



**Figure 4.16: Fatigue Life Prediction Curves (Shilling RAP)**



**Figure 4.17: Fatigue Life Prediction Curves (Konza RAP)**



**Figure 4.18: Fatigue Life Prediction Curves (US-73 RAP)**

#### 4.6 Statistical Analysis

In this study, statistical analysis was performed using the software package Statistical Analysis System (SAS Institute Inc., 2011). The objective of this study was to determine the effect of RAP sources and high RAP contents on cracking performance of Superpave mixtures.



Statistical analysis of test results was conducted in order to ascertain the significance of these factors.

#### 4.6.1 Fitness of the Cracking Test

Results of all cracking tests were analyzed using a two-way, full-factorial model of analysis of variance (ANOVA) in order to determine if the cracking test was sufficient to identify variation caused by two factors: RAP source and RAP content. The objective was to verify that the model used to analyze the results fit the outcomes of the cracking test. Therefore, the model p-value and F-value found by SAS were used at 95% confidence level. Table 4.7 lists p-values for various test results.

**Table 4.7: p-value and F-statistic Values for Cracking Tests**

Tests	p-value	F-Statistic
Semicircular Bending	<0.0001	11.28
Texas Overlay	<0.0001	16.28
Dynamic Modulus	0.0325	2.82
Viscoelastic Continuum Damage	<0.0001	196.05

At 95% level of confidence, p-values lower than 0.05 were considered to be significant. For all tests, p-values presented in Table 4.7 were lower than the level of significance, indicating that the model was accurate for determining variations caused by various levels of the factor RAP source (Shilling, Konza, and US-73) as well as the factor RAP content (20, 30, and 40%).

F-statistics values obtained during ANOVA were compared to the F-critical value found from the right-skewed F-distribution. For degrees of freedom of the model as 8 (3 x 3 -1) and degrees of freedom of the error associated with the model as 18 {3x3(3-1)},  $F_{critical}$  at 5% significance level was found to be 2.51. When the F-value of the model for each test as presented in Table 4.7 was compared to this  $F_{critical}$ , all test results captured variations caused by the two factors in this study.

#### 4.6.2 Significance of Factors

Table 4.8 lists p-values for the ANOVA performed for a full-factored model that used all levels of two factors, RAP source and RAP content, as well as factor interactions. The hypothesis that average results obtained from a certain test have no relationship to the factor(s) was tested in order to evaluate the significance of a factor. If the hypothesis was rejected, the factor contributes to the variation of mean results of the test at 5% level of significance. Results in Table 4.8 indicate that, for the SCB test, RAP source and RAP content significantly affect the FE outcome at various levels of the factor(s). However, the interaction between factors is not significant.

**Table 4.8: Significance of Factors**

Tests	p-value of RAP Source	p-value of RAP Content	p-value of Interaction
Semi-Circular Bending	<0.0001	0.0102	0.1655
Texas Overlay	<0.0001	0.0004	<0.0001
Dynamic Modulus	0.012	0.9816	0.0595
Viscoelastic Continuum Damage	<0.0001	<0.0001	<0.0001

For the OT and S-VECD tests, all three aspects were found to be significant at 5% level of significance. Various RAP sources uniquely affected OT cycles and S-VECD fatigue cycles of the same mixture. RAP content significantly contributed to outcome variations of these two tests. In addition, the conclusion was made that RAP source and RAP content together significantly contribute to OT cycles and S-VECD fatigue cycle numbers. This observation was justified by the drastic behavior of the mixture containing 40% Konza RAP. For the dynamic modulus test, analysis results were somewhat inconclusive. Although the RAP source was significant, the RAP content was not significant, thereby defying the traditional belief that increasing RAP content increases mixture stiffness. However, the high COV associated with the test results may have contributed to this outcome. Although a low cracking factor ( $|E^*| \sin\phi$ ), where  $|E^*|$  is the

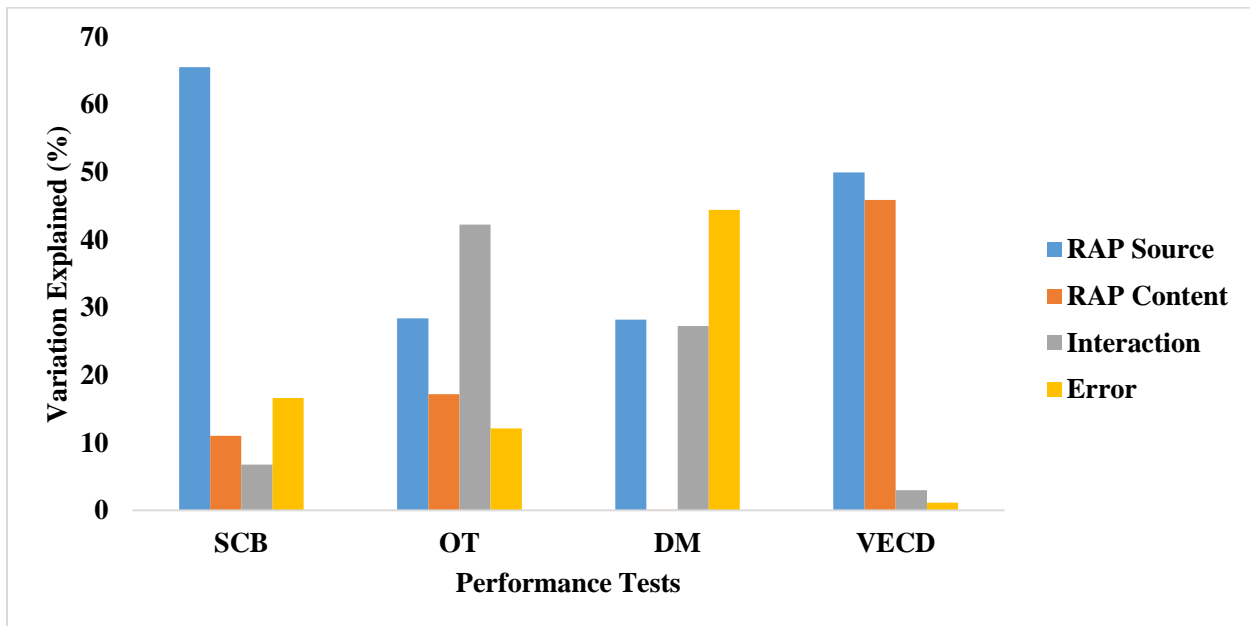
dynamic modulus and  $\phi$  is the phase angle, has been used as an indicator of improved fatigue performance, results in this study did not indicate the high potential of using the dynamic modulus test as a cracking evaluation test.

#### 4.6.3 Sources of Variation

Statistical analysis results obtained in this study showed that the tests were sufficient to varying degrees in capturing variation caused by different factor levels and interactions. Results summarized in Table 4.9 indicate the contribution of each factor. Figure 4.19 shows these results graphically.

**Table 4.9: Sources of Variability**

Test	Contribution in Variation (%)			
	RAP Source	RAP Content	Interaction	Error
Semi-Circular Bending	66	11	7	17
Texas Overlay	28	17	42	12
Dynamic Modulus	28	0	27	44
Viscoelastic Continuum Damage	50	46	3	1



**Figure 4.19: Distribution of Variation Among Sources**

For the SCB test, the largest source of variation in the test results occurred from the RAP source. RAP content was also significant, but the amount of resulting variation was small compared to RAP source. For the OT test, the primary source of variation was interaction, a combined effect of both factors. The test captured differentiation in the number of OT cycles caused by varying RAP content for individual RAP sources. Test results associated the dynamic modulus test with high error. The combined effect of RAP source and RAP content played a role in determining dynamic modulus values. The S-VECD test consistently predicted results for individual effect of RAP source and RAP content on fatigue cycles as well as evaluation of the combined effect of factors. Error associated with the S-VECD test was also minimal.

#### 4.6.4 Confidence Interval Analysis

Confidence interval analysis was conducted in order to thoroughly study factors. For the SCB test, interaction effect was not significant, so confidence interval prediction was performed for both factors individually. The OT and S-VECD tests demonstrated significant interaction effects, therefore, prediction based on each individual factor was not justified, and their combined effect was studied for prediction purposes. In Figures 4.20 and 4.21, predicted ranges of FE for each RAP source and RAP content at 95% confidence level are presented.

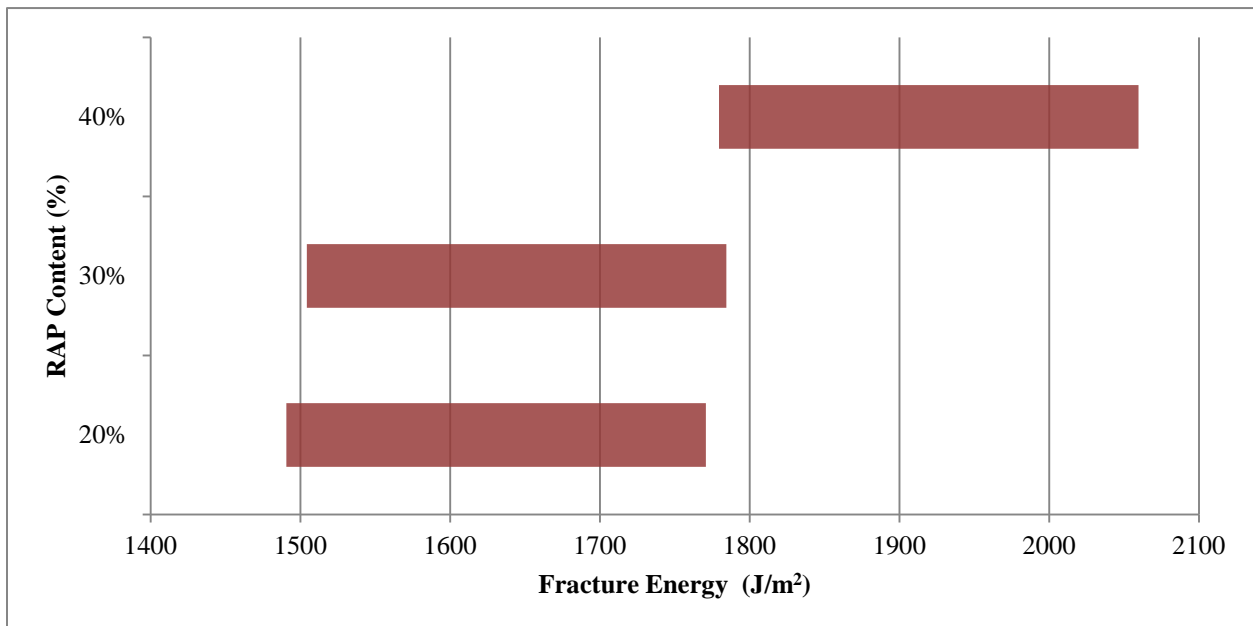
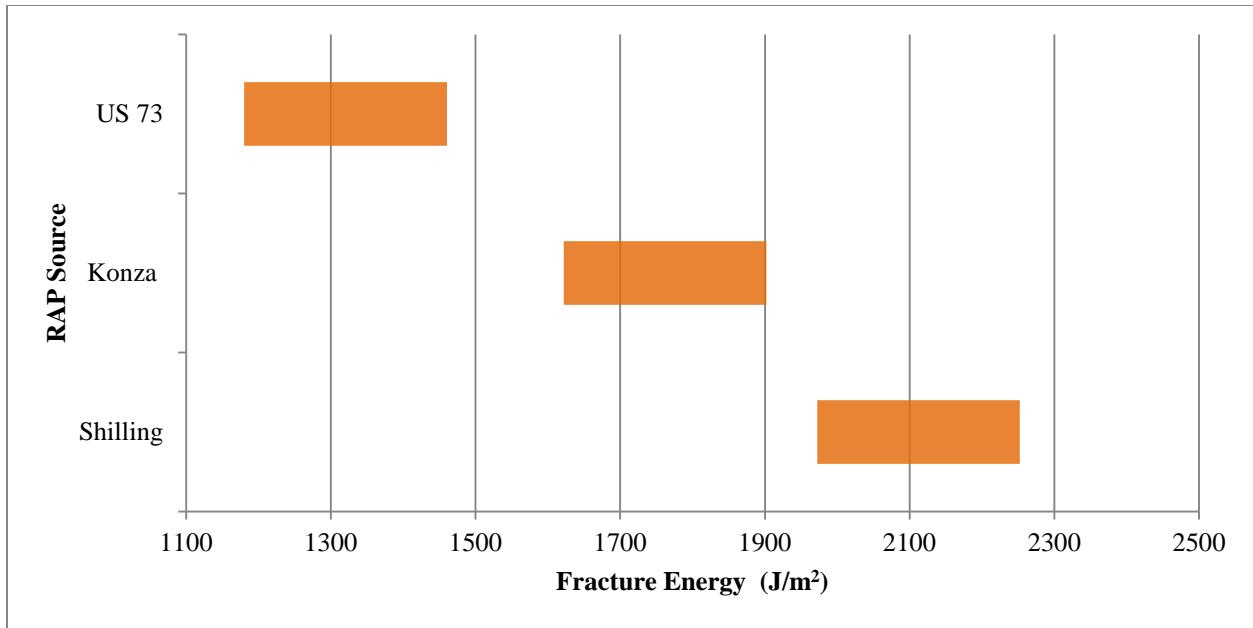


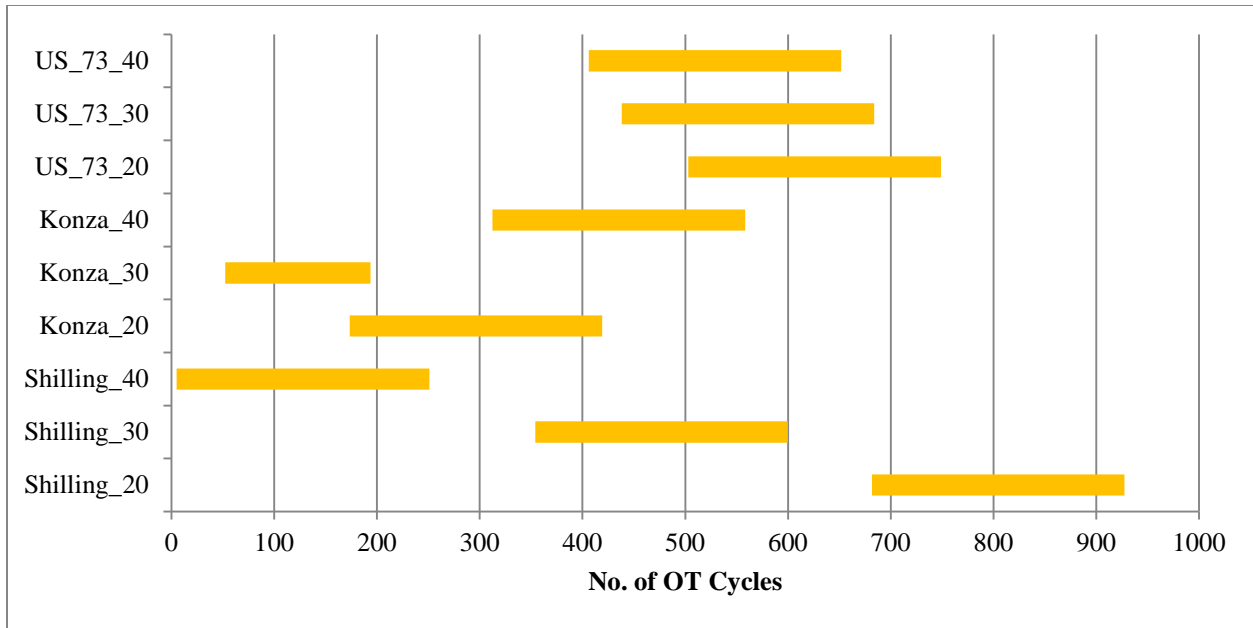
Figure 4.20: 95% Confidence Interval of RAP Content in SCB Test Results



**Figure 4.21: 95% Confidence Interval of RAP Sources in SCB Test Results**

Confidence intervals for varying RAP content showed significant overlaps, but it was discrete for the factor RAP source. Calculated confidence intervals are suggestive of expected FE values if different quantities of RAPs are added to the mixtures.

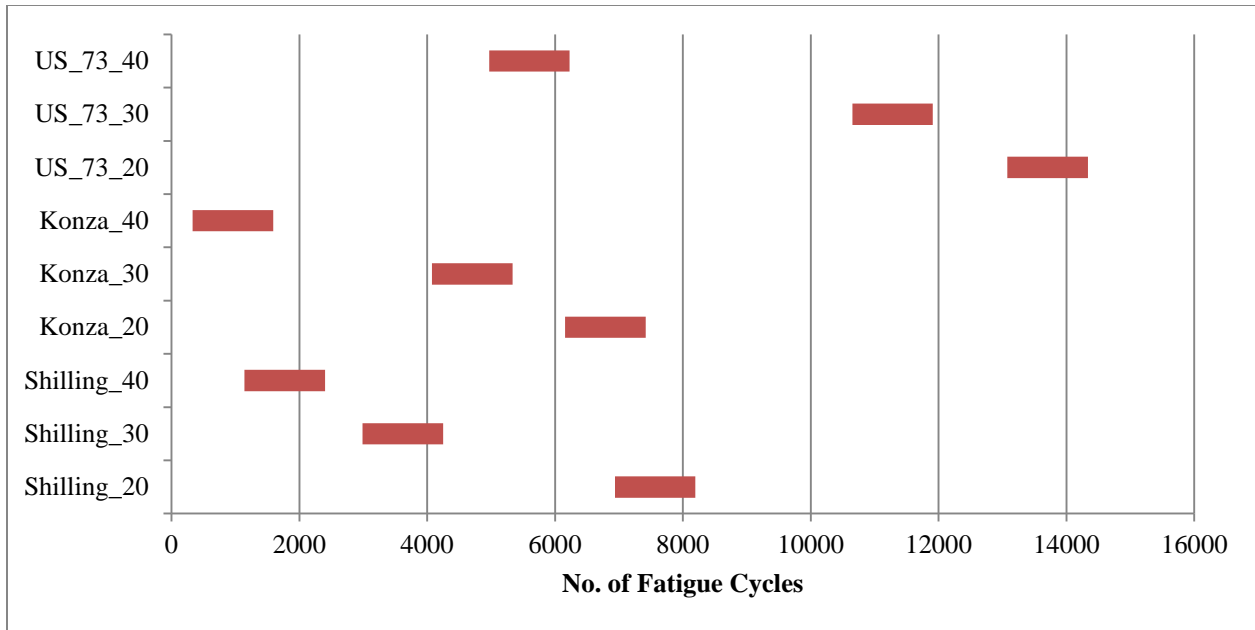
Figure 4.20 represents the range of expected FE values for a particular source of RAP. From Figure 4.20 and 4.21, the conclusion can be made that mixtures containing 20% to 30% RAP from US-73 RAP performed the best. In Figure 4.22 confidence intervals for combined effects for OT test results are presented. The mixture with 20% Shilling RAP had the highest number of OT cycles, but the US-73 mixture behaved consistently for all three RAP contents. At 95% confidence level, US-73 mixtures with RAP content as high as 40% demonstrated satisfactory performance. Interaction effect was more justified by performance of the mixture with 40% Konza RAP. The mixture performed significantly better than the other two mixtures with varying RAP contents from the same source.



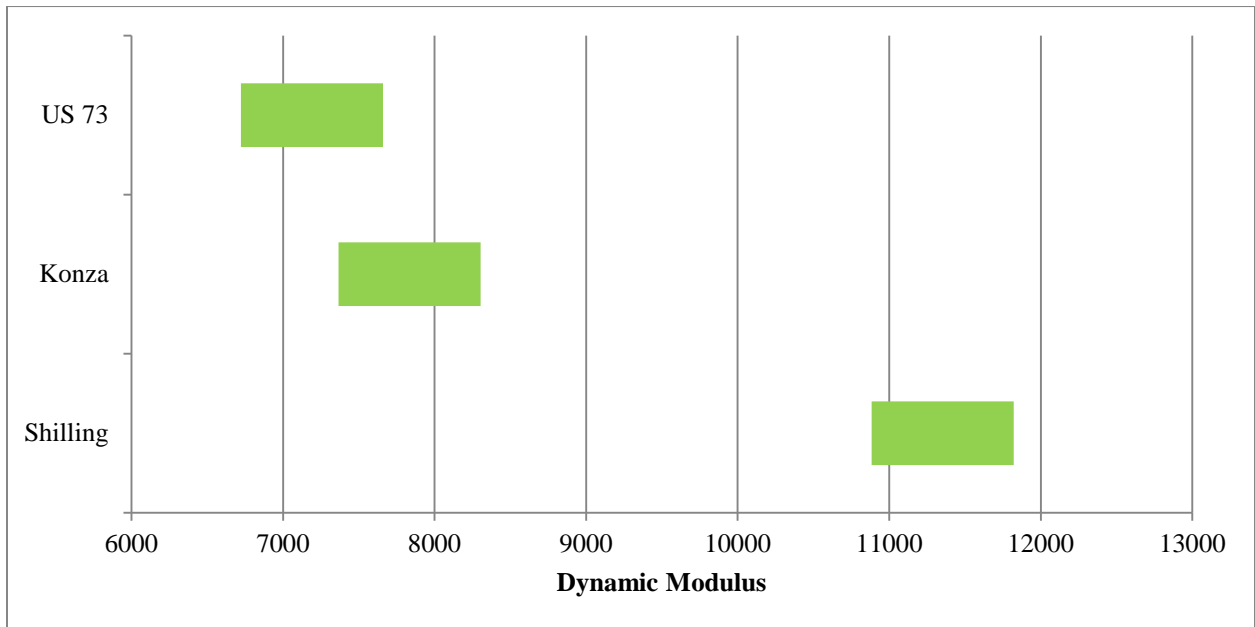
**Figure 4.22: 95% Confidence Interval of OT Test Results**

Confidence interval analysis results for the S-VECD test are presented in Figure 4.23, which shows the range of expected numbers of fatigue cycles for three RAP contents for each source of RAP. Mixtures containing 20% to 30% RAP from US-73 had the highest predicted fatigue cycles compared to the other mixtures. The mixture with 40% US-73 RAP was also satisfactory. Mixtures with 20% Shilling and Konza RAP demonstrated high expected values as well.

For dynamic modulus test results, the factor, RAP content, and combined effect of factors were not found to be significant; therefore, prediction was based only on RAP source. Confidence intervals are presented in Figure 4.24 for dynamic modulus test results. In general, this figure predicts expected ranges of dynamic modulus values from mixtures containing three RAP sources regardless of content.



**Figure 4.23: 95% Confidence Interval of S-VECD Test Results**



**Figure 4.24: 95% Confidence Interval of Dynamic Modulus Test Results**

#### 4.6.5 Comparison of Cracking Tests

Mean results obtained from the S-SCB test, the OT test, the dynamic modulus test, and the S-VECD test were compared using the Tukey's pairwise comparison method in order to find best performing combination of RAP source and RAP content and most promising cracking evaluation test.

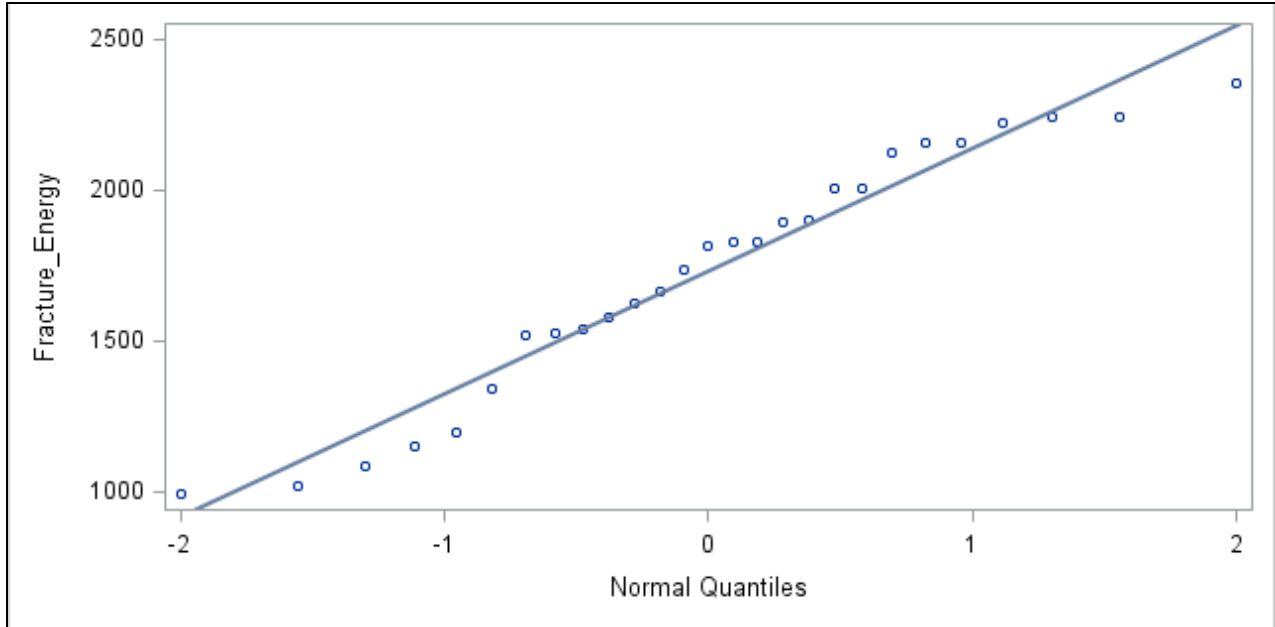
Data used in the analysis was first checked for homogeneity of variance, the null hypothesis was checked by the model likelihood ratio test, and the homogenous variance assumption was found to be true for all datasets. In the next step, data was checked for normality. Table 4.10 shows normality test results of the S-SCB test data. The dataset was normally distributed, as justified by the normal probability plot for the dataset illustrated in Figure 4.25.

**Table 4.10: Normality Test Results of SCB Test Data**

<b>Normality Test</b>	<b>p-value</b>
Shapiro-Wilk	0.954
Kolmogorov-Smirnov	0.097
Anderson-Darling	0.406

Tukey's pairwise comparison method is considered to be the most powerful tool for controlling Type-I error rate (Kuehl, 2000). Least squared means for S-SCB test results were grouped using this method. Depending on the grouping, mean results of the tests were compared in order to identify the better performing tests. The basic assumption was that the more capable the test was to capture statistically significant differences between the mixtures, the more the test was suitable for determining cracking potential. In other words, the test was capable of accurately evaluating effects of factors. Pairwise comparison of the means for S-SCB test results are shown in Table 4.11. Normal probability plot for SCB test results are presented in Figure 4.25.





**Figure 4.25: Normal Probability Plot of FE**

**Table 4.11: Tukey Pairwise Comparison of SCB Test Results**

Asphalt Content (%)	4.3	4.7	4.8	4.1	4.4	4.5	4.3	4.9	4.8
Shilling_40	A								
Shilling_20		A							
Shilling_30			AB						
Konza_40				AB					
Konza_30					ABC				
US_73_40						BC			
Konza_20							BC		
US_73_20								C	
US_73_30									C

Assumptions for this test included independence of observations and homogeneity of variance. Comparison of significance was done based on adjusted p-value of each pair: if the p-value was lower than the level of significance ( $\alpha = 0.05$ ), two compared mixtures had statistically significant differences. In Table 4.11, mixtures identifications are coded by source name and

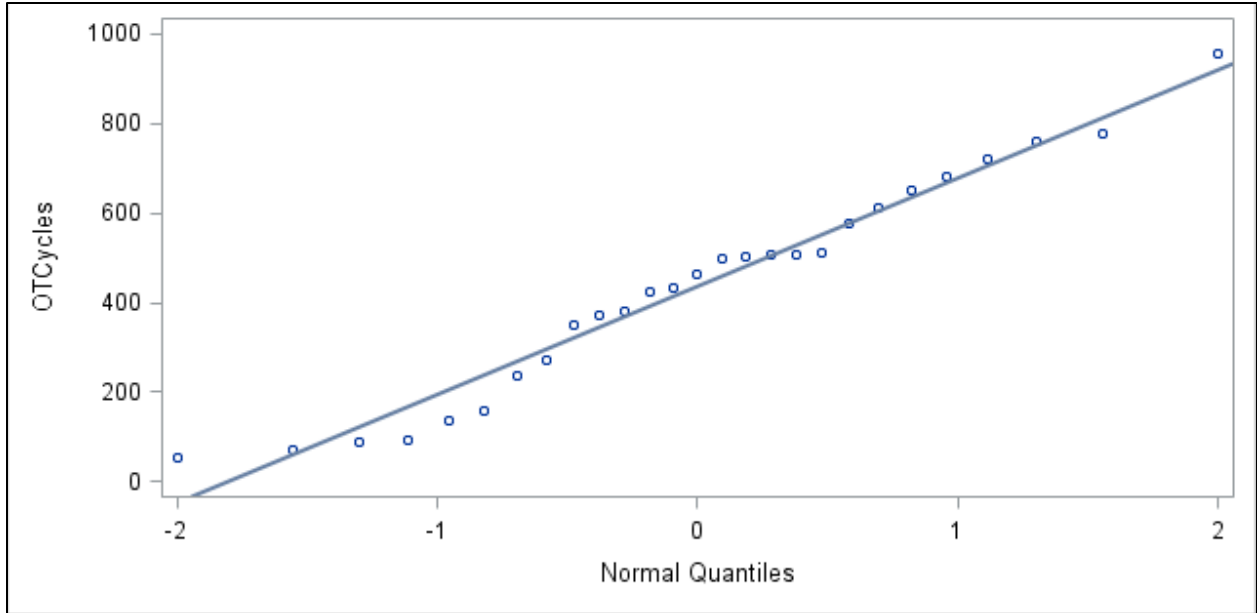
RAP content (%). Asphalt content of the mixture was reported in the corresponding columns. As shown in this table, mixtures bordered with the same colored box demonstrated no significant difference in their mean FE values. For example, the mixtures with 40% Shilling RAP and 20% Shilling RAP are different, but the difference was not statistically significant at 95% confidence level. Several overlapping did not allow formation of a full block. The best performing mixture contained 30% US-73 RAP at a binder content of 4.8%. Overall, the test was able to determine six blocks of similar means which showed significant difference among them.

OT test results were also checked for normality using the same methods previously mentioned. Table 4.12 and Figure 4.26 show p-values and normal probability plot, respectively. The dataset was found to be normally distributed.

**Table 4.12: Normality Test Results of OT Test**

<b>Normality Test</b>	<b>p-value</b>
Shapiro-Wilk	0.9649
Kolmogorov-Smirnov	0.0965
Anderson-Darling	0.3020

Tukey pairwise comparison results are shown in Table 4.13. The extent of overlapping was more concentrated according to the results of this test. The mixture with 20% Shilling RAP performed the best in this test and 30% Konza RAP mixture performed the worst. The test captured six blocks of similar least squared means.



**Figure 4.26: Normal Probability Plot of OT Cycles**

**Table 4.13: Tukey Pairwise Comparison of OT Test Results**

Asphalt Content (%)	4.7	4.9	4.8	4.5	4.8	4.1	4.3	4.3	4.4
Shilling_20	A								
US_73_20		AB							
US_73_30			ABC						
US_73_40				ABC					
Shilling_30					BC				
Konza_40						BC			
Konza_20							DC		
Shilling_40								D	
Konza_30									D

For statistical analysis of dynamic modulus test results, dynamic modulus values of 10 Hz and 21 °C were used. As mentioned, the dynamic modulus test was not suitable for evaluation of the effects of RAP source and RAP content on cracking performance. However, results were re-analyzed and compared to other test results. Results from the dynamic modulus

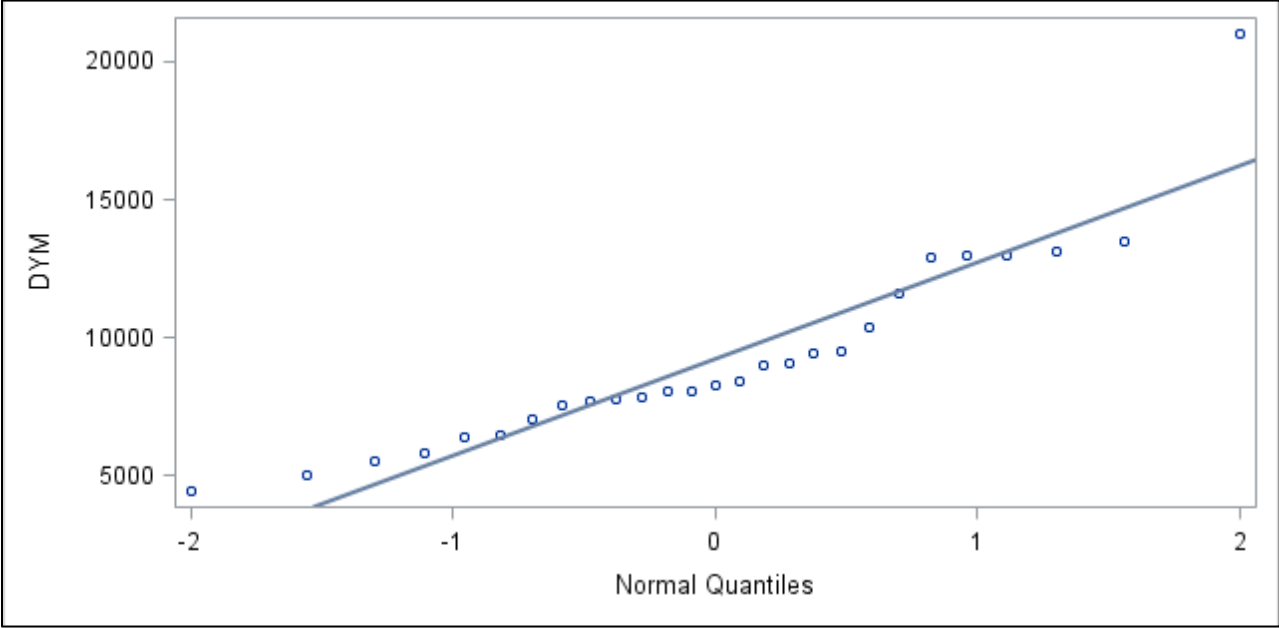
and S-VECD tests were tested for normality. Test outcomes are presented in Tables 4.14 and 4.15 and Figures 4.27 and 4.28. Both datasets were found to be normally distributed.

**Table 4.14: Normality Test Results of Dynamic Modulus Test**

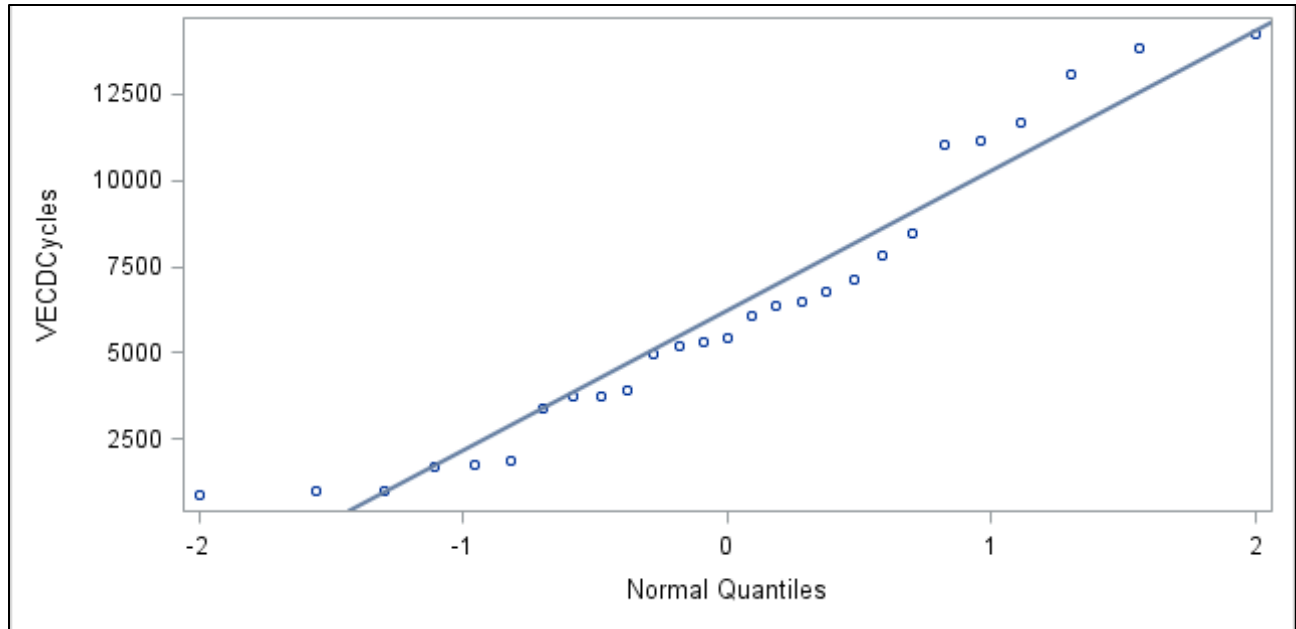
Normality Test	p-value
Shapiro-Wilk	0.88
Kolmogorov-Smirnov	0.18
Anderson-Darling	0.91

**Table 4.15: Normality Test Results of S-VECD Test**

Normality Test	p-value
Shapiro-Wilk	0.93
Kolmogorov-Smirnov	0.12
Anderson-Darling	0.59



**Figure 4.27: Normal Probability Plot of Dynamic Modulus Results**



**Figure 4.28: Normal Probability Plot of Fatigue Cycles Of S-VECD Test**

In Tables 4.16 and 4.17, Tukey pairwise comparison groups are presented for the means of dynamic modulus test results and S-VECD test results, respectively. Mixtures enclosed in boxes of identical color did not show any significant difference in their means. According to dynamic modulus test results, the test identified only three blocks, and the overlapping zone was much larger than other tests, indicating less identification of differences.

For S-VECD test results, three individual blocks were identified. Overlapping blocks were smaller in range, and eight groups overall were formed. According to this test, the mixture containing 20% RAP from US-73 showed optimum fatigue performance.

Based on analysis, the test with the most potential for assessing cracking potential of a mix with RAP is the S-VECD test, followed by the OT and S-SCB tests for capturing desired variability in cracking performance. The dynamic modulus test was not significant for evaluating the effect of RAP source and RAP content on cracking performance of HMA mixtures.

**Table 4.16: Tukey Pairwise Comparison of Dynamic Modulus Test Results**

Asphalt Content (%)	4.3	4.8	4.3	4.9	4.7	4.8	4.1	4.4	4.5
Shilling_40	A								
Shilling_30		AB							
Konza_20			AB						
US_73_20				AB					
Shilling_20					AB				
US_73_30						AB			
Konza_40							AB		
Konza_30								AB	
US_73_40									B

**Table 4.17: Tukey Pairwise Comparison of S-VECD Test Results**

Asphalt Content (%)	4.9	4.8	4.7	4.3	4.5	4.4	4.8	4.3	4.1
US_73_20	A								
US_73_30		B							
Shilling_20			C						
Konza_20				CD					
US_73_40					DE				
Konza_30						EF			
Shilling_30							F		
Shilling_40								G	
Konza_40									G

# Chapter 5: Conclusions and Recommendations

## 5.1 Conclusions

The objective of this research was to evaluate cracking resistance of Superpave mixtures with high RAP contents and various RAP sources. A total of nine mixtures were tested in four cracking test setups as well as in the moisture susceptibility test. The following conclusions were drawn based on analysis results:

1. Modified Lottman test results indicated that Tensile Strength Ratio (TSR) decreased with increasing RAP content, but the values were greater than 80%, as required by KDOT, for six mixtures with RAP from Shilling and US-73 sources. Three mixtures containing 20, 30, and 40% Konza RAP failed to achieve minimum TSR in this test.
2. The S-SCB test compared mixtures based on Fracture Energy (FE) and stiffness parameters. With increasing RAP content, rate of energy release and stiffness parameters increased. Mixtures with RAP from US-73 performed better than mixtures with two other sources of RAP. Differences in FE values for various RAP content and source were not found to be statistically significant because of dominant interaction effect between the RAP source and the RAP content at the 95% confidence interval. Overall, this test was able to identify differences in performance of various mixtures.
3. The R-SCB test showed anomalous results for the evaluation of fatigue performance. For six mixtures from the first and third sources of RAP, performance decreased with increasing RAP content. For the second source of RAP, however, the mixture containing 40% RAP performed the best.
4. The OT test provided straightforward evaluation of mixtures; mixtures passing a minimum of 300 OT cycles were considered to satisfactorily resist reflective cracking, but mixtures with 40% RAP from the first and second RAP sources failed in this category. Statistical analysis results

identified that the OT test is capable of assessing cracking performance of HMA mixtures and identifying interaction effect of RAP content and RAP source. Therefore, cracking prediction was made for each individual mixture using 95% confidence interval.

5. Typical trends of decreasing dynamic modulus values with increasing temperature and decreasing dynamic modulus values for low frequencies were observed in the dynamic modulus test results. However, the test results proved that the dynamic modulus test did not accurately evaluate cracking performance. High variability associated with the test may have contributed to this statistical conclusion.
6. The S-VECD test compared mixtures based on the number of standard fatigue cycles. Fatigue performance decreased with increasing RAP content. Mixtures with RAP from US-73 performed better than the other two sources. Statistically, dominance of individual RAP content or RAP quality was not found. Damage characteristic curves were produced for all mixtures, and performance was assessed based on damage parameter values, the higher the better. Remaining fatigue life of the mixtures was also simulated for corresponding stress and strain levels. The S-VECD test identified the highest number of significant differences among test results.
7. Mixtures with total binder content of 4.5 to 4.9% performed satisfactorily. Mixtures with 40% RAP were satisfactory for two sources, but the other source displayed anomalous results. However, for up to 30% RAP content, mixtures performed well.

## **5.2 Recommendations**

RAP properties included in this study were binder content and gradation of RAP sources. Study results show that the stiffening effect of RAP affects outputs of various test results; therefore, further study should include these RAP properties. For all tests, RAP source was found to be a significant, sometimes dominant, source of variance. If RAP sources individually



perform, one prescription of quantity is not possible to be given at least for higher quantity of RAPs. Characterization of RAP quality is essential for satisfactory performance. As a future research scope, characterization of RAP binder at the elemental level may help assess RAP better to provide more useful guidance to designers.

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## Appendix: Cracking Test Results

**Table A.1: Semi-Circular Bending Test: Fracture Energy Results**

RAP Source	RAP Content (%)	Area Under (P-u) Curve (J)	Ligament Area (m <sup>2</sup> )	Fracture Energy (J/m <sup>2</sup> )	Average Energy (J/m <sup>2</sup> )	Std. Dev	COV (%)
Shilling	20	5.8858	0.00725	2242.9	2098.7	180.9	8.6
		5.6612	0.00725	2157.4			
		4.9745	0.00725	1895.7			
	30	5.8892	0.00725	2244.2	2064.9	214.4	10.4
		5.5713	0.00725	2123.1			
		4.7956	0.00725	1827.5			
	40	5.2671	0.00725	2007.2	2173.8	175.4	8.1
		5.6613	0.00725	2157.4			
		6.1846	0.00725	2356.8			
Konza	20	4.0371	0.00725	1538.5	1632.2	98.5	6.0
		4.5525	0.00725	1734.9			
		4.2598	0.00725	1623.3			
	30	4.3656	0.00725	1663.6	1769.4	92.1	5.2
		4.8057	0.00725	1831.3			
		4.7584	0.00725	1813.3			
	40	5.8410	0.00725	2225.9	1884.0	350.0	18.6
		4.0054	0.00725	1526.4			
		4.9852	0.00725	1899.8			
US-73	20	2.5993	0.00725	990.5	1161.2	176.0	15.2
		3.5219	0.00725	1342.1			
		3.0206	0.00725	1151.1			
	30	2.5412	0.00725	968.4	1187.6	337.7	28.4
		2.6710	0.00725	1017.9			
		4.1367	0.00725	1576.4			
	40	5.2653	0.00725	2006.5	1612.2	377.1	23.4
		4.1332	0.00725	1575.1			
		3.2935	0.00725	1255.1			

**Table A.2: Semi-Circular Bending Stiffness Parameters**

RAP Source	RAP Content (%)	Load (KN)	Displacement (mm)	c(L/D)	Sample Thickness, t (mm)	Stiffness Modulus (MPa)	Average Modulus (MPa)	Std Dev.	COV (%)
Shilling.	20	2543.7	0.0019	0.8	50	21420.5	20491.1	1414.1	6.9
		2516.2	0.0019	0.8	50	21189.1			
		2004.3	0.0017	0.8	50	18863.7			
	30	3172.0	0.0015	0.8	50	33834.8	29439.9	8056.3	27.4
		3863.6	0.0018	0.8	50	34342.9			
		2769.5	0.0022	0.8	50	20141.9			
	40	4608.2	0.0012	0.8	50	61442.9	58594.2	5566.0	9.5
		3587.4	0.0011	0.8	50	52180.4			
		3885.0	0.0010	0.8	50	62159.2			
Konza	20	2354.5	0.0014	0.8	50	26908.2	26804.4	439.9	1.6
		2632.2	0.0016	0.8	50	26321.8			
		2548.4	0.0015	0.8	50	27183.0			
	30	2467.4	0.0015	0.8	50	26318.7	27406.9	1249.5	4.6
		2713.1	0.0016	0.8	50	27130.5			
		2517.5	0.0014	0.8	50	28771.4			
	40	3135.7	0.0016	0.8	50	31357.3	33843.1	2247.0	6.6
		2456.4	0.0011	0.8	50	35730.0			
		2798.4	0.0013	0.8	50	34441.8			
US-73	20	1549.6	0.0019	0.8	50	13048.8	11124.3	2374.4	21.3
		1482.4	0.0028	0.8	50	8470.9			
		1481.7	0.0020	0.8	50	11853.2			
	30	1893.6	0.0015	0.8	50	20198.8	18313.4	1639.7	9.0
		1829.7	0.0017	0.8	50	17220.7			
		2080.6	0.0019	0.8	50	17520.6			
	40	2414.7	0.0020	0.8	50	19317.9	18466.2	819.6	4.4
		2320.9	0.0021	0.8	50	17683.0			
		2414.7	0.0021	0.8	50	18397.7			

**Table A.3: Texas Overlay No. of OT Cycles**

RAP Source	RAP Content (%)	Air Void (%)	Initial Peak Load (kN)	No. of OT Cycles	Average Duration (min)	Average No. of OT Cycles	Std. dev	COV (%)
Shilling	20	7.2	2.341	956	134	805	140	17.4
		7.3	2.418	778				
		7.0	2.296	680				
	30	7.3	2.799	348	80	477	117	24.5
		7.4	2.987	576				
		7.2	2.903	507				
	40	7.3	3.432	91	21	128	34	26.9
		7.1	3.219	134				
		7.0	3.186	159				
Konza	20	7.3	2.615	236	48	296	75	25.2
		7.1	2.769	380				
		7.0	2.599	273				
	30	7.2	1.917	54	12	71	16	22.7
		7.4	2.069	72				
		7.3	2.318	86				
	40	7.2	2.989	372	73	435	65	14.9
		7.0	3.098	432				
		7.1	3.185	502				
US-73	20	7.8	1.152	612	97.7	626	126	20.1
		6.5	1.773	508				
		7.4	1.612	758				
	30	6.5	0.848	498	93	561	140	24.9
		7.0	0.581	721				
		7.2	0.713	464				
	40	7.1	0.581	649	47.3	529	112	21.3
		6.9	0.495	512				
		7.4	0.395	426				

**Table A.4: Dynamic Modulus Test Results**

Sample ID	Temperature (°C)	Dynamic Modulus (MPa) at different Loading Frequencies (Hz)					
		25	10	5	1	0.5	0.1
20_I	4	16190	15059	14198	12413	11586	9589
20_II		15390	13883	13120	11146	10234	7581
20_III		14511	14822	13757	11210	10074	7481
Mean		15363.7	14588	13691.7	11589.7	10631.3	8217
Std. Dev		839.8	621.9	542	713.7	830.6	1189.2
COV (%)		5.5	4.3	4	6.2	7.8	14.5
20_I	21	11532	9502	7799	6033	5880	5928
20_II		9192	8052	7446	6224	5558	5108
20_III		11447	10384	7739	5324	5016	4997
Mean		10723.7	9312.7	7661.3	5860.3	5484.7	5344.3
Std. Dev		1327.1	1177.5	188.9	474.2	436.6	508.5
COV (%)		12.4	12.6	2.5	8.1	8	9.5
20_I	37	6749	5347	4578	3942	2312	1925
20_II		6331	5493	4992	3977	2674	1944
20_III		7149	5399	4667	3216	2746	1780
Mean		6743	5413	4745.7	3711.7	2577.3	1883
Std. Dev		409	74	217.9	429.6	232.6	89.7
COV (%)		6.1	1.4	4.6	11.6	9	4.8
30_I	4	13140	11868	10802	8410	7404	5238
30_II		12800	11478	10555	8556	7765	6054
30_III		10564	9097	8616	7307	7563	5816
Mean		12168	10814.3	9991	8091	7577.3	5702.7
Std. Dev		1399.5	1500	1197.2	682.9	180.9	419.6
COV (%)		11.5	13.9	12	8.4	2.4	7.4
30_I	21	9221	8407	7835	6744	5381	4698
30_II		10615	9464	8551	6663	5948	4476
30_III		8589	7840	7158	5661	5091	4013
Mean		9475	8570.3	7848	6356	5473.3	4395.7
Std. Dev		1036.6	824.2	696.6	603.2	435.9	349.5
COV (%)		10.9	9.6	8.9	9.5	8	8
30_I	37	6427	5673	4148	3542	2815	2172
30_II		6428	5126	4230	4438	2731	2276
30_III		6322	5411	4758	3373	2878	2810
Mean		6392.3	5403.3	4378.7	3784.3	2808	2419.3
Std. Dev		60.9	273.6	331.1	572.4	73.7	342.3
COV (%)		1	5.1	7.6	15.1	2.6	14.1

**Table A.4: Dynamic Modulus Test Results (Continued)**

40_I	4	9337	8286	8636	7049	6323	4611
40_II		8671	7889	7205	5741	5135	3811
40_III		10478	9557	8804	6993	6128	4246
Mean		9495.3	8577.3	8215	6594.3	5862	4222.7
Std. Dev		913.8	871.3	878.7	739.5	637.1	400.5
COV (%)		9.6	10.2	10.7	11.2	10.9	9.5
40_I	21	8192	5032	4463	3224	2558	1866
40_II		5602	4442	3893	2801	2424	1696
40_III		6703	5815	5115	3740	2273	1410
Mean		6832.3	5096.3	4490.3	3255	2418.3	1657.3
Std. Dev		1299.8	688.8	611.5	470.3	142.6	230.4
COV (%)		19	13.5	13.6	14.4	5.9	13.9
40_I	37	4284	3535	3024	2056	1732	1074
40_II		4561	3599	3093	2131	1822	1193
40_III		5559	4550	3880	2553	2185	1394
Mean		4801.3	3894.7	3332.3	2246.7	1913	1220.3
Std. Dev		670.6	568.4	475.5	267.9	239.8	161.7
COV (%)		14	14.6	14.3	11.9	12.5	13.3

**Table A.5: Phase Angle Values**

Sample ID	Temperature (°C)	Phase Angle (°) at Different Loading Frequency (Hz)					
		25 Hz	10 Hz	5 Hz	1 Hz	0.5 Hz	0.1 Hz
20_I	4	13.5	34.2	32.3	22.5	23.6	25.9
20_II		17.9	36.8	35.5	27.8	25.3	22.8
20_III		10.1	11.8	12.5	14.1	15.4	18.3
Mean		13.8	27.6	26.8	21.5	21.4	22.3
Std. Dev		3.9	13.8	12.5	6.9	5.3	3.8
COV (%)		28.2	49.9	46.6	32.1	24.7	17.0
20_I		21	21.3	39.4	38.4	29.2	30.1
20_II	25.8		29.8	35.9	32.5	33.5	28.9
20_III	17.3		18.0	19.1	21.9	22.8	25.3
Mean	21.5		29.1	31.1	27.8	28.8	28.6
Std. Dev	4.2		10.7	10.5	5.4	5.5	3.1
COV (%)	19.7		37.0	33.7	19.5	19.0	10.9
20_I	37		17.8	31.5	36.5	27.6	31.5
20_II		21.1	30.0	37.5	28.3	32.4	27.4
20_III		22.2	21.2	22.2	24.8	25.5	27.7
Mean		20.4	27.6	32.1	26.9	29.8	27.2
Std. Dev		2.3	5.6	8.5	1.9	3.7	0.6
COV (%)		11.3	20.2	26.6	6.9	12.5	2.4
30_I		4	11.6	11.8	12.5	14.6	15.5
30_II	18.9		16.7	15.6	15.5	12.3	11.3
30_III	7.4		9.6	7.6	8.7	9.4	11.5
Mean	12.6		12.7	11.9	12.9	12.4	13.6
Std. Dev	5.8		3.6	4.0	3.7	3.0	3.8
COV (%)	46.1		28.7	33.8	28.6	24.5	28.0
30_I	21		14.2	15.9	14.8	17.3	18.2
30_II		13.2	14.7	17.5	16.4	19.1	17.6
30_III		9.7	13.9	11.6	13.6	14.5	16.7
Mean		12.3	14.8	14.6	15.8	17.3	18.4
Std. Dev		2.4	1.0	2.9	1.9	2.4	2.2
COV (%)		19.3	6.7	20.1	12.1	14.2	12.1
30_I		37	10.6	14.9	16.7	19.3	20.2
30_II	11.3		15.8	12.6	13.7	17.2	23.8
30_III	11.4		13.5	14.5	16.6	17.5	19.9
Mean	11.1		14.7	14.6	16.5	18.3	22.3
Std. Dev	0.4		1.1	2.0	2.8	1.7	2.1
COV (%)	3.7		7.7	14.0	16.9	9.1	9.3



**Table A.5: Phase Angle Values (Continued)**

40_I	4	15.9	12.7	10.9	12.7	13.7	16.9
40_II		18.5	17.6	11.5	13.5	11.5	17.4
40_III		9.5	29.8	31.9	22.9	18.7	21.6
Mean		14.6	20.0	18.1	16.4	14.6	18.7
Std. Dev		4.6	8.8	11.9	5.7	3.7	2.6
COV (%)		31.5	43.8	66.0	34.8	25.4	13.9
40_I	21	28.2	18.4	19.7	22.8	28.3	27.3
40_II		25.9	21.4	18.4	23.6	27.5	26.8
40_III		31.5	34.8	33.5	23.9	25.0	27.1
Mean		28.5	24.9	23.9	23.4	26.9	27.0
Std. Dev		2.8	8.7	8.4	0.6	1.7	0.2
COV (%)		9.8	35.2	35.1	2.4	6.5	0.8
40_I	37	21.7	29.5	24.6	27.7	28.6	31.7
40_II		17.9	32.8	25.7	27.8	25.3	21.3
40_III		17.8	20.2	21.4	24.8	25.8	29.0
Mean		19.1	27.5	23.9	26.8	26.5	27.3
Std. Dev		2.2	6.6	2.2	1.7	1.8	5.4
COV (%)		11.6	23.9	9.4	6.5	6.6	19.8

**Table A.6: S-VECD Cycles**

RAP Source	RAP Content	Air Void (%)	No. of fatigue Cycles	Average No. of cycles	Std. Dev	COV (%)
Shilling	20	7.1	6391	7567	1071	14.2
		7.0	8486			
		7.0	7824			
	30	6.8	3396	3619	193	5.3
		6.5	3730			
		6.8	3731			
	40	7.0	1700	1771	92	5.2
		7.0	1875			
		7.0	1739			
Konza	20	7.1	7109	6788	309	4.6
		7.2	6765			
		7.1	6491			
	30	7.3	4986	4705	697	14.8
		7.3	3911			
		7.4	5218			
	40	7.2	1009	960	76	7.9
		7.5	873			
		7.5	1000			
US-73	20	7.1	14238	13705	598	4.4
		7.1	13820			
		7.1	13059			
	30	6.5	11665	11281	336	3.0
		6.5	11134			
		6.8	11044			
	40	6.8	5310	5598	414	7.4
		7.1	5413			
		7.0	6073			

**Table A.7: Fatigue Life Prediction Parameters**

RAP Source	RAP Content	Dynamic Modulus (MPa)	K1	K2	K3	Strain Levels ( $\mu\epsilon$ )		
						300	500	700
Shilling	20	12999	2.8E-129	3.29	34.45	106905.8	19912.18	6581.994
	30	12726	2.3E-129	3.42	34.45	24514.01	4272.594	1351.866
	40	12180	1.8E-129	3.24	34.45	9389.652	1794.153	603.1213
Konza	20	8317	1.7E-127	3.11	35.64	174506.8	35633.84	12514.24
	30	6673	2.2E-126	3.17	35.64	603.6487	119.5429	41.14318
	40	8358	1.6E-128	3.35	35.64	4800.338	867.1186	280.8986
US-73	20	9313	2.9E-127	5.65	36.9	837652.2	46732.42	6982.242
	30	8570	1.8E-129	5.89	37.95	821015.3	40519.32	5584.291
	40	5096	2.4E-127	6.12	39.8	576316.2	25289.86	3225.84

# K-TRAN

## KANSAS TRANSPORTATION RESEARCH AND NEW-DEVELOPMENT PROGRAM

