Evaluation of a Surface Mixture with Delta-S Rejuvenator on NCAT Pavement Test Track

by

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A thesis submitted to the Graduate Faculty of
Auburn University
in partial fulfillment of the
requirements for the Degree of
Master of Science

Auburn, Alabama
December 16, 2017

Keywords: Asphalt, RAP, Rejuvenator

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ABSTRACT

Recycled materials have been commonly used in asphalt mixtures because of their economic, environmental, and social benefits. Reclaimed asphalt pavements (RAP) and reclaimed asphalt shingles (RAS) are the main sources of recycled binder in asphalt mixtures, but their use has been limited to low proportions. The main concern with the use of higher contents of recycled materials in asphalt mixtures is that the resulting pavements could be more susceptible to distresses associated with aged binder. The higher potential for the occurrence of these distresses could ultimately result in higher pavement maintenance and rehabilitation costs. Rejuvenators are said to address these problems by restoring the properties of aged binders in mixtures that contain higher proportions of recycled materials.

The objective of this study is to evaluate the effects of a bio-based rejuvenator, called Delta-S, on the laboratory properties and field performance of asphalt mixtures. The National Center for Asphalt Technology (NCAT) built a section on the NCAT Pavement Test Track for the 2015 Research Cycle using a high RAP surface mixture with Delta-S. This mixture was subjected to a series of laboratory performance tests to determine the performance grade of the extracted binder, rutting and moisture susceptibility, stiffness, and cracking resistance. Field performance data from the Test Track were also obtained. The laboratory properties and field performance data were analyzed and compared with three other sections of the Test Track built with surface mixtures of different material characteristics.

Results of the study indicated that Delta-S did not have any negative effects on ride quality or rutting performance of the mixture. The laboratory cracking performance for the mixture with Delta-S was generally lower than a mixture with similar gradation and a softer virgin binder, although field performance could be considered similar. Still, Delta-S gave the mixture performance properties close to those of a mixture with less RAP contents. In summary, Delta-S may be considered a viable alternative for asphalt mixtures with high RAP contents, and continuing evaluation of the Delta-S mixture is recommended on the Test Track.

ACKNOWLEDGEMENTS

I would first like to thank and acknowledge my advisor, Dr. Nam Tran, for the support he has given me from the day I arrived at Auburn. His advice, guidance, and the lessons I learned from him have been invaluable.

Secondly, I would like to thank Dr. David Timm and Dr. Adriana Vargas for serving on my thesis committee. Also, I want to thank all the great professors, engineers, technicians, office personnel, and graduate students at NCAT for their incredible work and the friendly environment that they provided every day.

I would like to finish by acknowledging the support that my family has given me. I always felt their encouragement, even being thousands of miles away.

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CHAPTER 1 – INTRODUCTION

1.1 Problem Statement

It has been a common practice in asphalt pavement design and construction to include a proportion of recycled materials in new mixtures. In the last decade, the rising prices of crude oil on the global market have made the cost of virgin asphalt binders increase. Consequently, the use of recycled materials has grown as an alternative to keep the prices of asphalt mixtures down. According to the National Asphalt Pavement Association (NAPA), the industry saved $2.6 billion on virgin materials in 2015 by using reclaimed asphalt pavements (RAP) and reclaimed asphalt shingles (RAS) in asphalt mixture production. The average percent RAP used in asphalt mixtures also grew from 15.3% in 2009 to 20.3% in 2015 in the United States, as shown in Figure 1 (Hansen and Copeland, 2017).

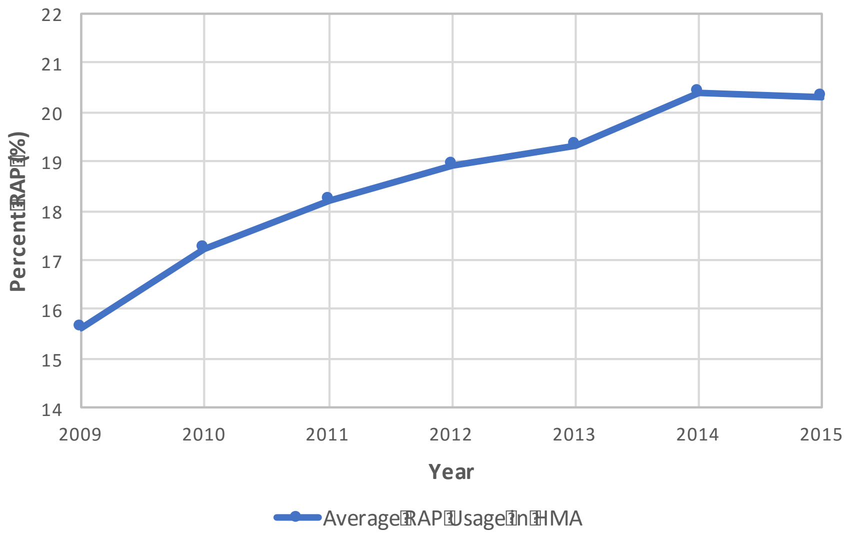

Figure 1. Average RAP usage in asphalt mixtures (Hansen and Copeland, 2017)

There have been concerns, however, that using high contents of recycled materials in asphalt mixtures could have a negative impact on the performance of asphalt pavements. Recycled binders in RAP and RAS are generally stiffer after having endured oxidation and aging processes that change their original properties. This increased stiffness may improve the rutting resistance of asphalt mixtures, but it can also make them more brittle and susceptible to cracking distresses. Recycling agents, such as softening agents or rejuvenators, are used in asphalt mixtures to restore the physical and chemical properties of aged asphalt binders from recycled materials (Brown et al., 2009).

In July 2015, the National Center for Asphalt Technology (NCAT) built a high RAP pavement section at its Test Track using a rejuvenating agent called Delta-S. According to Collaborative Aggregates, the manufacturer of Delta-S, this product restores some original properties of asphalt binders in recycled materials by reducing the effect of oxidation that causes asphalt mixtures to become brittle. Delta-S was chemically designed to not only soften asphalt binders, but to restore virgin properties of old and brittle binders (Collaborative Aggregates, 2017).

The current project presents the results of a field study done at the NCAT Test Track on several sections built with and without the use of Delta-S in their mixtures. Samples of the surface mixtures from those sections were subjected to a battery of laboratory performance tests to determine the binder properties, stiffness, and cracking performance. Field performance data of the pavements at the Test Track were also obtained, which included field measured cracking, rutting, and ride quality. The analysis and comparison of the performance results of Section N7, which was built with 35% RAP and Delta-S, with other test sections would serve to obtain more knowledge of the influence of the rejuvenator on the performance of asphalt mixtures with high contents of recycled materials.

1.2 Objective and Scope

The overall purpose of this study was to evaluate the effects of Delta-S on improving the field performance of a surface asphalt mixture with a high RAP content. This study included several tasks with the following specific activities:

- Measurement of the performance properties of the plant produced mixture in the laboratory.
- Evaluation of the field performance of the mixture at the NCAT Pavement Test Track.

- Comparison of the field and laboratory performance of the mixture containing Delta-S with the performance of mixtures from some other sections built at the Test Track.

1.3 Organization of Thesis

This thesis is divided into five chapters. Chapter 1 includes the background and objectives of the study. Chapter 2 consists of a literature review of recycled materials in asphalt mixtures, aging processes, and rejuvenating agents. Previous laboratory and field studies involving the effects of rejuvenators in asphalt mixtures are also covered in this chapter. The experimental plan is detailed in Chapter 3. This includes a brief description of the test sections analyzed in the study, and an overview of the tests and the methodologies followed to obtain the laboratory and field performance properties of the mixtures. The results are presented and analyzed in Chapter 4. Finally, Chapter 5 provides the conclusions based on the laboratory and field performance of the mixtures analyzed. Individual results of the tests performed are included in the Appendices.

CHAPTER 2 – LITERATURE REVIEW

2.1 Recycled Materials in Asphalt Mixtures

Recycled materials have been widely used in asphalt paving mixtures due to their environmental, economical, and social benefits. The conservation of natural resources and reduction of the energy consumption associated with material extraction and mixture production are among the main environmental benefits of using recycled materials in asphalt mixtures. The economic benefits include mainly the cost reduction of the mixtures when a proportion of virgin aggregates and binders are replaced by less expensive recycled materials. Decreasing landfill areas is another benefit associated with the use of recycled materials in asphalt mixtures. (West, 2015).

For these reasons, reclaimed asphalt pavement (RAP) has been the most widely used recycled material in new mixtures. It is obtained from removed asphalt pavements in roads or parking lots, plant waste, or rejected paving mixture that is further processed for reuse. RAP is considered a source of recycled aggregate and asphalt binder in new mixtures. It is a common practice among asphalt designers and producers to use RAP as a component in new mixtures, with most US states currently allowing the use of RAP in new pavements. Figure 2 shows the average RAP percentage used in asphalt mixtures in US states, from surveys conducted by the National Asphalt Pavement Association (NAPA) for several years (Hansen and Copeland, 2017). It was reported that 74.2 million tons of RAP were used in new pavements in the USA in 2015, with an estimated 85.1 million tons of RAP being stockpiled nationwide by the end of that year.

Most state highway agencies currently allow the use of up to 25% RAP in their asphalt mixtures (Hansen and Copeland, 2017).

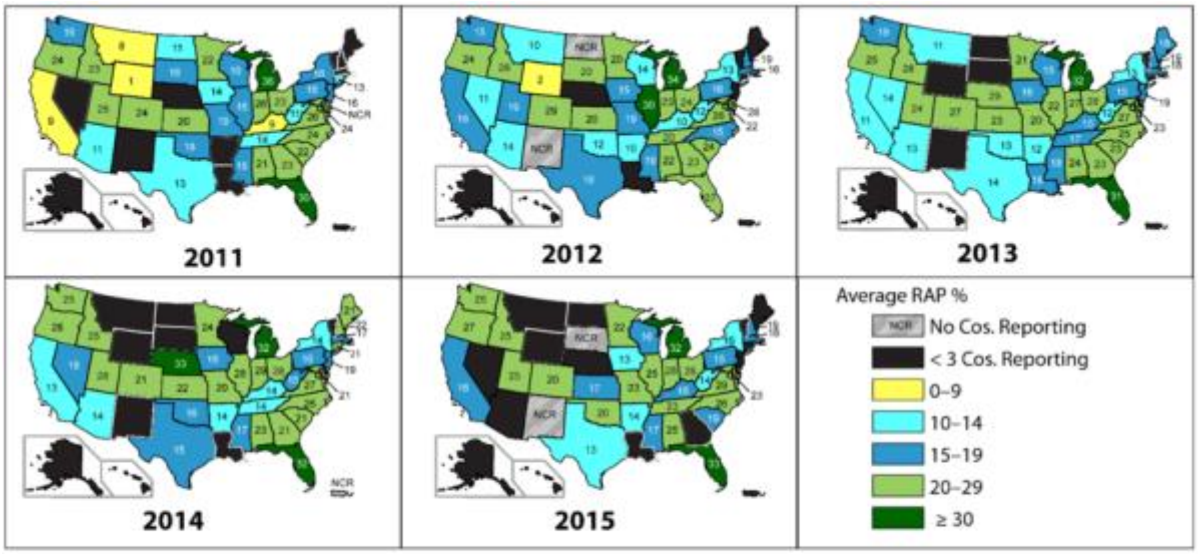

Figure 2. Average RAP used in asphalt mixtures by state (Hansen and Copeland, 2017)

Recycled asphalt shingles (RAS) is another material that has been explored mainly as a source of recycled binder in new mixtures. This material usually comes from roofing, either manufacture waste or post-consumer tear offs, that is ground and processed. Its major components include asphalt, mineral filler, glass fiber, and nails. RAS usually contains 19-36% of asphalt binder by weight (Willis et al., 2016). The use of RAS in asphalt mixtures is still relatively new. Some types of RAS have typically been allowed in proportions of up to five percent of in some US states. In 2015, the average percent RAS use on mixtures in the USA was found to be at around 0.5 according to surveys conducted by NAPA. Still, 1.93 million tons of RAS were used in asphalt mixtures in 2015 according to the same surveys, with the volume of unprocessed RAS in stockpiles at around 1.13 million tons (Hansen and Copeland, 2017).

The main reason behind the restrictions in the use of higher contents of recycled materials in asphalt mixtures has been that aged and oxidized recycled binders are usually stiffer than the virgin binders. The virgin binders are selected at the mix design stage, and their type and performance grade is based on the location of the pavement, expected traffic volume, among other factors. Due to their previous exposure to the environment, recycled binders can be more susceptible to cracking distresses. For this reason, there have been concerns that using higher proportions of RAP and RAS in asphalt mixtures could likely result in stiffer pavements that are prone to cracking and would ultimately result in higher maintenance and rehabilitation costs (Tran et al., 2012).

2.2 Asphalt Binder Composition

Asphalt binder is a dark brown to black mineral substance that can be found naturally or obtained through petroleum oil distillation. Most of the asphalt cement used as binder in mixtures comes from processing crude oils in refineries. Asphalt binders have two main components, which are asphaltenes and maltenes.

Asphaltenes are generally dark brown insoluble solids. They are the components with the highest polarity, and tend to interact and group together. Maltenes are the dissolved component of asphalt cement, and are usually comprised of resins and oils. Resins are a dark fluid in the presence of heat, and in cold conditions, they stiffen and become brittle. Oils are colorless or white liquids with paraffinic and naphthenic structures and little to no oxygen or nitrogen. As maltenes react to oxygen in the environment they tend to transform part of their composition into asphaltene type molecules (Brown et al., 2009).

2.3 Aging of Asphalt Binders

Asphalt binder is considered a colloidal or micellar system in which asphaltenes are the dispersed phase flowing through maltenes, which act as the dispersion medium. As the asphalt binders age over time and through exposure to the environment, changes in their chemical composition begin to occur that affect the balance of their structure. The maltenes start to transform into the asphaltene phase, and with less maltenes available to allow dispersion, the asphaltene particles start to flocculate. This affects the ductility and viscosity properties of the binders, which ultimately impacts their stretching properties. As the aging process increases, the resistance of asphalt binders to cracking and fracture decreases (Brown et al., 2009; Tran, et al., 2012).

During the production and construction stage of asphalt mixtures, elevated mixing and compaction temperatures are required. The mixing temperature must be in a range that permits proper softening of the binder that allows an adequate coating of the aggregate particles, while the compaction temperature must ensure good handling and constructability. Volatilization and oxidation of the binder happen at this stage, which results in an increase of the binder stiffness that leads to degradation of the mixture. This degradation process of asphalt mixtures due to high temperature and oxygen exposure during the production and construction stage is called short-term aging. This can manifest in the pavement being more susceptible to cracking, weathering, among other distresses associated with brittleness in the binder. (Tran, et al. 2012).

Long term aging, on the other hand, has been defined as the longer, never-ending process of oxidation that occurs more extensively over time while the mixture is in service. This process starts immediately after the short-term aging time frame. It evolves with time as the asphalt is exposed to the environment at relatively lower temperatures for a long duration throughout the lifetime of the pavement (Ezree, et al., 2013).

Due to changes caused to the binder by these aging processes, it is important to control the blending of recycled and virgin binders. Aged recycled binders blended with virgin binders may produce stiff mixtures that do not have an adequate performance. Previous research and experience suggest that the effect may not be too significant when aged or recycled binders represent 10% or less of the total binder content. At higher contents, aged binders can significantly affect the properties and performance grade of the blend (Al-Qadi et al., 2007).

The blending process between aged and virgin binders was analyzed by Oliver (2001). In this study, 0% and 50% RAP laboratory produced mixtures with similar aggregate gradation and binder contents were compared. The RAP used for this experiment came from laboratory produced samples that were compacted and long term aged. These compacted samples were then crushed and processed to obtain RAP. If complete blending of the virgin and aged RAP binder was achieved in the 50% RAP mixture, the blended binder viscosity would be similar to that of the binder on the 0% RAP mixture. Both mixtures were tested and the 50% RAP mixture had lower fatigue life, a higher modulus, and higher wheel tracking rates than the 0% RAP mixture. The results suggested that aged binders from recycled materials do not always fully blend with the virgin binders in a mixture. This was attributed to the agglomerations that might be formed when the virgin binder forms a shell around the particles covered with aged binder, which ultimately makes the mixture have regions with higher concentrations of low viscosity and fresh binder (Oliver, 2001).

Some recommendations to reduce the aging effects of the recycled binder during mixture production have been the use of counter flow drum mixers or softer virgin binders. Another alternative that has been considered has been the use of binder rejuvenators (Al-Qadi et al., 2007).

2.4 Rejuvenators

Rejuvenators are a type of recycling agent used to restore the rheological properties of aged asphalt binders, like those available in RAP and RAS. They differ from softening agents because they might not only lower the viscosity of aged binders, but also restore their original properties and composition (Brown et al., 2009). Lubricating oil extracts and extender oils are generally the main components of rejuvenators. The high proportion of maltene constituents in these substances restores the balance of maltenes that were lost or transformed to asphaltenes in aged recycled binders during the short and long term aging processes (Terrel et al., 1989).

Carpenter and Wolosick (1980) described the diffusion of rejuvenators into aged binders as a process that begins with the formation of a low viscosity layer around the aggregates that are coated with the aged binder. After this, the rejuvenator penetrates the aged binder, slowly making it softer. This continues until all the rejuvenator has penetrated the aged binder and an equilibrium is achieved (Carpenter and Wolosick, 1980). Previous studies have suggested that without using rejuvenators on mixtures with high recycled binder contents, a complete blend of the binders might not be achieved (Oliver, 2001; Veeraragavan et al., 2017). It has also been found that the interaction process between rejuvenators and aged binders in mixtures continues even after the mixing and compaction stages, which can have a significant influence on the mechanic properties of the mixture. (Carpenter and Wolosick, 1980).

2.4.1 Delta-S

Delta S is a bio-based rejuvenator developed by Collaborative Aggregates. It was designed to act as a softener and rejuvenator that restores virgin properties to old and brittle binders in recycled asphalt materials. According to the developers of this product, the chemical composition of Delta-S returns binders to their original functionality by reversing natural oxidation processes. It is also recommended as a softener to improve compactibility of asphalt mixtures that may or may not include recycled materials (Collaborative Aggregates, 2017).

2.5 Recent Studies on Effects of Rejuvenators

A study by Zaumanis et al. (2014) analyzed the effects of different types of rejuvenators on aged asphalt binders. The rejuvenators evaluated were petroleum and bio-based. Results showed that all the rejuvenator types reduced the aged binder viscosity to the level of virgin binders at intermediate temperatures. At increased temperatures, the binder viscosities remained higher on rejuvenated binders than on virgin binders. From tests performed on the rejuvenated binders, the fatigue performance was improved by the rejuvenators from bio-based rejuvenators, while petroleum-based rejuvenators had no significant effect on the fatigue life of the aged binder. Rutting tests were performed on laboratory produced mixture samples using the different rejuvenators, and a good correlation was found between rejuvenated binder penetration tests and rutting performance of the mixtures. The rejuvenators that produced a lower binder penetration had a better mixture rutting performance. A high rutting resistance was found on all the rejuvenated mixtures in Hamburg tests. (Zaumanis et al., 2014).

The effectiveness of bio-based rejuvenators was studied by Porot et al. (2016). The first part of this study focused on the effects of a bio-based rejuvenator on a single recycled binder source. The rejuvenator dosages tested ranged from 1 to 15% by weight of recycled binder. Results from this part of the study indicated that even at dosages below 10%, the bio-based rejuvenator helped restore the properties of the aged binders to meet penetration grade specifications (Figure 3). On average, it was found that a 5% dosage was appropriate to restore aged binder properties. It was also found that the viscosity of aged binders was reduced to levels acceptable for mixing at 160 °C. The study analyzed the effect of a 5% dosage of rejuvenator on four types of recycled binders and two virgin binders, of which one was aged in the pressure aging vessel (PAV).

Results showed consistent trends, in which the rejuvenators had a positive effect on the penetration values and softened the recycled asphalts, as shown in Figure 4 and Figure 5. In the end, it was concluded that bio-based rejuvenators at a 5% dosage had a positive effect in restoring the properties of aged binders, and that aged binders treated with this type of rejuvenator were less susceptible to aging over time (Porot et al., 2016)

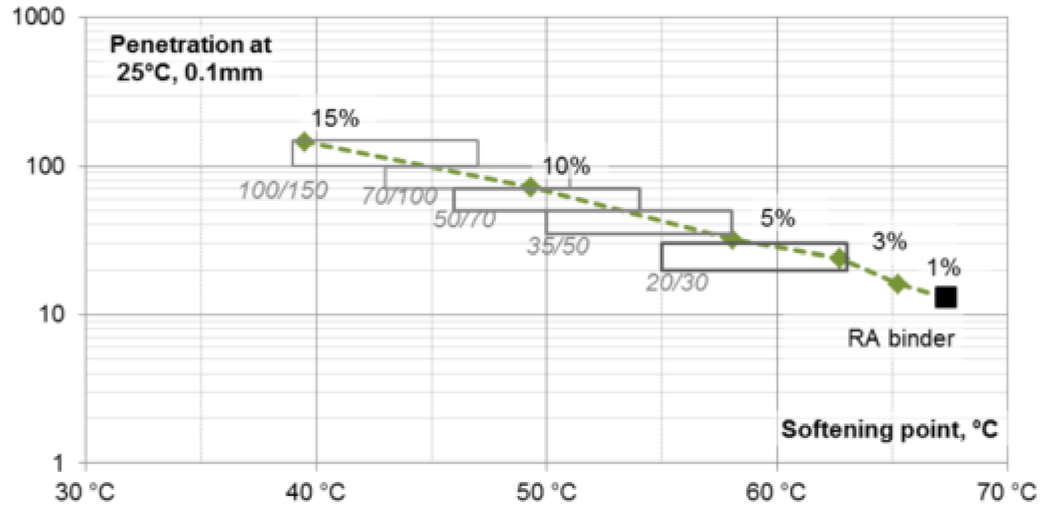

Figure 3. Effect of additive dosage on penetration and softening point (Porot et al., 2016)

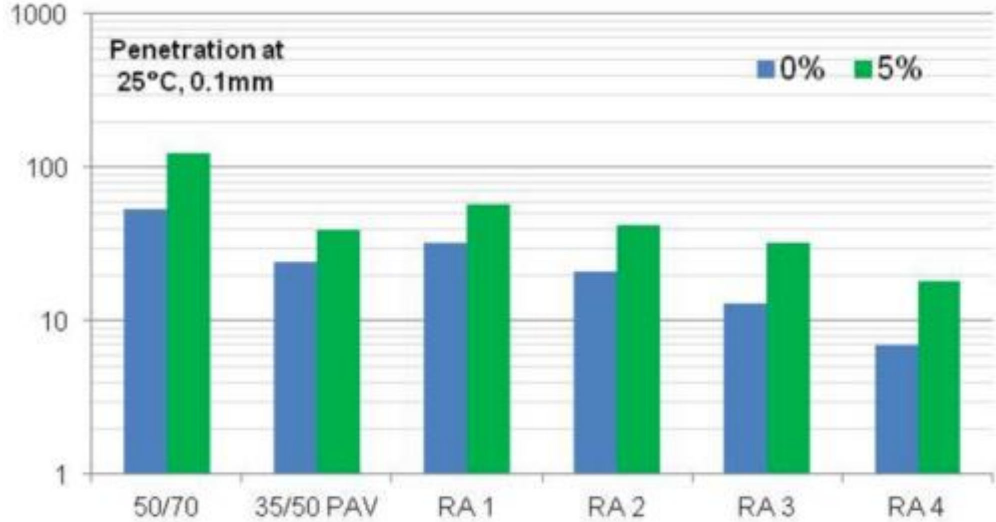

Figure 4 Effect of 5% rejuvenator on penetration (Porot et al., 2016)

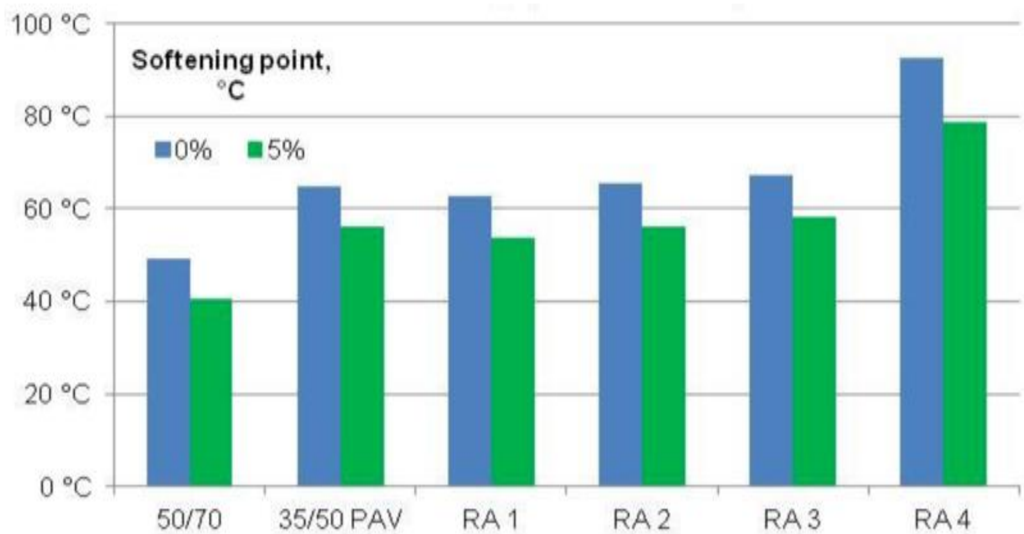

Figure 5. Effect of 5% rejuvenator on softening point (Porot et al., 2016)

A study was done by Grilli et al. (2016) in which the correlation of binder properties from laboratory and plant produced asphalt mixtures with 30% RAP and rejuvenators were analyzed. When comparing the interaction of binders and rejuvenators on laboratory and plant produced samples through infrared spectrometry, no significant differences were found. It was found that short term aged mixture samples produced in the laboratory with high RAP and rejuvenators had similar properties as plant produced mixtures and the interaction of the rejuvenators in the laboratory samples was representative of field performance (Grilli et al., 2016).

2.6 Rejuvenated Mixtures Laboratory and Field Performance

Tran et al. (2012) investigated the effects of rejuvenators on laboratory mixtures properties produced with high percentages of RAP and RAS. The mixtures used for this study consisted of a virgin control mixture, 50% RAP mixtures with and without rejuvenator, and 20% RAP 5% RAS mixtures with and without rejuvenator. The laboratory performance of these mixtures was analyzed using various cracking and rutting tests. The stiffness of the mixtures was also analyzed through dynamic modulus mastercurves. Results showed that the rejuvenator improved the cracking performance of the mixtures with high RAP percentages, while it did not seem as effective on the mixtures with RAP and RAS at some instances. The rejuvenated mixtures also appeared to age faster based on the dynamic modulus mastercurves of the long term aged specimens. The laboratory rutting performance was not affected on the mixtures with rejuvenators (Tran et al., 2012).

Veeraragavan et al. (2017) compared 20% RAP mixtures like those commonly used by the Maine DOT to 50% RAP mixtures produced in laboratory with and without rejuvenators. The rejuvenators used included a generic waste vegetable oil and a bio-based rejuvenator. Short term aged samples of these mixtures were tested for volumetric properties, moduli, and cracking potential. The results from this study indicated that 50% RAP mixtures with rejuvenators could have equal or better cracking performance than the more commonly used 20% RAP mixtures. This was based on semicircular bend, indirect tensile strength, and creep compliance test results. Other benefits of using rejuvenators pointed out were that they facilitate the production of mixtures with higher recycled binder contents, and the potential cost savings that could represent from the use of rejuvenated high RAP mixtures (Veeraragavan et al., 2017).

Similar results were found in another study by Kodippily (2016) using laboratory produced samples, where short term aged asphalt mixtures samples with 15 and 30% RAP contents with and without rejuvenators were compared. Maltene fractions and chemical rejuvenators were tested in this study. The results indicated that the overall performance of the 30% RAP mixtures with rejuvenators were better than that the 15% RAP mixture, with notably higher modulus and higher fatigue resistance. The study concluded that the addition of rejuvenators had the potential to increase the fatigue properties of asphalt mixtures without compromising the rutting performance benefits that high RAP mixtures provide (Kodippily, 2016).

The effects of a rejuvenator on the field performance of asphalt mixtures with high recycled binder proportions was analyzed using data from pavement sections built in Missouri by Tran et al. (2015). Three test sections were built. Section 1 with a 30% RAP, section 2 with 40% RAP, and section 3 with a 20% RAP 5% RAS mixture. The mixtures in sections 2 and 3 were produced with rejuvenators. According to laboratory tests, the rejuvenated mixtures in sections 2 and 3 had similar cracking resistance to the mixture in section 1, which had less recycled binder contents. The rutting performance was also appropriate for the rejuvenated mixtures based on wheel tacking tests. After 10 months of regular traffic circulation, the sections were evaluated. It was found that similar low severity cracks had formed on all the sections, and none of the sections had any measurable rutting. This study concluded that rejuvenators could give mixtures with high recycled binder contents similar laboratory and early performance to mixtures with lower recycled binder contents (Tran et al., 2016).

Early performance of test sections built with rejuvenated mixtures was also analyzed by Xie et al. (2017). This evaluation compared the laboratory and field performance of three test sections built in Alabama with experimental mixtures. Different types of rejuvenators and dosages were added to two of those mixtures, with both having 25% RAP and 5% RAS. The other section was built with 20% RAP and no rejuvenator, and it was established as the control mixture for the experiment. From laboratory tests, the rejuvenated mixtures appeared to have significantly lower cracking resistance than the control mixture. The rutting performance was also predicted to be better in the rejuvenated mixtures. After two years of trafficking, the sections were analyzed and the control mixture was found to have notably better cracking performance than both rejuvenated mixtures, while the IRI and rutting performance was acceptable for all the mixtures. It was concluded that the rejuvenators appeared to have a higher effect in the RAP binder than in the RAS binder, and that laboratory performance measured initially for the mixtures was in accordance with field performance after two years of traffic on the test sections (Xie et al., 2017).

A summary of previous studies and findings on the effects of rejuvenators from the literature review is shown in Table 1.

Table 1. Summary of previous studies on rejuvenators in asphalt mixtures

Study	Subject of Study	Findings
Zaumanis et al. (2014)	• Effects of different types of rejuvenators	• Reduced viscosity at intermediate temperatures. • Bio-based rejuvenators have higher effect on fatigue performance of aged binders • Good correlation between penetration tests and rutting performance
Porot et al. (2016)	• Effect of bio based rejuvenators at various dosages • Effect of 5% dosages on different sources of recycled binders	• Rejuvenators restored properties of binders to meet penetration grade specifications • 5% dosages restore binder properties by two grades
Grilli et al. (2016)	• Effect of binder and rejuvenator interaction on laboratory and plant produced mixtures	• No significant differences • Short term aged rejuvenated laboratory mixtures similar to plant produced mixture
Tran et al. (2012); Veeraragavan et al. (2016); Kodippily, (2016)	• Performance properties of laboratory produced asphalt mixtures with high recycled contents	• Rejuvenators improved the cracking performance of high RAP mixtures • Rutting performance not compromised • High RAP mixtures with rejuvenators can have equal performance to lower RAP mixtures • Rejuvenated mixtures aged faster based on analysis of long term aged specimens
Tran et al. (2016); Xie et al. (2017)	• Laboratory and field evaluation of rejuvenated mixtures on test sections	• Rejuvenators can give mixtures similar laboratory and early performance to mixtures with lower recycled binder contents • On RAS mixtures, rejuvenator not as effective • Good rutting performance on laboratory and field • Worse cracking performance on RAS mixtures

CHAPTER 3 – EXPERIMENTAL PLAN

3.1 Test Sections

The current study focused on the evaluation of laboratory and field performance of the surface mixture in Section N7 of the NCAT Test Track from the 2015 research cycle and how it compared to the surface mixtures in Section N1, N8, and S5. The 9.5 mm NMAS N7 mixture was produced with 35% RAP and a PG 64-22 virgin binder with Delta-S as a rejuvenator. The rutting and moisture susceptibility were analyzed for the surface mixture of Section N7 as part of the mix design process. Binders were extracted from all the mixtures analyzed to determine the performance grading of the blended binders. The stiffness and cracking performance were compared with the surface mixtures on sections N1, N8, and S5 using various laboratory performance tests, as shown in Table 2. The cross-sections of the pavements analyzed in this study are shown in Figure 6.

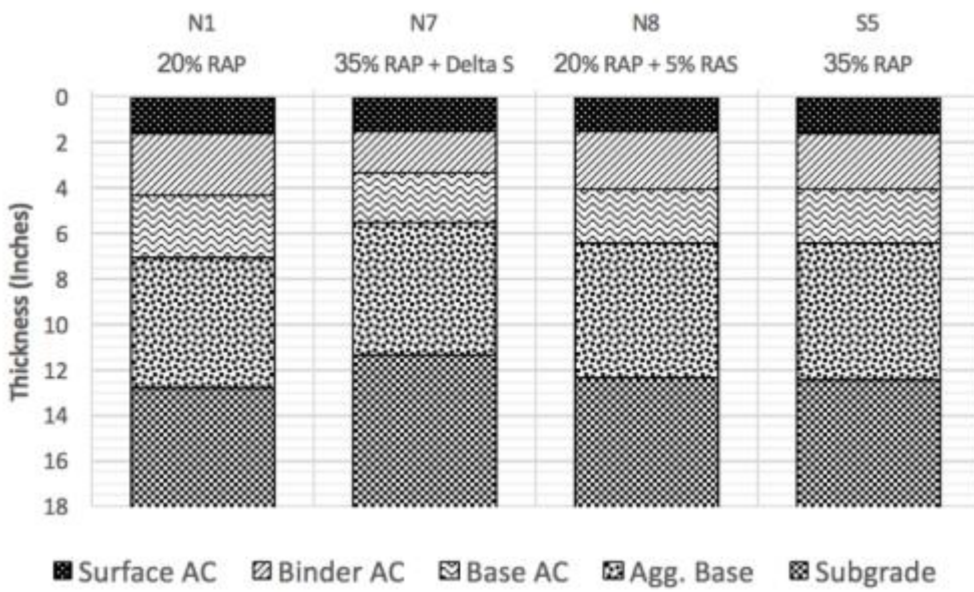

Figure 6. Layer structures of analyzed sections

Table 2. Laboratory testing plan

Property	**Test Conducted**
Binder properties	Extracted binder PG
Moisture susceptibility	Hamburg Wheel Tracking Test
Rutting resistance	
Mixture stiffness	Dynamic modulus testing
Cracking performance	Energy Ratio
	Texas Overlay Test
	NCAT Overlay Test
	Louisiana Semi Circular Bend
	Illinois Flexibility Index Test

A summary of properties of the surface mixtures of all the sections analyzed is shown in Table 3. A brief description of each mixture can be found after that. The base and binder AC layers in all the sections consisted of the same highly polymer-modified asphalt mixture, which was designed to be resistant to fatigue cracking. The aggregate base layer consisted of a crushed granite base. The subgrade at the Test Track is classified as an A-4 soil according to the AASHTO soil classification system.

Table 3. Surface mixture QC properties

Mixture	**Virgin Binder PG**	**As-built Thickness (in)**	**In Place %G_{mm}**	**QC P_{be} (%)**	**QC V_a (%)**	**QC VMA (%)**	**Recycled Binder Ratio**
N1 20% RAP	64-22	1.6	93.6	4.7	7.0	14.7	0.177
N7 35% RAP + Delta-S	64-22	1.5	92.1	5.2	7.0	16.0	0.282
N8 20% RAP + 5% RAS	64-22	1.5	91.5	4.8	7.0	14.4	0.372
S5 35% RAP	58-28	1.6	92.2	5.1	7.0	15.1	0.292

3.1.1 N1 (20% RAP)

Section N1 was built using a 9.5 mm NMAS mixture with 20% RAP and a PG 64-22 virgin binder. It represents a typical mixture in the industry, where according to NAPA, an average 20.3% RAP is used in asphalt mixtures (Hansen and Copeland, 2017). This mixture was selected as a control mixture for a Cracking Group experiment, which was carried out at the Test Track as part of its 2015 research cycle.

3.1.2 N7 (35% RAP + Delta-S)

The construction of Section N7 was done as part of a study sponsored by Collaborative Aggregates to test the effectiveness and influence on pavement performance of their Delta-S rejuvenator. Section N7 was initially built with a 20% RAP 5% RAS mixture and 10% Delta-S by weight of recycled binder. Cracking was noticed in the wheelpaths of this section shortly after traffic circulation began. It was revealed through core analysis that the premature failure was caused by debonding of the surface and intermediate layers. The entire AC structure was rebuilt using the same mixture designs, but this repair was short-lived and slippage problems related to debonding were noticed just days after finishing the reconstruction. After laboratory analysis, the debonding was attributed to the low silo storage time that the surface mixture had during construction. This reduced time did not allow a full interaction between the rejuvenator and the aged binder, making the virgin binder and the mixture excessively soft and susceptible to bond strength and shoving failure. After bond strength tests of laboratory compacted mixture samples aged for 0, 2, and 4 hours, a two-hour silo storage was recommended for the new repave of Section N7.

Section N7 was rebuilt once again, this time with a 35% RAP surface mixture. This mixture was designed with the same aggregate gradation and virgin binder content as Section S5, but with a PG 64-22 virgin binder. The mixture was produced with Delta-S injected in-line via the AC supply at a target rate of 5% by weight of aged binder. To give the Delta-S time to interact with the aged binder in the RAP and avoid the debonding issues caused by lack of interaction between the aged binder and rejuvenator, the mixture was given a two-hour silo storage time prior to being placed and compacted on the field.

3.1.3 N8 (20% RAP + 5% RAS)

Section N8, also part of the Cracking Group experiment at the Test Track, had a surface mixture designed to include 5% RAS content in addition to the 20% RAP and PG 64-22 virgin binder that the control mixture had. The aggregates used were also the same, but the gradation was slightly modified to accommodate the added RAS. The result was a pavement mixture with a high stiffness that was not expected to have an adequate cracking performance.

3.1.4 S5 (35% RAP)

The high RAP section of the Cracking Group, Section S5 was designed with 35% RAP surface mixture. The aggregates used were the same as in the control mixture in Section N1, but in different proportions to include the extra RAP. This mixture used the same aggregate proportions and virgin binder content as the final design of the surface mixture in Section N7. Due to its higher RAP contents, the virgin binder used was of a lower performance grade (PG-58-28) than the one in the other sections.

3.2 Binder Tests

Binders were extracted and recovered from loose plant mixture samples for all the sections. The binders were extracted through a solvent extraction process following AASHTO T164 Method A

“Quantitative Extraction of Asphalt Binder from Hot Mix Asphalt” (AASHTO, 2012). The binder was then recovered through a distillation process following ASTM D5404 / D5404M – 12 “Standard Practices for Recovery of Asphalt from Solution Using the Rotary Evaporator” (ASTM, 2012).

The performance grade of the asphalt binders was determined as per AASHTO R29 “Grading or Verifying the Performance Grade of an Asphalt Binder” (AASHTO, 2012), AASHTO M320 “Standard Specification for Performance Graded Asphalt Binder” (AASHTO, 2012), and AASHTO R49 “Determination of Low-Temperature Performance Grade (PG) of Asphalt Binders” (AASHTO, 2012). As per ASTM D7643 “Standard Practice for Determining the Continuous Grading Temperature and Continuous Grades for PG Graded Asphalt Binders”, the low temperature continuous grades for each binder were obtained and used to calculate the ΔT_c parameter, as illustrated in Figure 7. This parameter has been found to be an indicator of non-load related cracking potential. AASHTO PP78 “Design Considerations When Using Reclaimed Asphalt Shingles (RAS) in Asphalt Mixtures” recommends a ΔT_c threshold of -5.0 °C for recovered binders (Anderson, 2011).

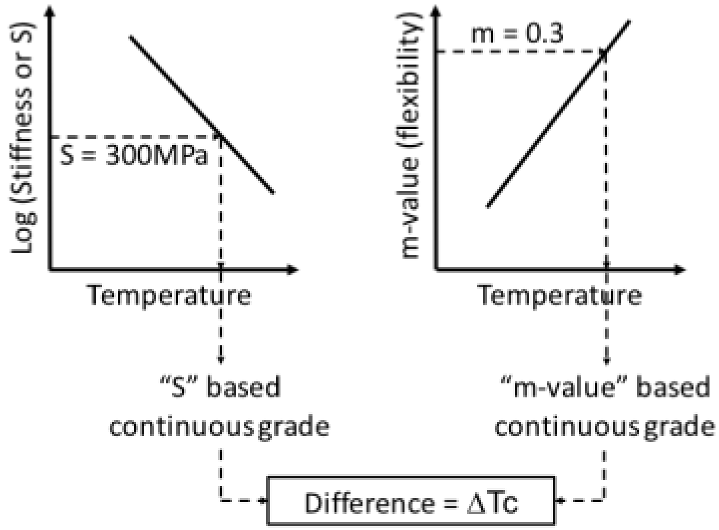

Figure 7. Calculation of ΔT_c

3.3 Hamburg Wheel Tracking Test

Hamburg Wheel Tracking tests were performed to determine the rutting resistance and moisture susceptibility in the surface mixture of Section N7, which was built using Delta-S. Hamburg tests were done to complement the mixture redesign following the slippage problems that occurred when the first design of this section was in service at the Test Track.

The Hamburg test was developed in Germany and has been used since the 1970s. It consists of applying wheel loads to mixture specimens while they are submerged in a water bath at a controlled temperature, as shown in Figure 8. The vertical deformation at every wheel pass is obtained by LVDT sensors and used to determine the rutting resistance and stripping potential of the mixtures.

Figure 8. Hamburg test setup

Hamburg tests were done following AASHTO T 324-14. Two Superpave gyratory compacted (SGC) samples were trimmed to produce four testing specimens. The specimens were compacted to 7.0 ± 1.0 percent air voids, with a diameter of 150 mm and a height of 60 mm. A load of 158 ± 1 lbs was applied with a steel wheel for 10,000 cycles, with each cycle representing two passes. While the wheel loads were applied, the specimens were submerged in water at a controlled temperature of 50 °C. The rut depth at each pass was recorded by the testing equipment. A typical Hamburg testing results output is shown in Figure 9. From these results, the point where the two main tangents intercept is known as the stripping inflection point, and it is used to determine the moisture susceptibility and stripping resistance of the mixture. A minimum threshold of 10,000 passes to the stripping inflection point has been established to differentiate mixtures with good moisture resistance. For rutting performance, the state of Texas has implemented the criteria shown in Table 4 for allowable rut depths in Hamburg testing (Advanced Asphalt Technologies, 2011).

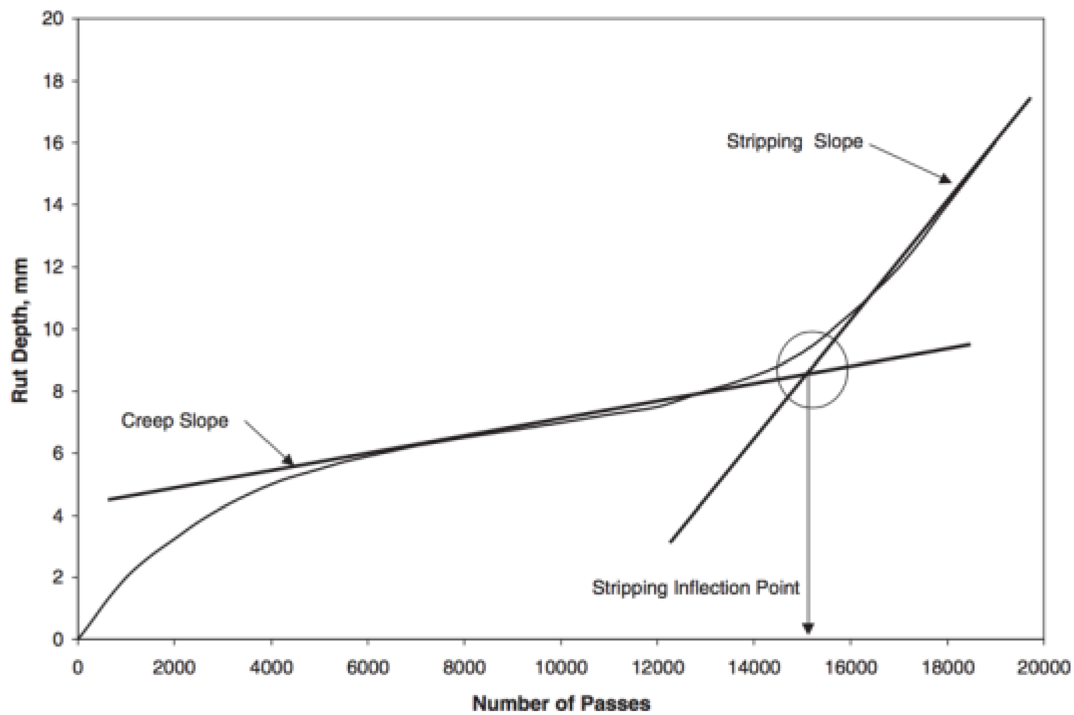

Figure 9. Typical Hamburg results output (Advanced Asphalt Technologies, 2011)

Table 4. Texas specifications for allowable Hamburg rut depth

High Binder PG	Minimum Passes to 12.5 mm Rut Depth
≤ 64	10,000
70	15,000
≥ 76	20,000

3.4 Dynamic Modulus

Dynamic modulus (E*) tests were done following AASHTO T 378-17. An IPC Global Asphalt Mixture Performance Tester (AMPT), shown in Figure 10, was used. The purpose of this test was to obtain the stiffness responses of asphalt mixtures at different temperatures and load frequencies. For this series of tests, three SGC samples were prepared from each mixture. A target 175 mm height and 150 mm diameter were set for the SGC samples. From these samples, cut and cored specimens were obtained for testing. Production tolerances considered for the resulting cut and cored specimens are shown in Table 5.

Figure 10. Asphalt mixture performance tester used for dynamic modulus testing

Table 5. Production tolerances for dynamic modulus testing specimens

Parameter	**Tolerance**
Average Diameter	100 – 104 mm
Standard Deviation of Diameter	≤ 0.5 mm
Height	147.5 – 152.5 mm
End Flatness	≤ 0.5 mm
End Perpendicularity	≤ 1.0 mm
Air Voids	7 ± 0.5%

Testing temperatures and load frequencies for dynamic modulus testing were selected as per AASHTO PP61-13. The high temperature of the test was selected based on the high grade of the PG classification of the binder of each mixture. Table 6 shows the recommended load frequencies based on the test temperature. The high testing temperatures recommended based on the PG grade of the binders are shown in Table 7. Testing for dynamic modulus was done in an unconfined condition.

Table 6. Dynamic modulus testing temperatures and frequencies

Test Temperature (°C)	**Loading Frequencies (Hz)**
4.0	10, 1, 0.1
20.0	10, 1, 0.1
High Testing Temperature	10, 1, 0,1, 0.01

Table 7. Dynamic Modulus high testing temperature

High Testing Temperature (°C)	Binder PG Grade
35	PG 58-XX and softer
40	PG 64-XX and PG 70-XX
45	PG 76-XX and stiffer

Following the method described in AASHTO PP 61-13 and using the Mastersolver.exe program, analysis of the obtained data was done and mastercurves were generated for each of the mixtures tested. Mastercurves use time-temperature superposition principles to express the stiffness of the mixture regardless of their temperature variability. The generated mastercurves between different mixtures can be used for comparison purposes. The higher temperature/low frequency side of the E* mastercurves has been suggested as an indicator to rutting resistance (Nair et al., 2013).

The equation of the mastercurve and the parameters obtained describe the stiffness behavior of the mixtures. The equation can also be used to calculate dynamic modulus values at various ranges of temperatures and frequencies to be used as inputs in mechanistic empirical (ME) pavement design (Brown et al., 2009). The standard error ratio (Se/Sy) and adjusted coefficient of determination obtained are considered goodness of fit statistics between measured and predicted data (Rais et al., 2013). Figure 11 shows an example of a generated mastercurve.

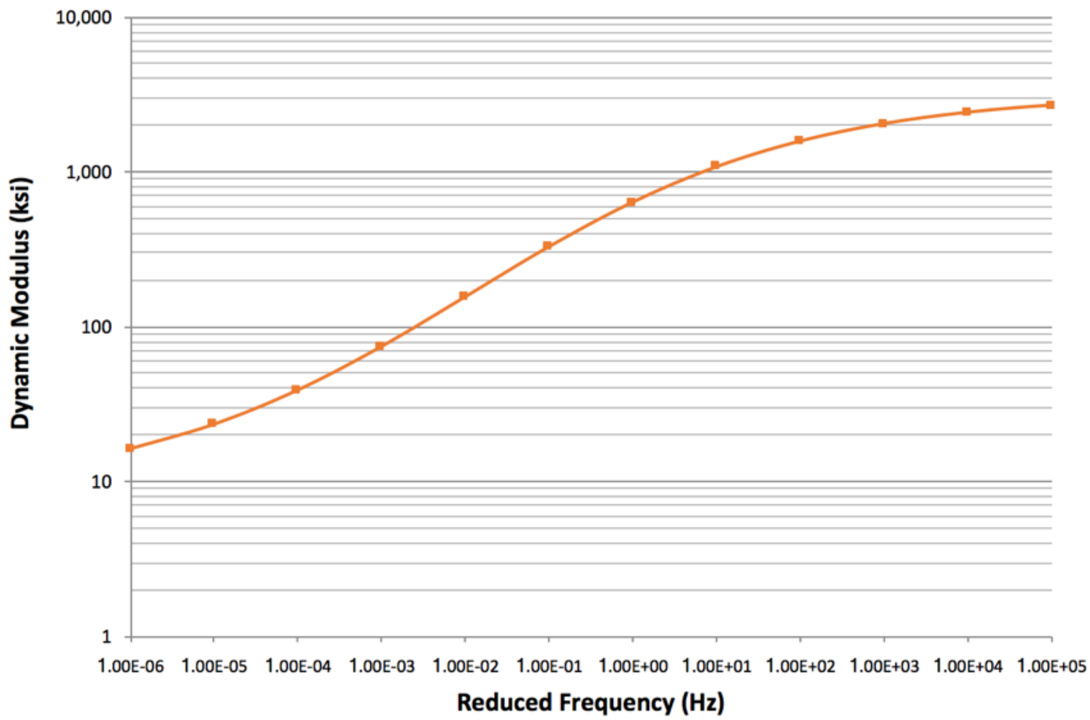

Figure 11. Example of generated mastercurve

3.5 Energy Ratio (ER)

Energy Ratio (ER) is a procedure generally used to assess top-down cracking resistance of asphalt pavements. Testing for resilient modulus as per ASTM D7369-11, creep compliance as per AASHTO T322-07, and indirect tensile strength as per ASTM D6931-12 are integrated in the Energy Ratio procedure. All these tests can be performed on a universal Superpave IDT testing device (shown in Figure 12) at a target temperature of 10 °C. For the different tests, four SGC samples at a target air voids of 7 ± 0.5% were trimmed to testing specimens of 150 mm in diameter and 38 mm thick.

Figure 12. IDT testing device

Using one of the specimens, the loads for resilient modulus and creep compliance testing were defined. The load used for resilient modulus testing was the one that produced a horizontal strain of 100 to 200 microstrain on the specimen. For creep compliance testing, the load used was the one that produces a 100 microstrain horizontal deformation after 100 seconds of testing. The creep compliance load is usually around 10 percent of the load used for resilient modulus testing. Resilient modulus testing was performed in a load-controlled mode, and the resilient modulus was obtained from the stress-strain curve, as shown in Figure 13 (Tran et al., 2012).

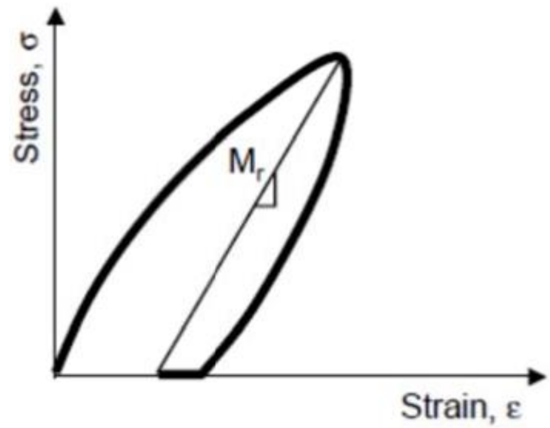

Figure 13. Stress-strain curve for resilient modulus determination (Tran et al., 2012)

Creep compliance testing was performed by applying the determined creep load in constant load control mode for 1,000 seconds. The creep compliance parameters m and D1, are obtained from the results output of this test, as shown in Figure 14 (Birgisson et al., 2006). These parameters are necessary to determine the minimum dissipated creep strain energy ($DCSE_{min}$) for adequate cracking performance (Tasdemir et al., 2010).

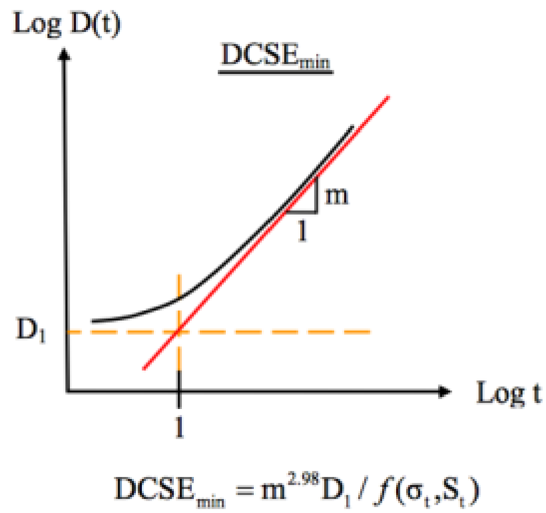

Figure 14. IDT creep compliance curve and parameters (Birgisson et al., 2006)

Indirect tensile strength tests were performed in displacement-controlled mode at a 2 in/min loading rate. The strength (St), fracture energy (FE), and dissipated creep strain energy at failure ($DCSE_{HMA}$) were obtained from stress-strain curves resulting from strength and resilient modulus testing, as illustrated in Figure 15. Using the data collected from the previously described tests, the Energy Ratio was obtained using Equation 1 (Roque et al., 2004).

$$ER = \frac{DCSE_{HMA}}{DCSE_{min}} = \frac{DCSE_{HMA} \times [7.249 \times 10^{-5} \times \sigma^{-3.1}(6.36 - St) + 2.46 \times 10^{-8}]}{m^{2,98} \times D_1} \qquad (1)$$

Where:

σ = Tensile stress at the bottom of the asphalt layer
Mr = Resilient modulus
D1, m = Power function parameters
St = Tensile strength
$DCSE_{HMA}$ = Dissipated creep strain energy at failure
$DCSE_{min}$ = Minimum dissipated creep strain energy required

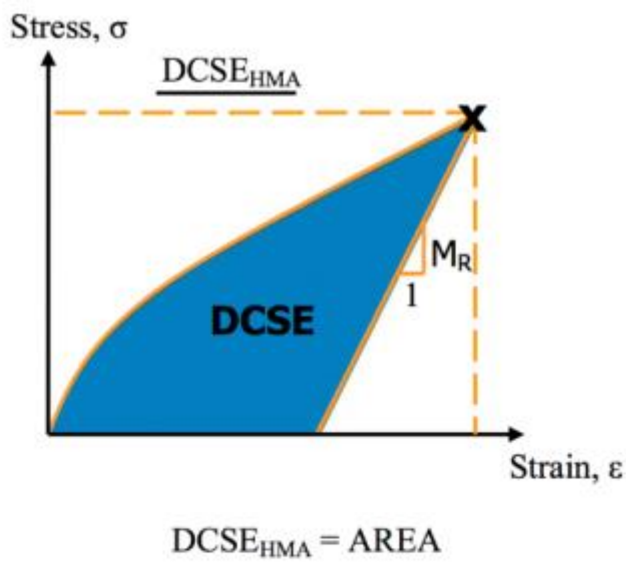

Figure 15. Graphic illustration of DCSE (Birgisson et al., 2006)

Asphalt mixtures with higher FE, $DCSE_{HMA}$, $DCSE_{min}$, and ER are normally expected to have higher resistance to top-down cracking. Based on previous research by Roque et al. (2004), a $DCSE_{HMA}$ range of 0.75-2.5 kJ/m^3 has been suggested as a threshold for top-down cracking resistance. Minimum ER requirements are based on the expected traffic levels, and are shown in Table 8 (Roque et al., 2004).

Table 8. Recommended Energy Ratios by traffic level (Roque et al., 2004)

Traffic (ESALs/Year)	**Minimum Energy Ratio**
Up to 250,000	1.0
250,001 – 500,000	1.3
500,001 – 1,000,000	1.95

3.6 Texas Overlay Test (OT)

The Overlay Test (OT) is a cracking test created in Texas as an approach to predict the reflective cracking resistance of asphalt overlays. The test was designed to simulate reflective cracks of asphalt overlays when they were placed on top of Portland cement concrete pavements (Ma, 2014). A national standard guide to perform this test has yet to be developed. In the state of Texas, the Tex-248F standard is followed.

The specimens used for this test were SGC, with a target height of 125 mm. OT testing specimens were obtained from SGC samples by trimming them to the following dimensions: 150 mm long, by 75 mm wide, by 38 mm tall. Figure 16 shows a SGC sample and trimmed OT testing specimen. The specimens were attached to two steel plates, as illustrated in Figure 17. In the test, loading occurs by moving one of the steel plates away from the other plate, which is fixed. The rate at which the loads are applied is once every 10 seconds, with a sawtooth waveform, as shown in Figure 18. The maximum opening displacement for each cycle is defined by the test operator. Testing was conducted on an IPC Global Asphalt Mixture Performance Tester (AMPT) at 25 °C on a controlled displacement mode, with a maximum opening of 0.635 mm, as shown in Figure 19. The maximum loads applied for each cycle were recorded, and the test was completed when the applied loads were reduced by 93% in comparison with the initial loading cycle.

Figure 16. SGC sample and trimmed OT specimen (Ma, 2014)

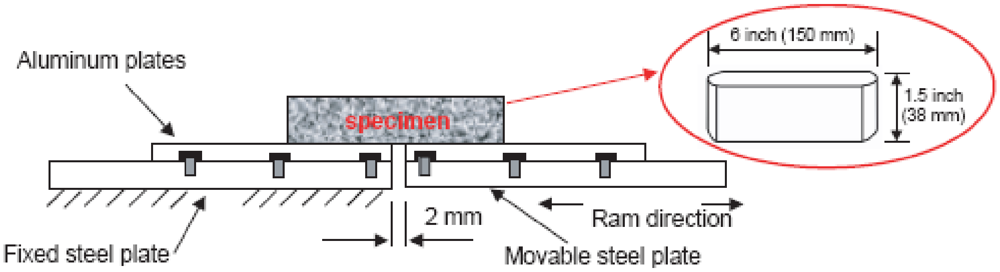

Figure 17. Overlay test illustration (Zhou et al., 2007)

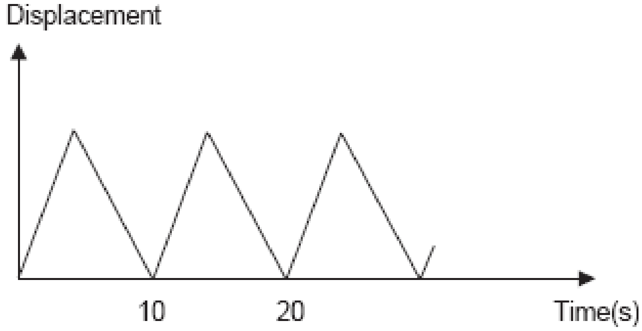

Figure 18. Saw tooth loading waveform (Zhou et al., 2007)

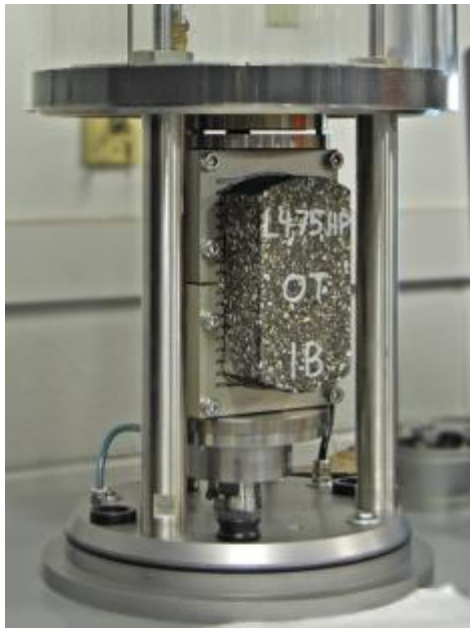

Figure 19. Overlay test setup on AMPT

The Texas Overlay Test has been reported to produce variabilities greater than 30% (Walubita et al., 2012). A threshold of 700 OT cycles has been established for classifying mixtures as fatigue resistant (Chen, 2008). For reflective cracking, 300 OT cycles has been suggested as a pass/fail criterion (Zhou et al., 2007).

3.7 NCAT Overlay Test (NCAT-OT)

A modification of the Texas Overlay Test was recently developed at NCAT (Ma, 2014). This new approach is referred to as NCAT-OT. In the test setup, one of the modifications incorporated was an increased testing frequency of 1 Hz, up from 0.1 Hz in the original Texas OT. The maximum opening displacement was also changed to 0.381 mm. These changes were selected based on studies where asphalt mixtures at certain higher frequencies and maximum opening displacement were tested, and the resulting number of cycles to failure was similar to the results obtained from Texas OT testing. The NCAT-OT maximum opening displacement was also reduced because asphalt mixtures do not expand and contract as much as portland cement concrete, which the Texas OT simulates with its 0.635 mm maximum opening displacement. (Ma, 2014; Tran et al., 2012).

In the NCAT-OT approach, the failure point was also redefined. The “load x cycle” curve peak was determined as the failure point. An illustration of the failure point obtained from the NCAT-OT and Texas OT approaches is shown in Figure 20 (Moore, 2016). Video analysis captured during testing that showed crack propagation through the testing specimen also supported the selection of this failure criteria. The NCAT-OT approach has also been reported to produce lower variabilities than the traditional Texas OT method (Ma, 2014).

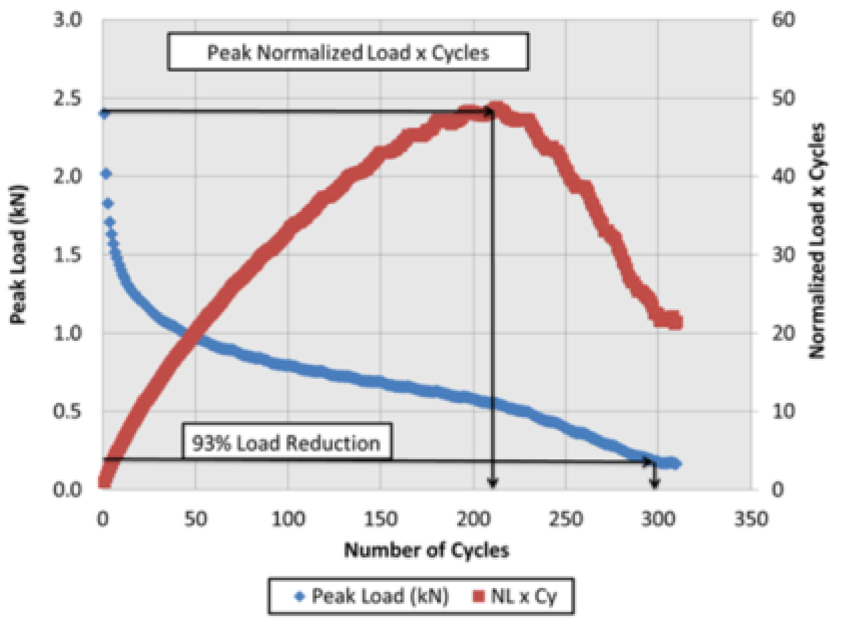

Figure 20. Comparison of Texas OT and NCAT-OT failure point definitions (Moore, 2016)

The specimens used for NCAT-OT testing were of the same dimensions as those in Texas OT testing. The equipment, setup, and preparation were also similar. The 1 Hz load frequency and maximum opening displacement were set in the testing machine operating software. Testing was carried out and the outputs were used to determine the failure point using the "load x cycles" principle. Since this is a new testing approach, a standard Nf in NCAT-OT has not been established to identify fatigue or cracking resistant mixtures.

3.8 Semi Circular Bend (SCB)

Semi Circular Bend (SCB) testing methods have gained the attention of researchers due to their relative simplicity in specimen fabrication from laboratory produced samples or field cores and repeatability (Nsengiyumva et al., 2015). For the current study, SCB tests were performed using the method developed by the Louisiana Transportation Research Center (LTRC) following the DOTD TR 330-14 specification. Semi-circular testing specimens were fabricated for this purpose by cutting SGC samples in half. The SGC samples were produced with a height of 57 mm and an air void content of 7.0 ± 0.5%. In the semi-circular specimens, a 3.0 ± 0.5 mm wide notch was cut at either 25.4, 31.8 or 38.0 ± 1.0 mm deep. At least three specimens were required for each notch depth. Figure 21 shows SCB specimens with the different notch depths used for SCB tests following the LTRC method. The specimens were conditioned for two hours at a temperature of 25 ± 0.5 °C before testing. After setting up the specimen for testing on an IPC Global Asphalt Mixture Performance Tester (AMPT), as shown in Figure 22, a loading rate of 0.5mm/min was applied monotonically until fracture. The deformation and load were recorded and this information was used to determine the parameters that would indicate the cracking resistance of each of the mixtures tested.

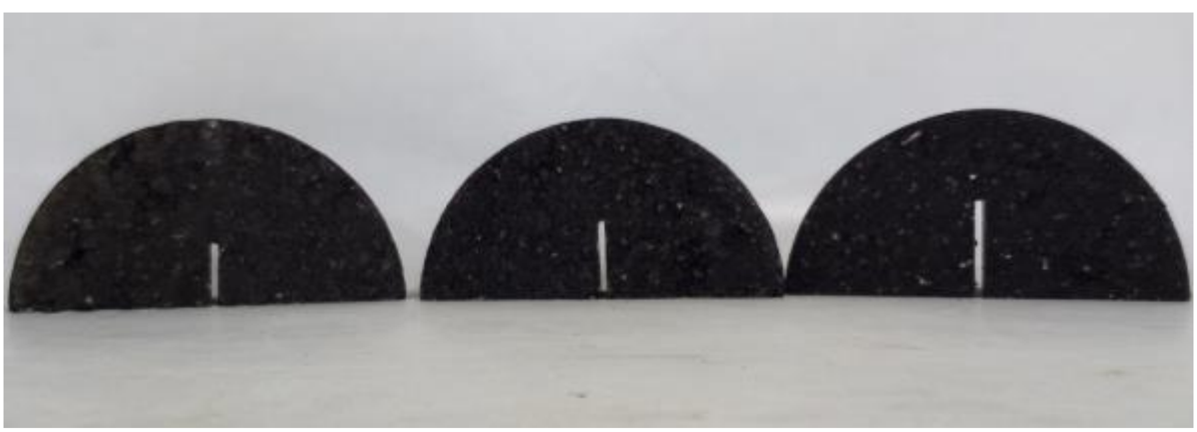

Figure 21. SCB specimens with different notch depths used (Moore, 2016)

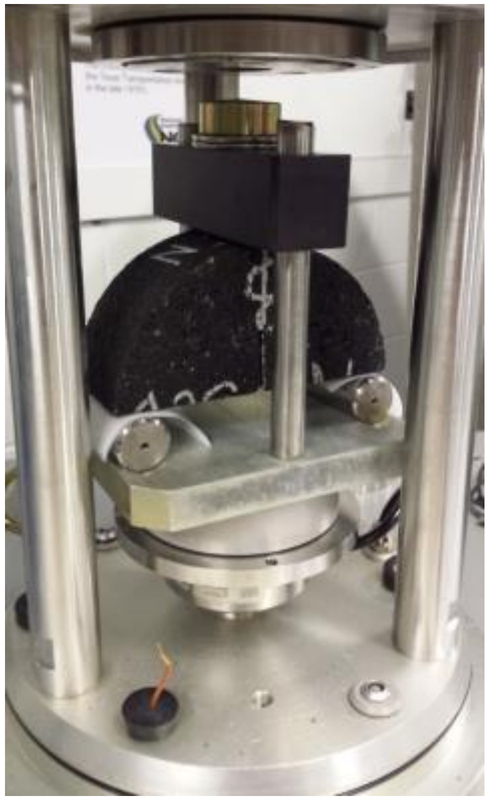

Figure 22. SCB test setup on AMPT

The Louisiana SCB method is based on fracture mechanics concepts, and uses fracture energy and J-integral (critical strain energy release rate, Jc) as response parameters. The strain energy at the peak load, which is usually inversely proportional to notch depth, is calculated from the load deformation curves obtained from the testing outputs, as illustrated in Figure 23. The strain energy values obtained for each mixture are then plotted over the three different notch depths (Figure 24), and the slope of the trend line that is generated (dU/da) is used to obtain the critical strain energy release rate (Jc) with Equation 2 (Mohammad et al., 2016).

$$Jc = -\left(\frac{1}{b}\right)\frac{dU}{da} \tag{2}$$

Where:

Jc = Critical strain energy release rate (kJ/m^2)
b = Sample thickness (mm)
a = Notch depth (mm)
U = Strain energy to failure (kN-mm)
dU/da = Change of strain energy with notch depth

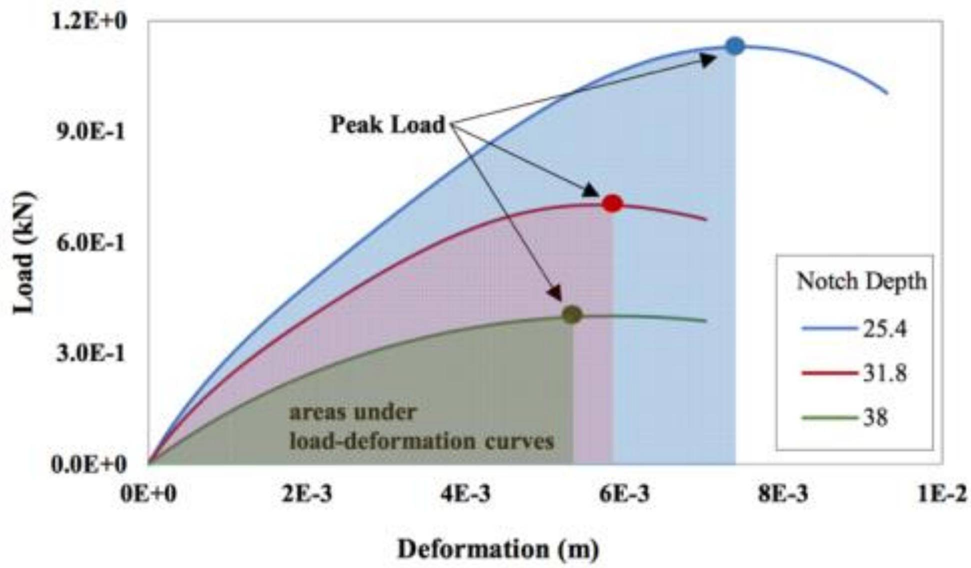

Figure 23. SCB load deformation curves at different notch depths (Mohammad et al., 2016)

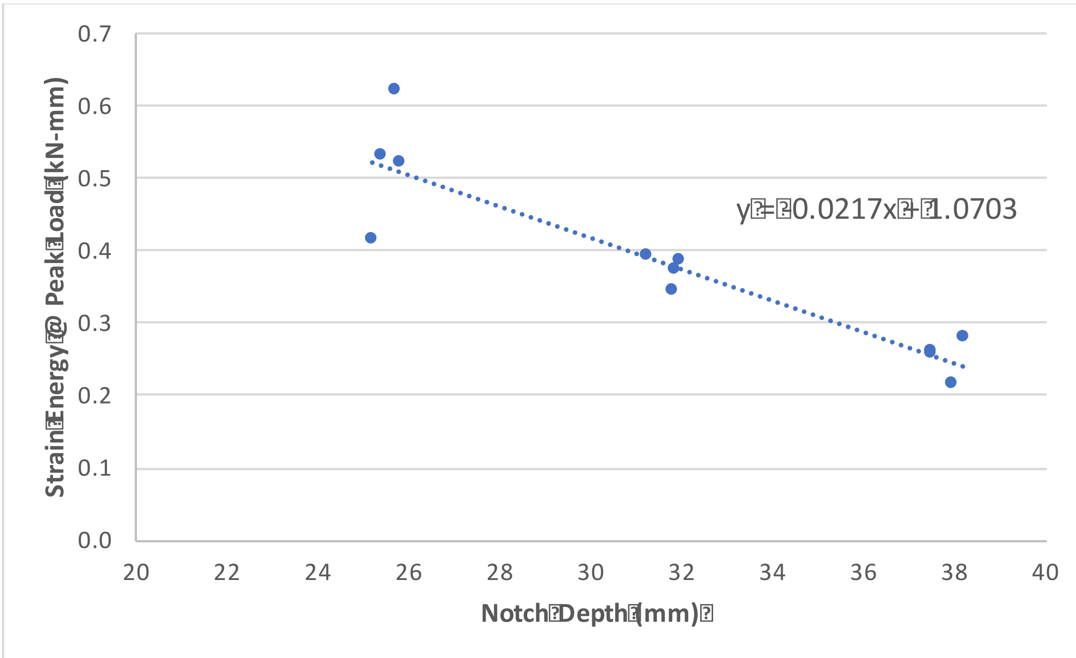

Figure 24. Fracture energy at different notch depths illustration

This SCB method has been used as a research tool in Louisiana since 2004. Kim et al. (2012) found good correlations of field cracking performance with Jc values obtained by SCB testing in existing pavements. This study also found that for laboratory produced samples, the SCB determined Jc had a relatively meaningful correlation with the Toughness Index (TI), which was obtained from IDT testing (Kim et al., 2012). In its standard specifications for road and bridge construction, the Louisiana Department of Transportation currently requires asphalt mixtures to meet a Jc threshold of 0.5 or 0.6 kJ/m^2, depending on the mixture type and traffic volume.

3.9 Illinois Flexibility Index Test (I-FIT)

The Illinois Flexibility Index Test (I-FIT), developed at the University of Illinois, is a SCB cracking test alternative for use in mixtures with higher proportions of RAP and RAS. The goal for this development was to have a simple, affordable, and reliable test that allows better control of the development of durable, crack- and rutting-resistant asphalt mixtures (Al-Qadi, 2015).

The concept of fracture energy was used in I-FIT testing to determine the cracking resistance of asphalt mixtures. The fracture energy is calculated by integrating the area under the load-displacement curve in the outputs and dividing it by the area through which the crack will propagate in the semi-circular specimen used for I-FIT testing. It was found, however, that at low temperature testing, this parameter did not show good correlation with the mixture performance (Ozer et al., 2016). To address this issue, a new parameter was introduced, called the Flexibility Index (FI). The FI is calculated with Equation 3 using the fracture energy and slope of the post-peak load portion of the curve, which was also found to be an indicator of mixture performance. Figure 25 shows typical results of I-FIT outputs, and the variables used to calculate the FI (Al-Qadi et al., 2015).

$$FI = \frac{G_f}{|m|} \times A \tag{3}$$

Where:

FI = Flexibility Index
G_f = Fracture Energy (J/m^2)
m = Post-Peak Slope (kN/mm)
A = Scaling Factor (0.01 for gyratory specimens)

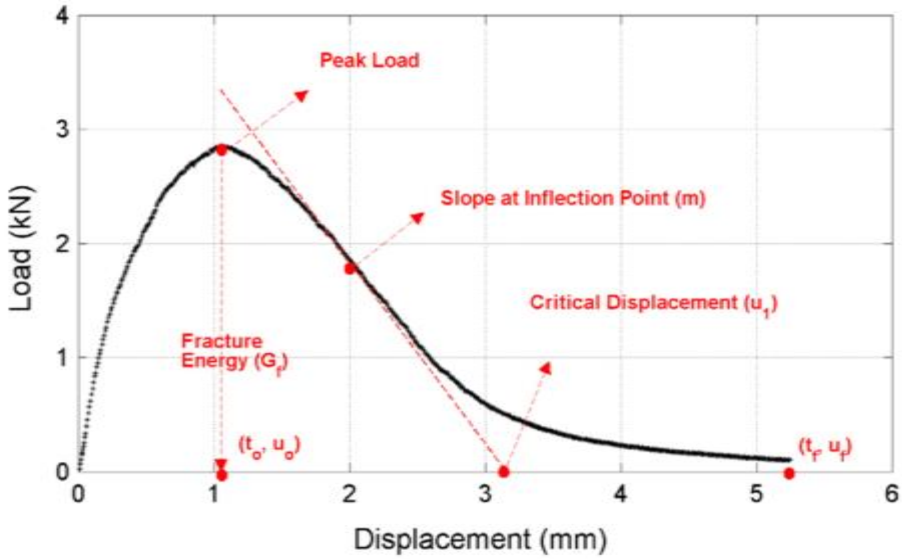

Figure 25. I-FIT parameters used for FI determination (Al-Qadi et al., 2015)

I-FIT testing was performed using a Material Test System (MTS) frame and controller, shown in Figure 26. Semi-circular testing specimens at an air void level of 7.0 ± 0.5% were obtained from larger SGC samples. These SGC samples were trimmed, and four semi-circular specimens could be obtained from each SGC sample. At the center of each semi-circular specimen, a 15 mm deep by 1.5 mm wide notch was trimmed. The testing temperature was 25.0 ± 0.5 °C, and the specimens were conditioned for two hours prior to I-FIT testing. A monotonic 50 mm/min load was applied in the specimens until the loads applied went below 0.1 KN after the peak was recorded. Force and displacement were recorded at a 50 Hz rate. From these results, the Fracture Energy and FI were calculated. A higher Fracture Energy usually represents better cracking resistance. The FI can be related to the type of failure, with higher FI values indicating ductile failures, and lower FI results representing brittle failures (Al-Qadi et al., 2015). The Illinois Department of Transportation requires a minimum FI of 8 for asphalt mixtures.

Figure 26. I-FIT test setup

3.10 Field Performance

The NCAT Test Track is an accelerated pavement testing facility (APT). It consists of a closed road loop divided into different 200 feet test sections where a fleet of loaded trucks apply wheel loads at controlled levels. This produces an accelerated accumulation of traffic-produced damage over a reduced time. For each research cycle, 10 million ESALs are applied by trucks at the Test Track, and pavement responses are constantly monitored.

At the Test Track, traffic is suspended each Monday and the pavement sections are measured for ride quality, rutting, and cracking performance. Ride quality is measured with a Dynatest inertial profiler that determines the international roughness index (IRI) of each section (Shown in Figure 27). Rutting is measured using the ALDOT beam procedure detailed in the ALDOT T-392 specification. This method uses a 4-foot beam with a dial gauge (Figure 28) to measure rut depths in each wheel path at predetermined locations in the sections. The reported rut depths consist of the average rut measurements for each section. The accuracy of the readings following the ALDOT beam method is estimated at ± 2.5 mm. For cracking performance, sections are inspected visually, and when observed, cracks are mapped and measured (Figure 29). Cracks are considered to have an area of influence that is 6 inches to each side. Linear measurements of the cracks are performed to determine the cracked area of the sections. With these measurements, the cracking percent of each section is then calculated.

Figure 27. Inertial profiler van used to assess ride quality

Figure 28. ALDOT beam used for field rut depth measurements

Figure 29. Mapped cracks on Test Track section

The measured distresses were used to rate the condition of the pavement sections based on the ratings scale recommended by the Federal Highway Administration (FHWA) in the code of federal regulations 23 490.313 "Calculation of Performance Management Measures". This method rates the pavement condition in each of the distress categories. Table 9 shows the condition thresholds for each of the distresses (FHWA, 2016).

Table 9. Pavement condition thresholds (FHWA, 2016)

Distress	**Condition**		
	Good	**Fair**	**Poor**
Ride quality (IRI – m/km)	< 1.5	1.5 - 2.7	> 2.7
Rut depth (mm)	< 5	5 - 10	> 10
Cracking area (%)	< 5	5 - 20	> 20

CHAPTER 4 – RESULTS AND DISCUSSION

4.1 Binder Tests

Results of binder tests are shown in Table 10. The blended binder with Delta-S extracted from the N7 mixture had a high PG grade of 94 ^{o}C, two grades higher than the S5 binder and one grade higher than the N1 binder. From these results, all the mixtures had a high PG over 76, which is typically required in the southern states for a good rutting performance of the mixture. The low PG grade of the extracted binder in the N7 Delta-S mixture was -10^{o}C, while the S5 binder was graded as -22 ^{o}C, with -16 ^{o}C and -4 ^{o}C for N1 and N8 binders, respectively. Only the S5 binder met the low temperature grade of -22 ^{o}C commonly used for asphalt mixtures in the southern states. From these results, the S5 mixture would be expected to have better performance to thermal cracking. All the binders extracted had a ΔT_c below the recommended -5 °C, meaning that the mixtures could be susceptible to non-load related cracking. The binder from sections N7 had a similar ΔT_c as N1 and S5 at approximately -10°C, while the binder from Section N8 had a ΔT_c that was considerably lower at -20°C, making this the most susceptible to block cracking or weathering distresses of all the mixtures compared in this study.

Table 10. Extracted binder analysis results

Mixture	**Virgin PG**	**Extracted PG**	**ΔT_c**
N1 20% RAP	64-22	88-16	-9.4
N7 35% RAP + Delta-S	64-22	94-10	-10.1
N8 20% RAP + 5% RAS	64-22	106-4	-20.0
S5 35% RAP	58-28	82-22	-9.3

4.2 Hamburg Wheel Tracking Test

Results from Hamburg Wheel Tracking tests of the N7 surface mixture are shown in Figure 30, which shows the outputs for the two replicates tested, and summarized in Table 11. The results gave an average stripping inflection point for the mixture at 15,000 passes, which exceeds the 10,000 passes proposed for stripping resistant mixtures. The rutting performance was also analyzed for the Hamburg test results. The N7 mixture, with 35% RAP and Delta-S had a blended PG 94-10 binder according to the binder test results (Table 10). For a mixture with this binder grade, a minimum 20,000 passes to reach the critical 12.5 mm rut in the Hamburg test are required to be considered resistant to rutting. The N7 specimens took an average 19,200 passes to reach 12.5 mm rut depth, barely missing the rutting threshold of 20,000 passes. Based on the Hamburg test results, the N7 mixture with Delta-S can be expected to have good resistance to moisture-induced damage, but its resistance to rutting might not be optimal.

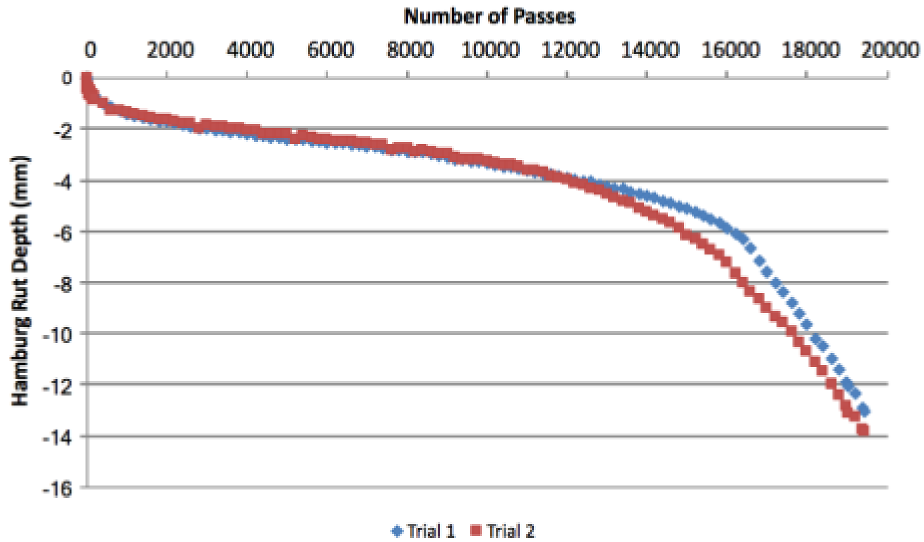

Figure 30. Hamburg results output

Table 11. Summary of Hamburg results for N7 mixture

Parameter	**Result**	**Criteria**	**Pass/Fail**
Rut Depth at 10,000 passes (mm)	3.29	N/A	N/A
Rut Depth at 20,000 passes (mm)	13.41	N/A	N/A
Passes to 12.5 mm rut	19,200	20,000	Fail
Stripping Inflection Point	15,000	10,000	Pass

4.3 Dynamic Modulus

The mastercurves obtained from dynamic modulus testing at different frequencies and temperatures are shown in Figure 31. The fitting statistics for the generated mastercurves are shown in Table 12, and Table 13 gives a summary of the regression coefficients. The fitting statistics Se/Sy and R^2 indicated an excellent fit between measured and predicted data for all the mixtures. Based on the Gamma factor, which describes the steepness of the curves, the N8 mixture had the lowest slope, meaning this mixture was less susceptible to changes in temperature/frequency. The N1 mixture had the highest slope, meaning it was the most susceptible of the mixtures to changes in temperature/frequency.

Table 12. Mastercurves goodness of fit parameters

Mixture	**R^2**	**Se/Sy**
N1 20% RAP	0.997	0.036
N7 35% RAP + Delta-S	0.997	0.036
N8 20% RAP + 5% RAS	0.999	0.023
S5 35% RAP	0.997	0.042

Table 13. Mastercurves coefficients

Mixture	**Max E* (ksi)**	**Min E* (ksi)**	**Beta**	**Gamma**	**EA**
N1 20% RAP	3159.19	8.36	-0.989	-0.510	201423.2
N7 35% RAP + Delta-S	3125.02	5.33	-0.997	-0.441	219584.8
N8 20% RAP + 5% RAS	3148.88	5.60	-1.463	-0.395	216839.5
S5 35% RAP	3106.46	7.59	-0.600	-0.440	204042.7

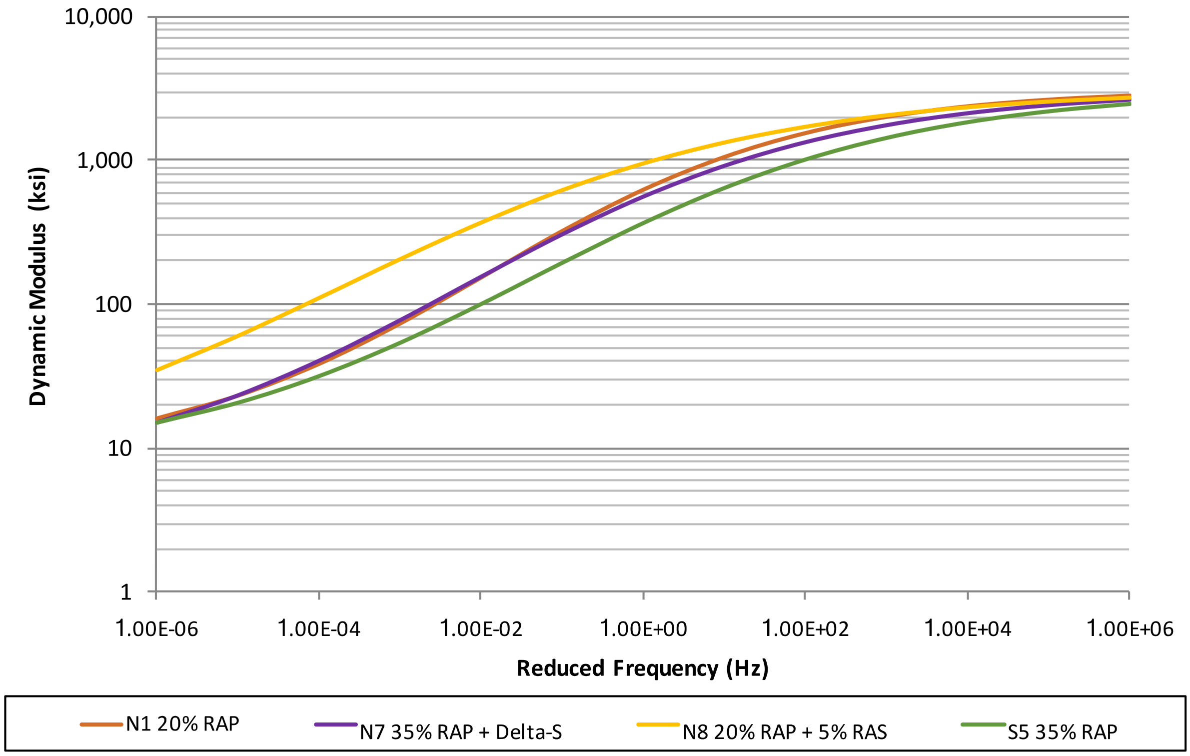

Figure 31. Mastercurves of analyzed mixtures

The generated mastercurves divided the mixtures into three distinct groups. The N7 mixture with Delta-S and 35% RAP had a mastercurve that was close to the one from N1, which had 20% RAP content. The N8 mixture, with 20% RAP and 5% RAS, was the stiffest mix. The softest mixture was found to be the one in Section S5. This mixture had a similar gradation and 35% RAP content as the N7 mix. The softness in the S5 mixture could be attributed to the fact that it had a virgin binder with a lower PG grade (PG 58-28), which produced a PG 82-22 blended binder, while N7 had a PG 64-22 virgin binder with Delta-S that produced a PG 92-10 blended binder. The mastercurves for the N1 and N7 mixtures were located between that of the softer S5 and the stiffer N8 mixtures. On the high temperature-low frequency side, the mastercurves for

N1 and N7 were closer to that of S5. On the low temperature-high frequency side, however, the mastercurves of N1 and N7 tended to be closer to the N8 mixture. These results indicate that dropping the binder grade in the S5 mixture design had a higher softening effect than what was achieved by using Delta-S in the N7 mixture. The addition of Delta-S lowered the N7 mixture stiffness similar to that of N1, which had a 15% less RAP content.

4.4 Energy Ratio (ER)

The FE, $DCSE_{HMA}$, $DCSE_{Min}$, and ER of the mixtures obtained from the Energy Ratio tests are shown in Figure 32 through Figure 35. The trends shown in the FE and $DCSE_{HMA}$ were similar. The $DCSE_{Min}$ results varied slightly, and the ER trends were different from those of the other properties. Based on the FE and $DCSE_{HMA}$ results, the highest resistance to top-down cracking was from the S5 mixture, followed by N1, N7, and N8. The only mixture to meet the recommended $DCSE_{HMA}$ range of 0.75 – 2.5 kJ/m^3 was N8, while the others exceeded that range. In the $DCSE_{Min}$ plot, the N1 result was slightly higher than the S5 result, while the other two mixtures showed lower results. ER results showed a completely different trend, where the mixture with the best top-down cracking resistance was N8, which was the worst ranked in the other tests. This can be explained by the low $DCSE_{Min}$ results obtained from the mixture, which are inversely proportional to the ER values. The $DCSE_{Min}$ is affected by creep compliance parameters, and in this test the N8 results were very low due to its high stiffness. All the mixtures exceeded the minimum ER criteria established in Table 8 for any traffic level. Table 14 shows the results of the fracture properties from the ER evaluation. From the results in all the parameters evaluated in this procedure, the N7 mixture with Delta-S would be expected to have lower top down cracking resistance than the S5 mixture, which was designed with a similar gradation and RAP contents but with a softer virgin binder and no rejuvenator.

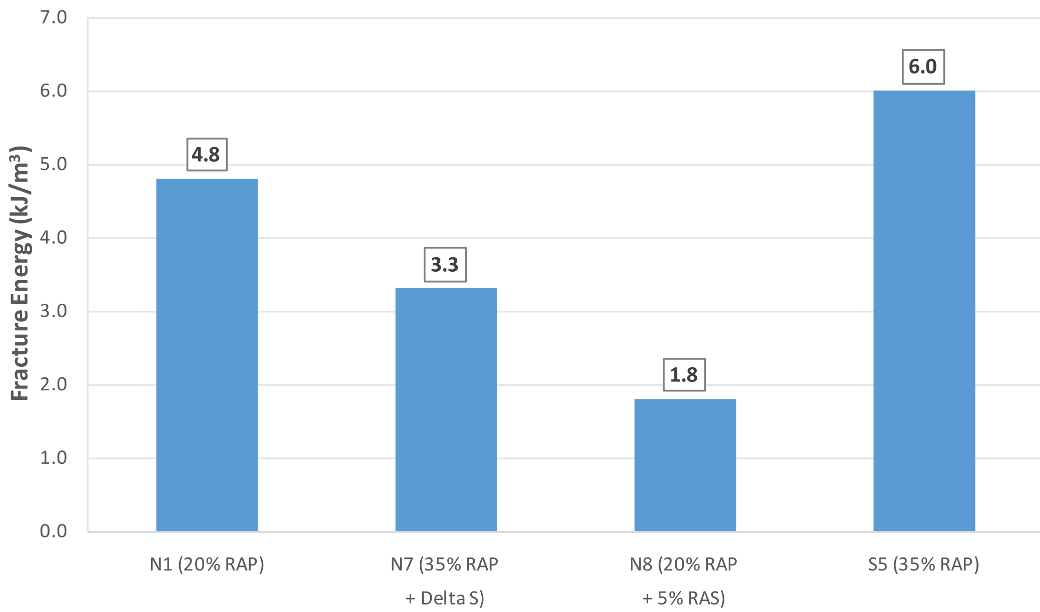

Figure 32. Fracture Energy

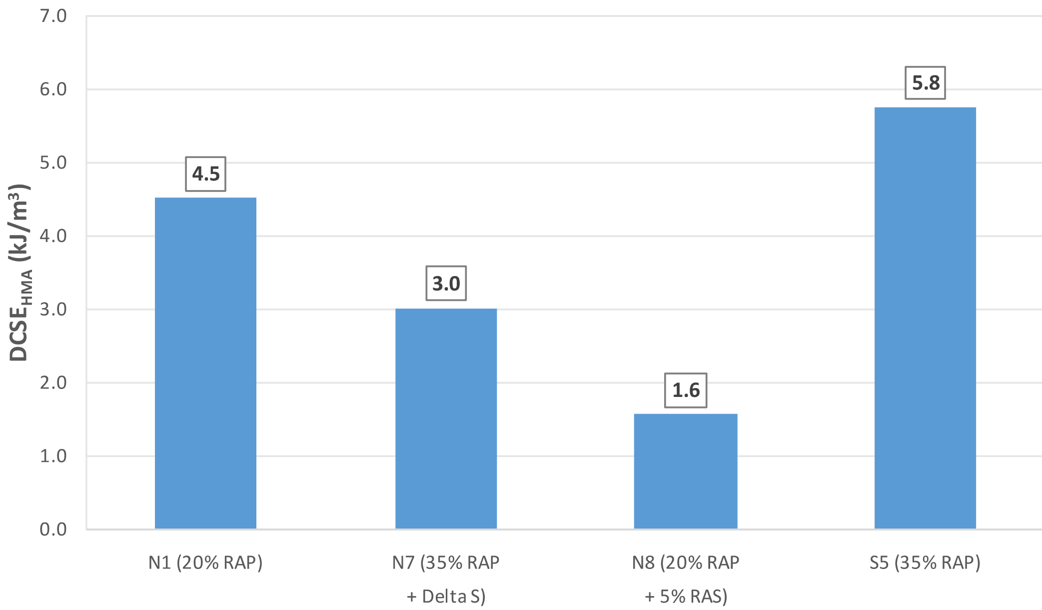

Figure 33. Dissipated creep strain energy at failure

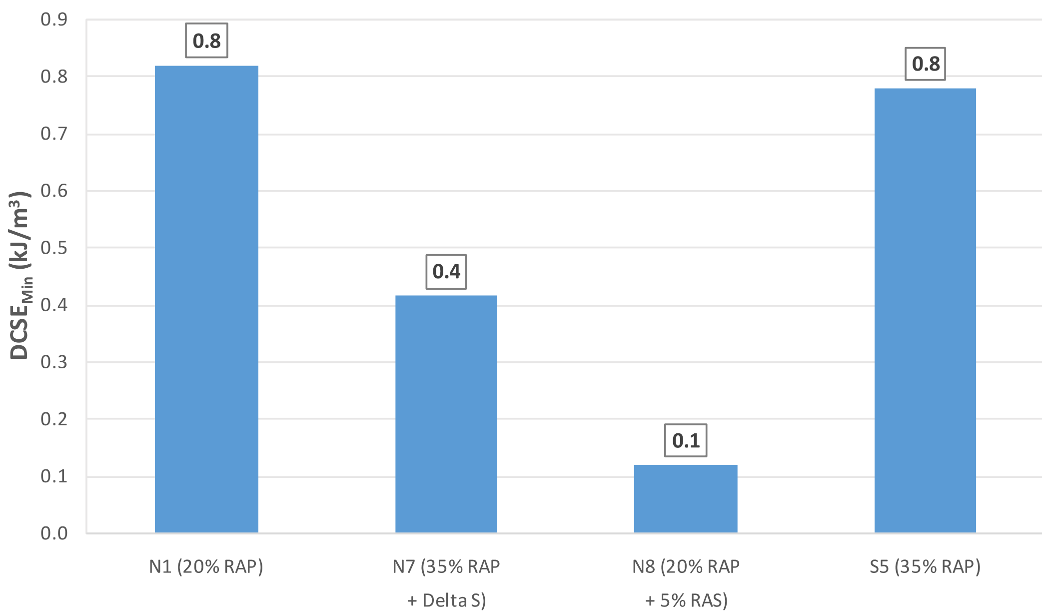

Figure 34. Minimum dissipated creep strain energy

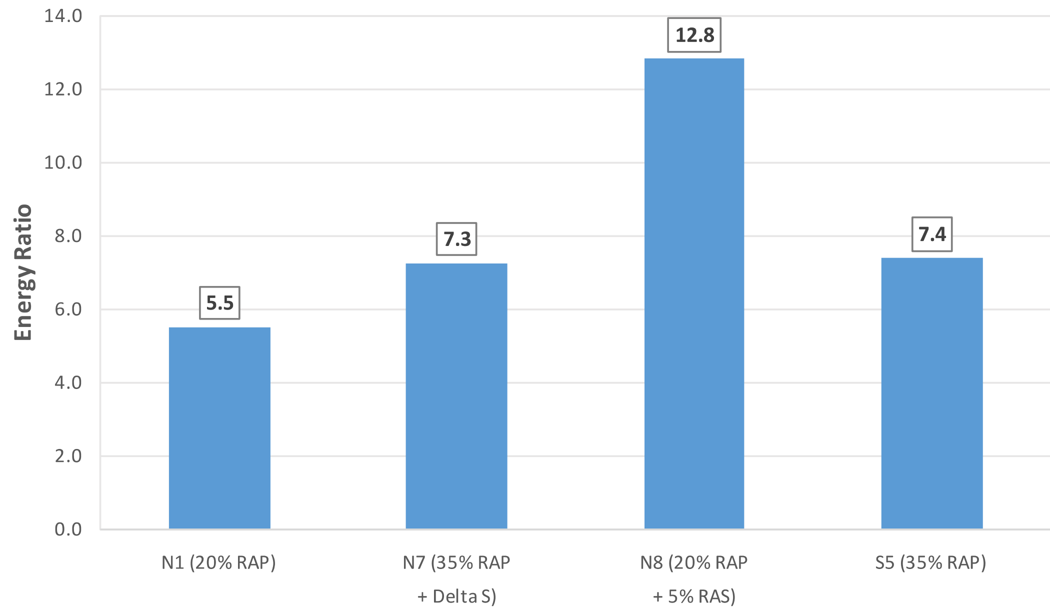

Figure 35. Energy ratio

Table 14. ER fracture properties of mixtures analyzed

Mixture	**FE (kJ/m^3)**	**$DCSE_{HMA}$ (kJ/m^3)**	**$DCSE_{Min}$ (kJ/m^3)**	**ER**
N1 20% RAP	4.8	4.5	0.8	5.5
N7 35% RAP + Delta-S	3.3	3.0	0.4	7.3
N8 20% RAP + 5% RAS	1.8	1.6	0.1	12.8
S5 35% RAP	6.0	5.8	0.8	7.4

4.5 Texas Overlay Test (OT)

The Texas Overlay Test results are shown in Figure 36, with the error bars representing the standard deviation from the tests in each mixture. Results showed that the best performance was from the S5 mixture with an average 61 cycles to failure. It was followed by the N1 mixture, which failed at an average 25 OT cycles. The N7 mixture, with 35% RAP and Delta-S, failed at an average 10 loading cycles. Finally, the N8 mixture was the one that failed earliest in the testing cycles. In all replicates tested for the N8 mixture, the failing point was reached at the second loading cycle. None of the mixtures reached the minimum 300-cycle OT criterion.

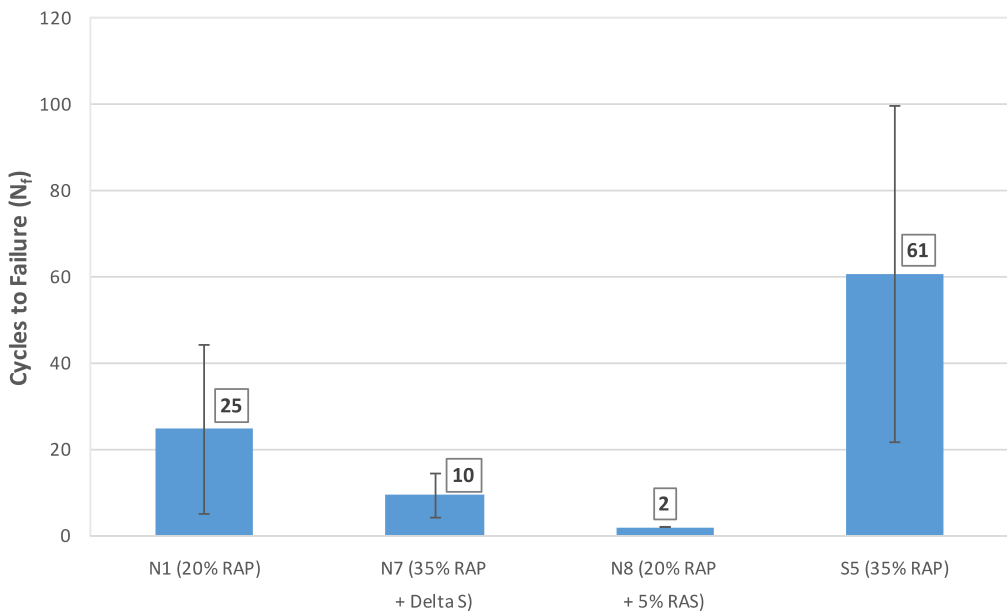

Figure 36. Texas OT results

On average, the variability for all the data sets was very high. The S5 mixture, which had the apparent best performance, had an average N_f of 61 with a standard deviation of 39, which overlapped with the results from N1, with an N_f of 25 and a standard deviation of 19.5. This also happened when comparing the results for N7 and N1. A Tukey-Kramer statistical comparison with a 95% confidence interval was used to rank and group the Texas OT results. Table 15 shows the results ranked from best to worst performance in Texas OT according to the statistical analysis, with their standard deviation and coefficient of variability (CV). Also shown are the statistical groupings found for the results. Mixture N1 was in Group A with S5 and in Group B with N7 and N8 due to the high variability of its results. The Delta-S mixture in N7 had statistically lower results than the other 35% RAP S5 mixture, meaning the Delta-S inclusion in the virgin PG 64-22 was not as effective in this test as using a softer PG 58-28 virgin binder. The results from N7 were found to be statistically similar to those from N8, which had the worst performance in this test.

Table 15. Texas OT results and ranking of performance

Mixture	N_f	Standard Deviation	CV (%)	Grouping
S5 35% RAP	61	39.0	64	A
N1 20% RAP	25	19.5	79	A, B
N7 35% RAP + Delta-S	10	5.1	53	B
N8 20% RAP + 5% RAS	2	0	0	B

* Means that do not share a letter are statistically different

4.6 NCAT-Modified Overlay Test (NCAT-OT)

The NCAT-Modified Overlay Test results are shown in Figure 37, with the error bars representing the standard deviation from the results for each mixture. The trends and rankings of performance for the mixtures were similar to Texas OT results. The S5 mixture had the best performance with the highest number of cycles to failure. It was followed by N1 and the Delta-S N7 mixture. Finally, the N8 mixture had the worst NCAT-OT performance.

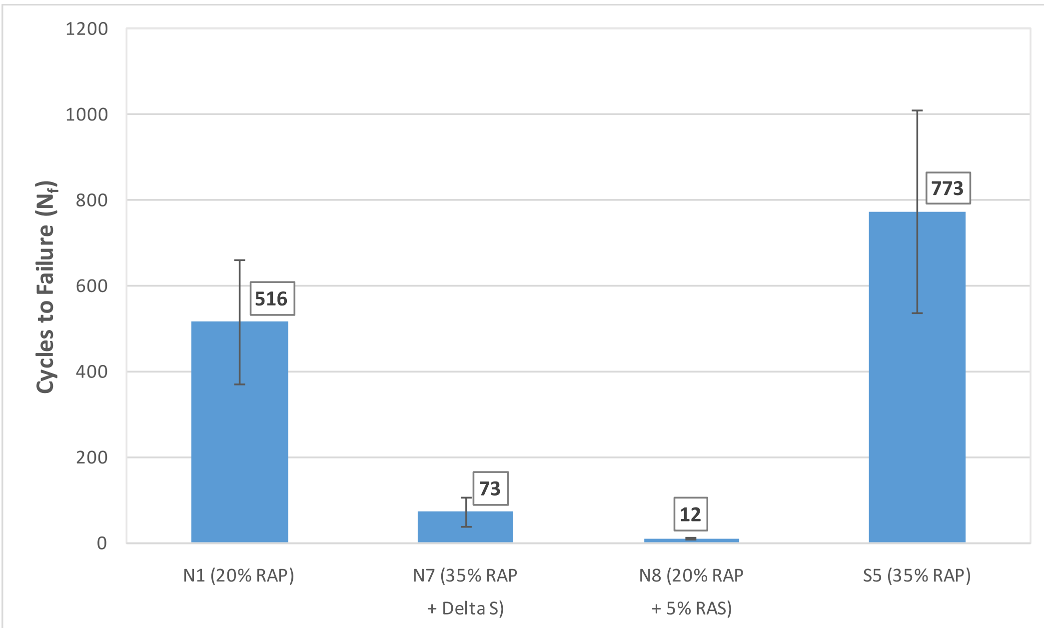

Figure 37. NCAT-OT results

A Tukey-Kramer statistical analysis with a 95% confidence interval was done to rank and group the NCAT-OT results. Table 16 shows the results of this analysis ranked from best to worst, with standard deviation and CVs also shown. The variabilities in these results were considerably lower than those in Texas OT, with an average CV of 31%. The statistical groupings paired the S5 and N1 results together with better performance and no significant difference in their means. N7 and N8 results were also grouped together with no significant difference and a lower performance. The N7 mixture with Delta-S had once again statistically worse performance in this test than the other 35% RAP mixture in S5, and its results were statistically similar to N8, which had the worst NCAT-OT performance.

Table 16. NCAT-OT results and ranking of performance

Mixture	**N_f**	**Standard Deviation**	**CV (%)**	**Grouping**
S5 35% RAP	773	235.0	30	A
N1 20% RAP	516	146.0	28	A
N7 35% RAP + Delta-S	73	34.1	47	B
N8 20% RAP + 5% RAS	12	2.1	17	B

* Means that do not share a letter are statistically different

4.7 Semi Circular Bend (LTRC)

Results from the Semi Circular Bend (SCB) tests performed following the Louisiana Transportation Research Center (LTRC) procedure are shown in Figure 38. The Jc values obtained from the SCB tests were very close, and they showed a different trend than the other cracking tests. None of the mixtures reached the Jc threshold required in the Louisiana specifications. Overall, these results indicated that all the mixtures would not have a good cracking resistance.

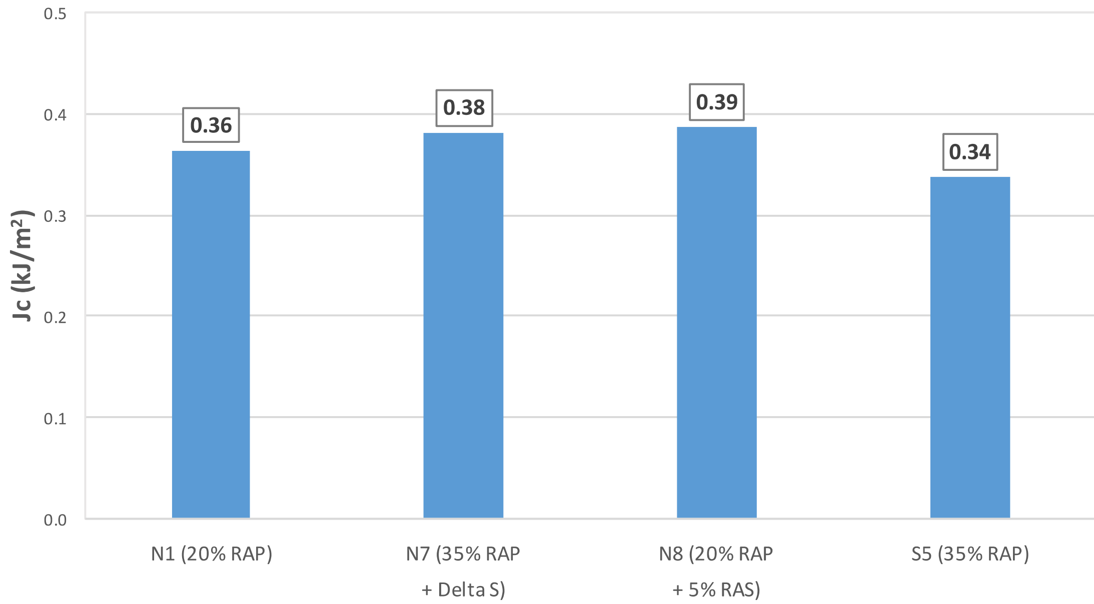

Figure 38. SCB results

4.8 Illinois Flexibility Index Test (I-FIT)

I-FIT results are shown in Figure 39, with the error bars representing the standard deviations. Similar trends were observed as in Texas and NCAT-Modified Overlay Tests and the mixtures ranked in the same order, although the variabilities were less significant. The best performance was found in the S5 mixture, which had the same gradation and RAP contents as N7 but with a softer virgin binder. The N7 mixture with Delta S had results that were close to N1. The mixture with the worst I-FIT performance was N8. None of the mixtures met the Illinois minimum FI criterion of 8.

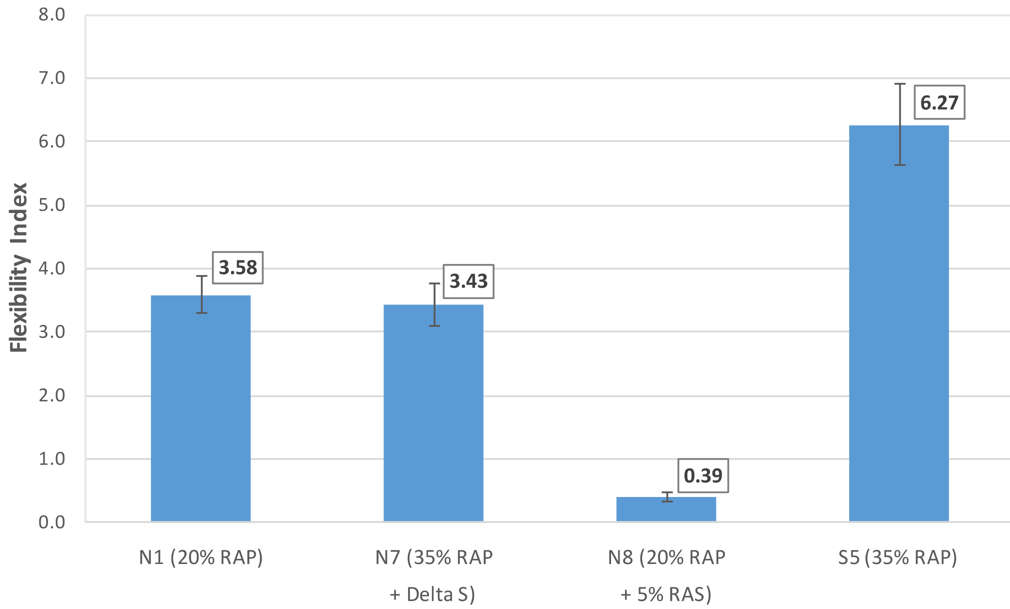

Figure 39. I-FIT results

Table 17 shows the I-FIT test results and standard deviations, as well as CVs and statistical groupings from a Tukey-Kramer statistical analysis and a 95% confidence interval. The variabilities of these results were significantly lower than those of the OT results, with an average CV of 12%. In the statistical analysis, the S5 mixture was placed in Group A with the best I-FIT performance, while a middle tier Group B consisted of the N1 and N7 mixtures with no significant difference in their means, and the N8 mixture once again with the worst performance in Group C. In comparing both 35% RAP mixtures, the N7 mixture with Delta-S had statistically lower I-FIT results than those of the S5 mixture. Based on the I-FIT results, however, the performance of N7 was statistically similar to N1, which had less RAP contents. Performance of N7 was statistically better than N8, which had the worst performance of all the mixtures tested.

Table 17. I-FIT results and ranking of performance

Mixture	**FI**	**Standard Deviation**	**CV (%)**	**Grouping**
S5 35% RAP	6.27	0.65	10	A
N1 20% RAP	3.58	0.30	8	B
N7 35% RAP + Delta-S	3.43	0.32	9	B
N8 20% RAP + 5% RAS	0.39	0.07	18	C

* Means that do not share a letter are statistically different

4.9 Field Performance

Field performance data from the four sections were obtained from the Test Track. When this information was collected, the total traffic at the Test Track from the beginning of the 2015 Research Cycle was estimated to represent 8.6 million ESALs. A full testing cycle is expected to apply 10 million ESALs to the pavement sections. Records of field measured ride quality and rut depth were obtained at 8.3 million ESALs, while the data obtained for field cracking measurements were at 8.6 million ESALs.

4.9.1 Ride Quality

Field measurements of ride quality are shown in Figure 40, expressed in IRI. For Section N7, the segmented section represents the first stages when the surface mixture presented performance issues that required a redesign and reconstruction. From these results, all the sections have relatively low IRI readings, indicating a good ride quality. Since its repave, Section N7 with

Delta-S had stable IRIs that were at first higher than S5 and N8, but lower than N1. Currently, the section with the highest IRI is N1 with 1.1 m/km (70 in/mile), which would still be considered a good condition in the PCI scale recommended by FHWA. Section N8 had the lowest IRI readings during the first half of the current cycle but showed a slight trend to increase and currently has an IRI that is equal to N7. Starting at approximately 5 million ESALs, Section S5 had the best ride quality with the lowest measured IRIs by a small margin.

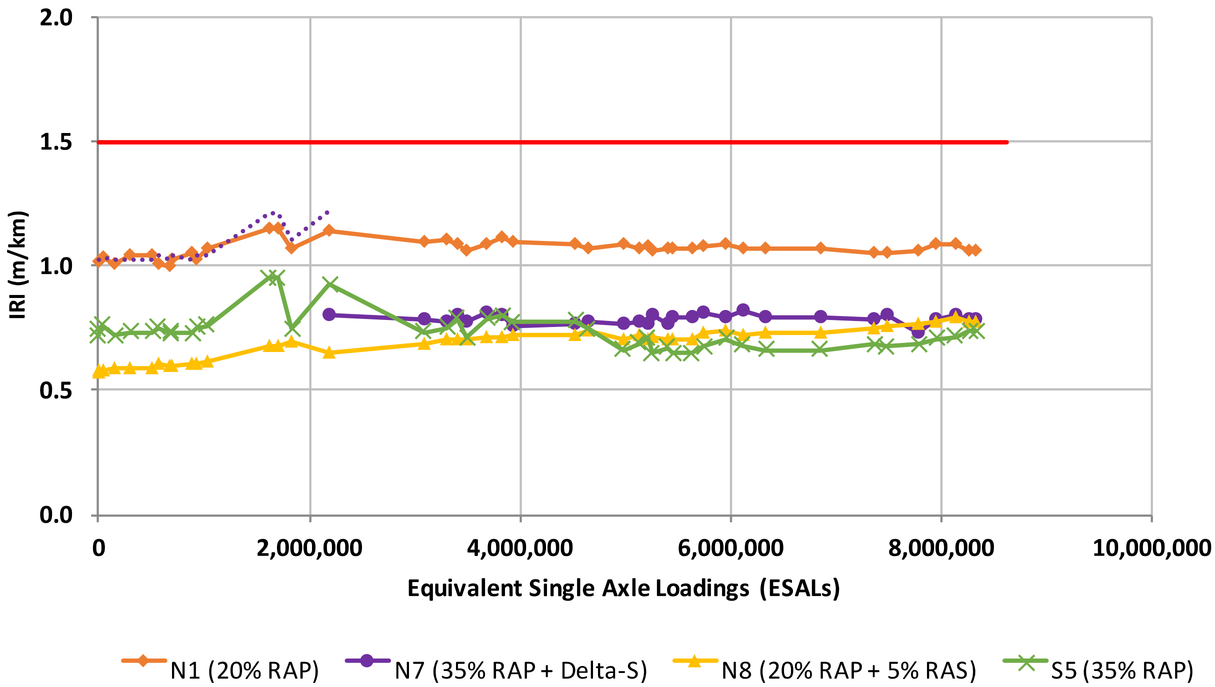

Figure 40. Roughness measurements

4.9.2 Rutting

The field measured rut depths are shown in Figure 41. From the data collected, all the sections had rutting measurements under 6 mm, which is considered low severity rutting according to the

ALDOT-392 specifications. Sections N8 and S5 had the lowest rut depths measured, with sections N7 and N1 having slightly higher rutting. If the ± 2.5 mm accuracy of the data collection method is taken into consideration, the differences in field measured rutting between the sections could be negligible. Overall, all the sections have shown good rutting performance and would be classified as good condition according to the FHWA recommended rating scale.

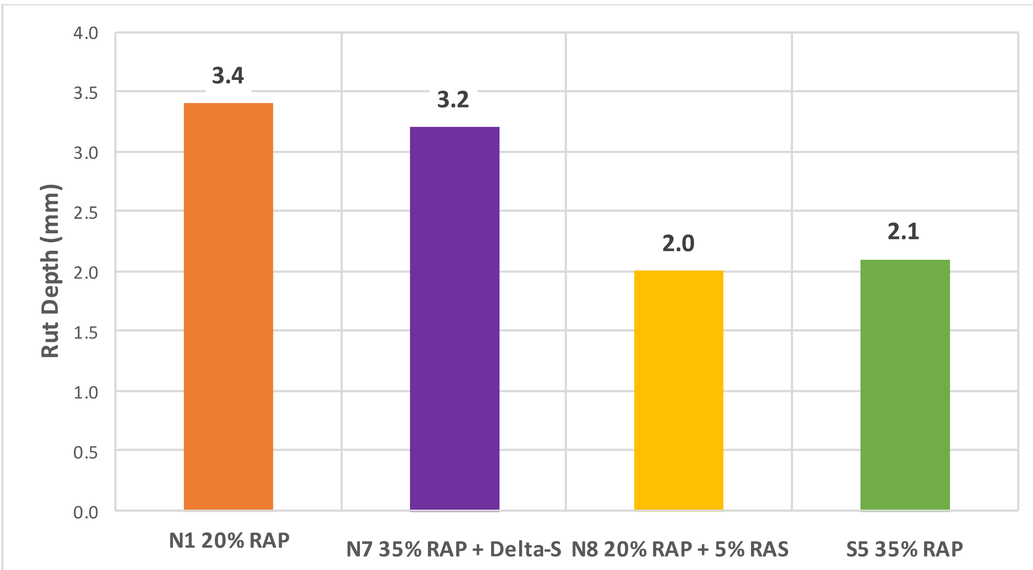

Figure 41. Rutting measurements

4.9.3 Cracking

Based on the field cracking measurements, shown in Figure 42, the mixture with the best performance is S5, which after 8 million ESALs has not had any cracking. The appearance of cracking in the Delta-S Section N7 and N1 were noticed shortly after 6 million ESALs. These cracks were attributed to constructive defects and not considered load-related. The cracking density did not increase for these two sections after these occurrences. Section N8, which was built with the stiffest mixture, started to have cracks appearing at 5 million ESALs, and the cracking density has been steadily increasing since then. Still, the cracking measured in all the sections is within the ranges that would be classified as good cracking performance in the FHWA recommended PCI scale.

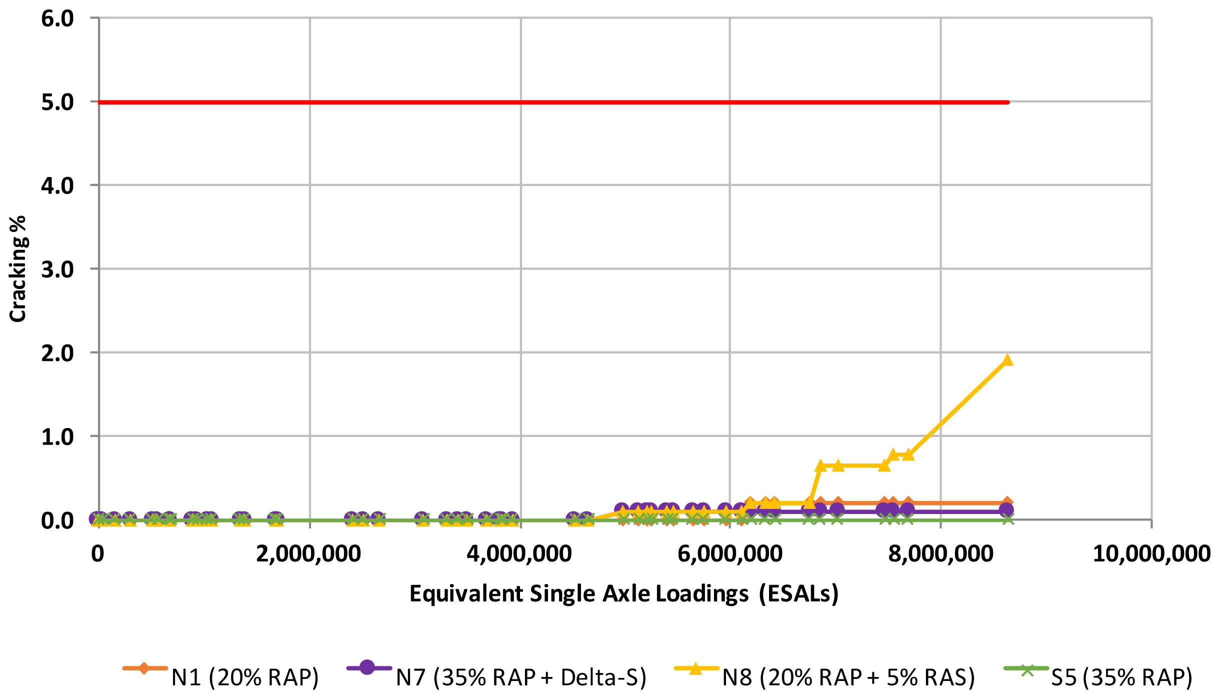

Figure 42. Field cracking measurements

4.10 Summary of Results

Results from the extracted binder tests and the recently recorded field measurements of IRI and rut depth are shown in Table 18. Results from the laboratory cracking tests performed on these mixtures and the recently recorded field cracking measurements are shown in Table 19. Also shown are the ΔT_c data from the extracted binders. The results for the laboratory cracking tests are compared in Figure 43. Note that for laboratory tests, higher results indicate better cracking performance. Observations made from the results are presented below.

Table 18. Extracted binder, field measured IRI and rutting results

Mixture	Extracted PG	IRI (m/km)	Rutting (mm)
N1 20% RAP	88-16	1.1	3.4
N7 35% RAP + Delta-S	94-10	0.8	3.2
N8 20% RAP + 5% RAS	106-4	0.8	2.0
S5 35% RAP	82-22	0.7	2.1

Table 19. Laboratory and field cracking performance

Mixture	Laboratory Tests						Field Crack (%)
	ΔT_c	ER	Texas OT (Nf)	NCAT OT (Nf)	SCB (Jc)	I-FIT (FI)	
N1 20% RAP	-9.4	5.5	25 (A-B)	556 (A)	0.36	3.58 (B)	0.2
N7 35% RAP + Delta-S	-10.1	7.3	10 (B)	73 (B)	0.38	3.43 (B)	0.1
N8 20% RAP + 5% RAS	-20.0	12.8	2 (B)	12 (B)	0.39	0.39 (C)	1.9
S5 35% RAP	-9.3	7.4	61 (A)	773 (A)	0.34	6.27 (A)	0.0

* Letters next to Texas OT, NCAT OT, and I-FIT results represent groupings from statistical analysis

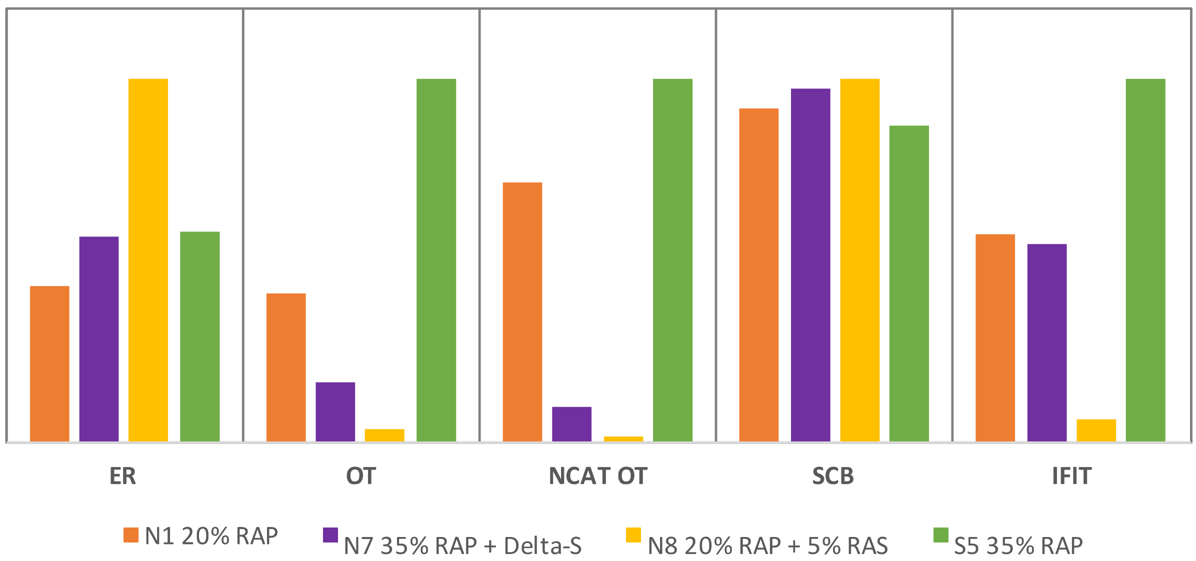

Figure 43. Comparison of laboratory cracking performance

- From the binder test results, the high performance grade of the blended binders would indicate that all the mixtures would have a good rutting performance.
- The ΔT_c parameter suggests N8 as the mixture with the highest potential for non-load related cracking. N7, N1, and S5 showed a similar performance in this regard. None of the binders met the ΔT_c criterion of -5°C for non-load related cracking resistance.
- The dynamic modulus mastercurves showed that N8 was the stiffest mixture. On the high temperature/low frequency side of the mastercurves, which could be used for predicting rutting potential, S5 was the softest mixture with the highest rutting potential, followed by N7 and N1 which had similar performance. N1 and N7 had mastercurves that were very close on both the high temperature/low frequency and low temperature/high frequency sides.

- The ER results indicated that the mixture with the highest ER and the best resistance to top down cracking would be N8, followed by N7 and S5 with close results, and finally N1. These results can be misleading because creep compliance results were extremely low for N8, and the parameters obtained from this test are inversely proportional in ER calculations.
- Based on the Overlay Tests using the Texas and NCAT-modified methods, the trends were similar, but the variabilities were lower for the NCAT-modified method. Both these tests placed S5 as the mixture most resistant to cracking, followed by N1, N7, and finally N8.
- SCB results determined based on the LTRC method from all the mixtures were close, with N8 as the most resistant to cracking, followed by N7, N1, and S5.
- I-FIT results showed the same trends as those of Overlay Tests, with the mixtures ranked in the same order. The variabilities were lower for I-FIT results and three different tiers were formed from the statistical analysis, with S5 as the mixture with the best performance, N1 and N7 in the middle tier, and N8 in the tier representing the worst cracking performance.
- The IRI measurements showed that all the pavement sections had a good ride quality, with Section N1 having a slightly lower IRI than S5, N7, and N8.
- All the sections have shown good rutting performance that could be considered similar based on the accuracy of the data collection method. Low severity rutting has been observed in every section.

- Field measured cracking ranked S5 with the best performance and no cracking observed. N7 and N1 had very low cracking densities that were attributed to construction defects. Section N8 had the worst cracking performance, and is currently on a trend where cracking density is steadily increasing.

CHAPTER 5 – CONCLUSIONS AND RECOMMENDATIONS

This study consisted of an evaluation of field and laboratory performance of the surface asphalt mixture from Section N7 of the NCAT Test Track. This section was designed with 35% RAP and a PG 64-22 virgin binder with a bio based rejuvenator called Delta-S. The performance of this mixture was compared with surface mixtures of three other sections with different mixture characteristics. The other mixtures used for this evaluation were from Section N1 (20% RAP PG 64-22), Section N8 (20% RAP 5% RAS PG 64-22), and Section S5 (35% RAP PG 58-28). Performance tests were used to determine the stiffness and cracking resistance of all the mixtures. These results were compared with the recently recorded ride quality, rutting, and cracking measurements at the Test Track. Based on the results obtained, the following conclusions can be made:

- From the extracted binder test results, the N7 mixture with Delta-S would be expected to have a good rutting performance.
- The N7 mixture had good resistance to moisture induced damage according to Hamburg test results, but failed to meet the rutting criterion by a small margin.
- Delta-S did not soften the 35% RAP mixture as much as a softer binder (PG 58-28) did. The N7 mixture (with 35% RAP and Delta-S) had similar stiffness to the N1 mixture (with 20% RAP).
- The extracted binder and dynamic modulus test results indicated that Delta-S would not have a negative effect on rutting performance for the 35% RAP mixture.

- The pavement section of the Test Track built with Delta-S had a ride quality that would be classified as good condition in the PCI scale recommended by the FHWA.
- Based on the field rutting measurements, Delta-S did not produce any negative effects on rutting performance. Low severity rutting has been recorded for all the sections after approximately 8 million ESALs.
- From the ΔT_c obtained from extracted binders, the N7 mixture with Delta-S had the same resistance to non-load related cracking as S5, made with a softer binder, and N1, which had lower RAP contents.
- The laboratory cracking test that ranked the mixture cracking performance similar to the field performance was the Illinois Flexibility Index Test, where S5 had the best test result, followed by N1 and N7 with statistically similar results, and N8 as the mixture with the worst performance.
- The field cracking performance of Section N7 with Delta-S could be considered equal to N1. A stiffer virgin binder or higher RAP contents would normally make the N7 mixture more susceptible to cracking, but this was not reflected in the field, which can be attributed to the effect of Delta-S.
- High RAP mixtures with Delta-S can have good cracking performance at traffic levels up to 6 million ESALs.
- Overall, Delta-S did not produce any negative effects on the ride quality, rutting, or cracking performance of the mixture, and could be considered a viable alternative in the design and production of asphalt mixtures with high RAP contents.

REFERENCES

Advanced Asphalt Technologies (2011). NCHRP Report 673: A Manual for Design of Hot Mix Asphalt with Commentary. Washington, DC.

Al-Qadi, I; Elseifi, M.; and Carpenter S. (2007). Reclaimed Asphalt Pavement - A Literature Review. Illinois Center for Transportation Report FHWA-ICT-07-001. University of Illinois at Urbana-Champaign. Urbana, IL.

Al-Qadi, I.; Ozer, H..; Lambros, J.; El Khatib, A.; Singhvi, P.; Khan, T.; Rivera-Perez, J.; and Doll, B. (2015) Testing protocols to ensure performance of high asphalt binder replacement mixes using RAP and RAS. Illinois Center for Transportation Report FHWA-ICT-15-017. University of Illinois at Urbana-Champaign. Urbana, IL.

Anderson, M.; King, G.; Hanson, D; and Blankenship, P. (2011) Evaluation of the Relationship between Asphalt Binder Properties and Non-Load Related Cracking. Journal of the Association of Asphalt Paving Technologists, Vol. 80, 615-663.

Birgisson, B.; Wang, J.; and Roque R. (2006). Implementation of the Florida Cracking Model into the Mechanistic-Empirical Pavement Design. UF Report 0003932. University of Florida. Gainesville, FL.

Brown, R.; Kandhal, P; Roberts, F; Kim, R.; Lee, D.; and Kennedy, T. (2009) Hot Mix Asphalt Materials, Mixture Design and Construction, 3rd Ed. NAPA, Lanham, MD, 2009.

Carpenter, S. and Wolosick, J. (1980) Modifier Influence in the Characterization of Hot-Mix Recycled Material. Transportation Research Record 777. Transportation Research Board. Washington, DC.

Chen, D. (2008). Field Experiences with RDD and Overlay Tester for Concrete Pavement Rehabilitation. ASCE Journal of Transportation Engineering, Volume 134, Issue 1. Austin, TX.

Collaborative Aggregates (2017) PG and Viscosity of Rejuvenated Recycled Binder. Accessed on September 15th, 2017. (http://collaborativeaggregates.com/wp-content/uploads/2017/03/Delta-S_PG_Viscocity_3-2017.pdf)

Ezree, M.; Zamhari, K.; Buhari, R.; Nayan, M.; and Hainin M. (2013). Short Term and Long Term Aging Effects of Asphalt Binder Modified with Montmorillonite. Key Engineering Materials Volumes 594-595, pp 996-1002. Trans Tech Publications. Switzerland.

Grilli, A., Bocci, M.; Cardone, F.; Conti, C.; and Giorgini, E. (2013). Laboratory and In-Plant Validation of Hot Mix Recycling Using a Rejuvenator. International Journal of Pavement Research and Technology, Vol. 6, Issue 4.

Hansen, K. and Copeland, A. (2017). Asphalt Pavement Industry Survey on Recycled Materials and Warm-Mix Asphalt Usage: 2015. National Asphalt Pavement Association (NAPA). Lanham, MD.

Kim, M.; Mohammad, L.; and Elseifi, M.A. Characterization of Fracture Properties of Aspahlt Mixtures as Measured by Semicircular Bend Test and Indirect Tension Test. Transportation Research Record 2296. Transportation Research Board, National Research Council, Washington, D.C., 2012, pp. 115–124.

Kim, M.; Mohammad, L.; and Elsefi, M. (2012). Characterization of Fracture Properties of Asphalt Mixtures as Measured by Semicircular Bend Test and Indirect Tension Test. Transportation Research Record 2296. Transportation Research Board. Washington, DC

Kodippily, S.; Holleran, G.; and Henning, T. (2016). Improving Recycled Asphalt Mix Performance through Rejuvenation. Transportation Research Board 95th Annual Meeting Compendium of Papers. Washington, DC.

Ma, W. (2014). Proposed Improvements to Overlay Test for Determining Cracking Resistance of Asphalt Mixtures. Auburn University, Department of Civil Engineering. Auburn, AL: Auburn University.

Mohammad, L.; Kim, M.; and Calla, H. (2016). Development of Performance-based Specifications for Louisiana Asphalt Mixtures. Louisiana Transportation Research Center, Final Report 558. Baton Rouge, LA.

Moore, N. (2016). Evaluation of Laboratory Cracking Tests Related to Top-Down Cracking in Asphalt Pavements. Auburn University, Department of Civil Engineering. Auburn, AL: Auburn University.

Nair, R. and Priyadharshini, Y. (2013) Evaluation of Dynamic Modulus of Modified and Unmodified Asphalt Mixtures. Proceedings of International Conference on Energy and Environment-2013 (ICEE 2013). International Journal of Innovative Research in Science, Engineering and Technology Volume 2, Special Issue 1.

Nsengiyumva, G.; Kim, Y.; and You, T. (2015). Development of a Semicircular Bend (SCB) Test Method for Performance Testing of Nebraska Asphalt Mixtures. Nebraska Department of Roads, Report SPR-P1(15) MPMC07. Lincoln, NE.

Oliver, J. W. H. (2001). The Influence of the Binder in RAP on Recycled Asphalt Properties. International Journal of Road Materials and Pavement Design, Vol. 2, No. 3, pp. 311- 325.

Ozer, H.; Al-Qadi, I.; Lambros, J.; El-Khatib, A.; Singhvi, P.; and Doll, B. (2016). Development of Fracture-Based Flexibility Index for Asphalt Concrete Cracking Potential Using Modified Semi-Circle Bending Test Parameters. Construction and Building Materials 115, pp. 390- 401.

Rais, N.; Wahab, Y.; Endut, I.; Latif, A. (2013). Dynamic Modulus Master Curve Construction Using the Modified MEPDG Model. First International Conference on Artificial Intelligence, Modeling, and Simulation.

Roque, R.; Birgisson, B.; Drakos, C.; and Dietrich, B. (2004). Development and Field Evaluation of Energy-Based Criteria for Top-down Cracking Performance of Hot Mix Asphalt. Journal of the Association of Asphalt Paving Technologists, Vol. 73, 229-260.

Tasdemir, Y.; Das, P.; and Birgisson, B. (2010). Determination of Mixture Fracture Performance with the Help of Fracture Mechanics. 9th International Congress on Advances in Civil Engineering. Turkey.

Terrel, R. and Epps, J. (1989). Using Additives and Modifiers in Hot-Mix Asphalt. Quality Improvement Series 114 A. NAPA. Lanham, MD.

Tran, N.; Taylor, A.; and Turner, P. (2012). Effect of Rejuvenator on Performance Properties of HMA Mixtures with High RAP and RAS Contents. NCAT Report 12-05, National Center for Asphalt Technology at Auburn University. Auburn, AL.

Tran, N.; Xie, Z.; Julian, G.; Taylor, A.; Willis, R.; Robbins, M.; Buchanan, S. (2016) Effect of a Recycling Agent on the Performance of High-RAP and High-RAS Mixtures: Field and Lab Experiments. ASCE Journal of Materials in Civil Engineering Vol. 29, Issue 1.

Veeraragavan, R.; Mallick, R.; Tao, M.; Zaumanis, M.; Frank, R.; and Bradbury, R. (2017). Laboratory Comparison of Rejuvenated 50 Percent Reclaimed Asphalt Pavement and Hot-Mix Asphalt with Conventional 20 Percent RAP Mix. Transportation Research Board 96th Annual Meeting Compendium of Papers. Washington, DC.

Walubita, L.; Faruk, A.; Das, G.; Tanvir, H.; Zhang, J.; and Scullion, T. (2012). The Overlay Tester: A Sensitivity Study to Improve Repeatability and Minimize Variability in the Test Results. College Station, TX: Texas Transportation Institute.

West, R. (2015). Best Practices for RAP and RAS Management. National Pavement Association Quality Improvement Series 129. Lanham, MD.

Willis, R. and Turner, P. (2016). Characterization of Extracted Binders Extracted from Reclaimed Asphalt Shingles. NCAT Report 16-01, National Center for Asphalt Technology at Auburn University. Auburn, AL.

Xie, Z.; Tran, N.; Julian, G.; Taylor, A.; and Davis, L. (2017) Performance of Asphalt Mixtures with High Recycled Contents Using Rejuvenators and Warm-Mix Additive: Field and Lab Experiments. ASCE Journal of Materials in Civil Engineering Vol. 29, Issue 10.

Zhou, F.; Hu, S.; Chen, D.; and Scullion, T. (2007). Overlay Tester: Simple Performance Test for Fatigue Cracking. Transportation Research Record: Journal of the Transportation Research Board, No. 2001, 1-8. Washington, DC.

Zhou, F.; Hu, S.; and Scullion, T. (2007). Development and Verification of the Overlay Tester Based Fatigue Cracking Prediction Approach. Texas Transportation Institute, Texas A&M University. College Station, TX.

Zaumanis, M.; Mallick, R.; and Frank, R. (2015). Evaluation of Different Recycling Agents for Restoring Aged Asphalt Binder and Performance of 100% Recycled Asphalt. Rilem Materials and Structures, pp. 2475-2488.

APPENDICES

Extracted Binder PG

Mixture	***T_{cont} Low S***	***T_{cont} Low m***	***Delta Tc***
N1	-26.0	-16.6	-9.4
N7	-26.5	-16.4	-10.1
N8	-25.4	-5.4	-20.0
S5	-32.3	-23.0	-9.3

Energy Ratio Test Results

Mixture ID	*Creep Compliance*			*Resilient Modulus*	*Indirect Tension*		
	m-value	*D_1*	*Compliance Rate*	*M_R (GPa)*	*S_T (MPa)*	*FE (kJ/m^3)*	*Failure Strain*
N1 (20% RAP)	0.407	5.582E-07	3.79E-09	9.9	2.37	4.8	2,584
N7 (35% RAP + Delta S)	0.335	5.122E-07	1.74E-09	9.3	2.28	3.3	1,906
N8 (20% RAP + 5% RAS)	0.252	3.475E-07	4.98E-10	12.8	2.42	1.8	1,047
S5 (35% RAP)	0.347	9.102E-07	3.46E-09	7.4	1.87	6.0	3,840

Mixture ID	*Target V_a (%)*	*Stress (psi)*	*α*	*$DCSE_{HMA}$ (kJ/m^3)*	*$DCSE_{Min}$ (kJ/m^3)*	*ER*
N1 (20% RAP)	7	150	4.68E-08	4.5	0.8	5.5
N7 (35% RAP + Delta S)	7	150	4.73E-08	3.0	0.4	7.3
N8 (20% RAP + 5% RAS)	7	150	4.66E-08	1.6	0.1	12.8
S5 (35% RAP)	7	150	4.96E-08	5.8	0.8	7.4

Texas Overlay Test Results

Mixture ID	*Sample ID*	*V_a (%)*	*Temp (°C)*	*MOD (in)*	*Frequency*	*N_f*
N1 20% RAP (Control)	#1B	6.8	25	0.635	0.1	32
N1 20% RAP (Control)	#2A	7.0	25	0.635	0.1	10
N1 20% RAP (Control)	#3B	7.0	25	0.635	0.1	49
N1 20% RAP (Control)	#4A	7.2	25	0.635	0.1	8
N7 35% RAP (Repave)	#103A	6.7	25	0.635	0.1	8
N7 35% RAP (Repave)	#105A	7.0	25	0.635	0.1	17
N7 35% RAP (Repave)	#107A	7.0	25	0.635	0.1	7
N7 35% RAP (Repave)	#109A	6.6	25	0.635	0.1	6
N8 20% RAP + 5% RAS	#1B	6.7	25	0.635	0.1	2
N8 20% RAP + 5% RAS	#2A	6.3	25	0.635	0.1	2
N8 20% RAP + 5% RAS	#3A	6.5	25	0.635	0.1	2
N8 20% RAP + 5% RAS	#4B	6.6	25	0.635	0.1	2
S5 35% RAP (Control)	#1A	7.3	25	0.635	0.1	37
S5 35% RAP (Control)	#2A	7.4	25	0.635	0.1	73
S5 35% RAP (Control)	#3B	7.0	25	0.635	0.1	110
S5 35% RAP (Control)	#4B	7.3	25	0.635	0.1	23

NCAT-Modified Overlay Test Results

Mixture ID	*Sample ID*	*V_a (%)*	*Temp (°C)*	*MOD (in)*	*Frequency*	*N_f*
N1 20% RAP (Control)	#1A	7.2	25	0.381	1	342
N1 20% RAP (Control)	#2B	6.7	25	0.381	1	452
N1 20% RAP (Control)	#3A	7.0	25	0.381	1	607
N1 20% RAP (Control)	#5A	6.8	25	0.381	1	662
N7 35% RAP (Repave)	#104A	6.6	25	0.381	1	40
N7 35% RAP (Repave)	#105B	6.7	25	0.381	1	110
N7 35% RAP (Repave)	#108B	7.0	25	0.381	1	48
N7 35% RAP (Repave)	#109B	7.0	25	0.381	1	93
N8 20% RAP + 5% RAS	#1A	6.8	25	0.381	1	14
N8 20% RAP + 5% RAS	#2B	6.8	25	0.381	1	10
N8 20% RAP + 5% RAS	#3B	6.5	25	0.381	1	13
S5 35% RAP (Control)	#2B	7.2	25	0.381	1	586
S5 35% RAP (Control)	#4A	7.5	25	0.381	1	1037
S5 35% RAP (Control)	#6A	6.9	25	0.381	1	697

LTRC Semi Circular Bend Test Results

Mixture ID	*Sample ID*	*V_a (%)*	*Notch Length (mm)*	*Ligament Length (mm)*	*Peak Load (kN)*	*Disp. @ Peak Load (mm)*	*Strain Energy @ Peak Load (KN-mm)*
N1 20% RAP	8A-1	7.1	24.75	48.75	1.017	0.832	0.500
N1 20% RAP	9A-1	7.0	24.90	48.40	0.971	0.937	0.586
N1 20% RAP	9B-1	6.9	25.85	48.20	1.050	0.861	0.573
N1 20% RAP	10A-1	6.9	24.95	48.65	0.930	0.795	0.477
N1 20% RAP	2A-2	7.2	31.65	41.65	0.783	0.879	0.465
N1 20% RAP	2B-2	7.2	32.80	41.50	0.787	0.773	0.385
N1 20% RAP	3A-2	6.9	31.40	42.40	0.807	0.685	0.341
N1 20% RAP	3B-2	7.2	32.30	41.75	0.882	0.707	0.390
N1 20% RAP	4B-3	6.8	38.20	35.60	0.536	0.756	0.273
N1 20% RAP	5A-3	7.3	37.30	36.10	0.537	0.710	0.239
N1 20% RAP	5B-3	7.1	38.05	35.85	0.565	0.776	0.280
N7 35% RAP	20A	6.9	25.73	48.05	0.913	1.070	0.621
N7 35% RAP	20B	7.5	25.42	48.08	0.849	0.964	0.530
N7 35% RAP	21A	6.7	25.81	48.16	0.994	0.857	0.519
N7 35% RAP	21B	7.5	25.20	47.93	0.757	0.842	0.415
N7 35% RAP	24A	6.7	31.86	42.11	0.670	0.876	0.373
N7 35% RAP	27A	6.9	31.25	41.89	0.697	0.883	0.392
N7 35% RAP	27B	7.4	31.98	42.10	0.619	0.916	0.386
N7 35% RAP	28B	7.3	31.85	41.94	0.639	0.879	0.343
N7 35% RAP	22A	6.6	37.53	36.41	0.583	0.718	0.255
N7 35% RAP	22B	7.4	37.50	35.87	0.541	0.757	0.261
N7 35% RAP	25A	6.9	37.96	36.06	0.535	0.681	0.213
N7 35% RAP	26A	7.4	38.25	35.89	0.491	0.856	0.280
N8 20% RAP 5% RAS	3B-1	6.5	25.45	48.75	1.700	0.466	0.409
N8 20% RAP 5% RAS	8A-1	6.9	24.75	48.80	1.658	0.512	0.457
N8 20% RAP 5% RAS	8B-1	6.7	25.05	49.05	1.350	0.628	0.551

N8 20% RAP 5% RAS	10A-1	7.1	24.50	48.75	1.750	0.516	0.469
N8 20% RAP 5% RAS	5B-2	7.0	31.70	41.80	1.175	0.498	0.279
N8 20% RAP 5% RAS	9A-2	7.3	32.60	41.80	1.078	0.472	0.285
N8 20% RAP 5% RAS	9B-2	6.9	31.50	41.95	1.221	0.477	0.282
N8 20% RAP 5% RAS	2A-3	7.1	38.05	36.00	1.046	0.411	0.199
N8 20% RAP 5% RAS	4A-3	6.9	37.65	36.15	1.031	0.399	0.208
N8 20% RAP 5% RAS	7A-3	7.1	37.60	36.55	0.980	0.354	0.176
N8 20% RAP 5% RAS	7B-3	7.0	37.10	36.25	1.143	0.394	0.196
S5 35% RAP	3A-1	7.0	25.60	48.08	0.731	0.966	0.413
S5 35% RAP	3B-1	6.8	26.08	48.03	0.735	1.125	0.547
S5 35% RAP	4A-1	6.9	25.34	48.16	0.673	1.041	0.446
S5 35% RAP	4B-1	6.8	25.97	48.14	0.654	1.139	0.481
S5 35% RAP	6A-2	6.9	31.78	42.16	0.519	0.973	0.332
S5 35% RAP	6B-2	7.1	31.10	42.55	0.481	0.940	0.291
S5 35% RAP	9A-2	7.1	31.44	42.29	0.491	1.054	0.353
S5 35% RAP	9B-2	7.0	31.65	42.30	0.585	0.928	0.329
S5 35% RAP	2B-3	7.1	38.24	35.75	0.416	0.857	0.235
S5 35% RAP	7A-3	6.9	37.52	36.15	0.397	0.812	0.189
S5 35% RAP	7B-3	7.0	37.93	36.05	0.372	1.094	0.283
S5 35% RAP	10A-3	7.1	37.77	35.91	0.428	0.835	0.229

Illinois Flexibility Index Test Results

Mixture ID	*Sample ID*	*V_a (%)*	*Peak Load (kN)*	*Slope (kN/mm)*	*Strength (psi)*	*Fracture Energy (J/m^2)*	*Flexibility Index (FI)*
N1 20% RAP	1A	7.1	3.46	-4.56	65.8	1509	3.31
N1 20% RAP	2A	7.3	3.36	-4.18	65.0	1481	3.54
N1 20% RAP	2B	7.4	3.55	-4.13	69.0	1597	3.87
N1 20% RAP	2C	7.4	3.43	-3.93	66.4	1553	3.95
N1 20% RAP	3A	7.0	3.60	-4.21	69.4	1607	3.82
N1 20% RAP	3B	7.5	3.73	-4.66	72.5	1582	3.39
N1 20% RAP	4D	7.4	3.36	-4.45	64.3	1422	3.20
N7 35% RAP Delta-S	12A	6.7	3.34	-4.20	64.1	1426	3.40
N7 35% RAP Delta-S	12B	6.9	3.26	-3.95	62.2	1426	3.61
N7 35% RAP Delta-S	13A	6.9	3.28	-3.84	63.1	1311	3.41
N7 35% RAP Delta-S	13B	6.7	3.20	-3.95	61.9	1298	3.29
N7 35% RAP Delta-S	13D	6.8	3.22	-4.43	61.5	1304	2.94
N7 35% RAP Delta-S	14A	7.0	3.23	-3.53	61.6	1379	3.91
N8 20% RAP 5% RAS	1B	7.0	5.21	-30.39	99.4	1025	0.34
N8 20% RAP 5% RAS	1D	7.1	4.93	-18.77	94.2	947	0.50
N8 20% RAP 5% RAS	2B	6.7	5.66	-29.51	107.5	999	0.34
N8 20% RAP 5% RAS	2C	7.0	5.11	-25.68	98.1	980	0.38
N8 20% RAP 5% RAS	2D	6.7	5.66	-27.34	108.3	964	0.35
N8 20% RAP 5% RAS	4A	6.8	4.80	-18.99	91.4	851	0.45
N8 20% RAP 5% RAS	4B	6.8	5.17	-21.72	98.5	1062	0.49
N8 20% RAP 5% RAS	4C	7.0	5.09	-27.44	97.9	931	0.34
N8 20% RAP 5% RAS	4D	6.9	5.35	-29.27	102.8	906	0.31
S5 35% RAP	1A	7.6	3.01	-2.65	57.3	1549	5.85
S5 35% RAP	2C	7.3	2.65	-2.31	52.0	1473	6.38
S5 35% RAP	2D	6.8	2.41	-2.04	48.0	1389	6.81
S5 35% RAP	4A	7.2	2.60	-2.27	49.6	1463	6.45
S5 35% RAP	4B	7.0	2.77	-2.72	53.3	1412	5.19
S5 35% RAP	4D	7.6	2.63	-2.21	50.8	1527	6.91