Use of Anova Asphalt Rejuvenator and Balanced Mix Design Principle to Improve the Performance of High-RAP Asphalt Mixtures

by

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ABSTRACT

The use of recycled materials like Reclaimed Asphalt Pavement (RAP) in asphalt mixtures is now a common practice with associated economic, social, and environmental benefits. However, asphalt mixture performance issues, especially cracking resistance, is a source of concern when the RAP content in the asphalt mixture is high. To address this challenge, utilization of recycling agents, also known as rejuvenators, can potentially restore the properties of the aged asphalt binders and improve the overall cracking resistance of the asphalt mixture. To that effect, this project was conducted to evaluate the laboratory and field performance of a high-RAP asphalt mixture, which contains Anova asphalt rejuvenator and was designed by following the Virginia Departments of Transportation (VDOT) provisional Balanced Mix Design (BMD) specifications.

To evaluate and compare the laboratory performance of rejuvenated and non-rejuvenated RAP mixtures, three plant-produced asphalt mixtures, including the control and two experimental asphalt mixtures, were considered. The control asphalt mixture has 30% RAP with no rejuvenator, the first experimental asphalt mixture has 45% RAP with Anova asphalt rejuvenator, and the second experimental asphalt mixture has 45% RAP but no rejuvenator. As a comparison with the laboratory performance of the plant-produced mixes, the control asphalt mixture and the first experimental asphalt mixtures were produced and placed in the surface layer of two 100-foot Sections N3A and N3B, which were divided from a 200-foot Section N3 at the National Center for Asphalt Technology (NCAT) Pavement Test Track.

The results of the laboratory rutting tests of the control asphalt mixture and the first experimental asphalt mixture showed that the rutting resistance of both asphalt mixtures is not statistically different. Concerning cracking resistance, cracking test results of the Illinois Flexibility Index Test (I-FIT), Overlay Test (OT), and Disc-Shaped Compact Tension (DCT) test suggested that the first experimental asphalt mixture outperformed both the control asphalt mixture and the second experimental asphalt mixture. However, the indirect tensile asphalt cracking test (IDEAL-CT) results suggested that the control asphalt mixture had the best cracking resistance.

At the NCAT Pavement Test Track, the early field performance of Sections N3A and N3B, with approximately 2 million Equivalent Single Axle Loads (ESALs) of traffic applied, indicates no visible cracking and low rutting of the surface asphalt mixtures. The current average International Roughness Index (IRI) values of N3A and N3B, by VDOT standard, classify the ride quality of both Sections as “good.”

In summary, the laboratory test results and the early field performance of Sections N3A and N3B have shown that the use of Anova asphalt rejuvenator in the 45% RAP asphalt mixtures performs the same or in some instances, better than the control 30% RAP asphalt mixture without rejuvenator, with the exception of the IDEAL – CT result. Both asphalt mixtures were designed by following the VDOT’s BMD provisional specifications.

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CHAPTER ONE – INTRODUCTION

1.1 Problem Statement

The past research efforts have made the inclusion of recycled materials, such as RAP in the design and construction of a new asphalt mixture a common practice. In road construction, asphalt pavement is one of the most recycled products. Going by the statistics supplied by the Federal Highway Administration (FHWA) and the U.S. Environmental Protection Agency (EPA), it was estimated that in the early 1990s, over 90 million tons of asphalt pavement was milled off every year during resurfacing or rehabilitation of roads built with asphalt material, and more than 80 percent of the resulting RAP is reused in new asphalt mixtures (FHWA, 1993). The National Asphalt Pavement Association's (NAPA) yearly RAP and WMA utilization survey indicates that on the average, RAP usage rose from approximately 16% in 2009 to 20% in 2017 (NAPA, 2018).

The consistent increase in the cost and demand for the limited aggregate and asphalt binder supply has made the use of RAP in asphalt mixture production a valuable alternative. To this effect, ways of increasing the RAP content in asphalt mixture without compromising the performance has been of economic and environmental sustainability interest to the asphalt paving industry. According to Robinett and Epps (2010), a hot-mix asphalt (HMA) mixture containing an intermediate RAP content (25% RAP) resulted in 10% reduction in emissions, 10% energy savings, and 20 – 25% preservation of natural resources, which translates to reduced production and construction costs. Furthermore, NAPA (2018) reported that approximately $2.2 billion cost savings were recorded with the replacement of virgin materials with recycled materials in 2017.

Despite the economic and environmental benefits associated with the use of a high percentage of RAP in asphalt mixtures, the observed high stiffness in such asphalt mixtures is a source of major concern (Haghshenas et al., 2016). The high asphalt mixture stiffness is due to the blending of the oxidized asphalt binder with the unaged virgin asphalt binder in RAP mixes. Stiff mixes are difficult to compact to the design air voids and may ultimately result in premature field failure (Mogawer et al., 2012).

According to Haghshenas et al. (2016), extensive research has been conducted on the effect of rejuvenators on the mechanical and performance properties of asphalt binders and mixtures by Hajj et al. (2013); Hill et al. (2013); Im and Zhou (2014); Zaumanis et al. (2013); and Pan et al. (2018). Rejuvenators have been used to restore some of the performance properties of aged asphalt for both cold in-place recycling application and as a pavement surface treatment emulsion to preserve weathered asphalt pavements (Tran et al., 2012). Also, research by Mogawer et al. (2013) showed that rejuvenators play a major role in softening aged asphalt binders and improving the overall cracking resistance of asphalt mixtures.

Despite the positive effects of rejuvenators on aged asphalt mixtures, there is a knowledge gap on the potentials of designing high-RAP asphalt mixtures with the relatively new asphalt mix design principle – BMD method. Therefore, this research is focusing on the use of the BMD method to design a high-RAP asphalt mixture with rejuvenator, to provide long-lasting, quality pavement performance.

1.2 Research Objective and Scope

The purpose of this research study was to evaluate and compare the laboratory and field performance of a high-RAP asphalt mixture containing Anova asphalt rejuvenator with a lower RAP asphalt mixture without rejuvenator. Both asphalt mixtures were designed by following VDOT's BMD provisional specifications.

Three asphalt mixtures, including the control and two experimental asphalt mixtures, were designed for this research study based on VDOT's proposed BMD specification. The proposed BMD specification comprises of asphalt performance tests such as Asphalt Pavement Analyzer (APA) test, IDEAL-CT, and Cantabro abrasion test, to evaluate rutting, cracking, and mass loss, respectfully. The control asphalt mixture had 30% RAP with no rejuvenator, the first experimental asphalt mixture had 45% RAP with Anova asphalt rejuvenator, and the second experimental asphalt mixture had 45% RAP without rejuvenator. The second experimental asphalt mixture was sampled to determine the benefit of the Anova asphalt rejuvenator in the first experimental mix.

As a part of the field evaluation, the control asphalt mixture and the first experimental asphalt mixture were placed in the surface layers of two 100-foot Test Sections N3A and N3B of the NCAT Test Track, respectfully. Further, the three plant-produced asphalt mixtures were reheated and evaluated in the laboratory using several other performance tests to support future implementation.

1.3 Organization of Thesis

This report comprises of five chapters. The first chapter covers the problem statement of the research, the research scope and objectives, and the organization of the overall report. The second chapter comprises of the literature review on RAP, asphalt binder composition and aging process, rejuvenation of asphalt binders, Anova asphalt rejuvenator, and a review of previous studies on the effects of rejuvenators on asphalt binders and asphalt mixtures. Chapter three contains the research methodology, which includes the mix design approach implemented, the laboratory testing plan, and the field data collection. Chapter four presents the result of the experiments described in the previous chapter. Finally, Chapter 5 summarizes the main findings of this study and provided some recommendations for a future research study.

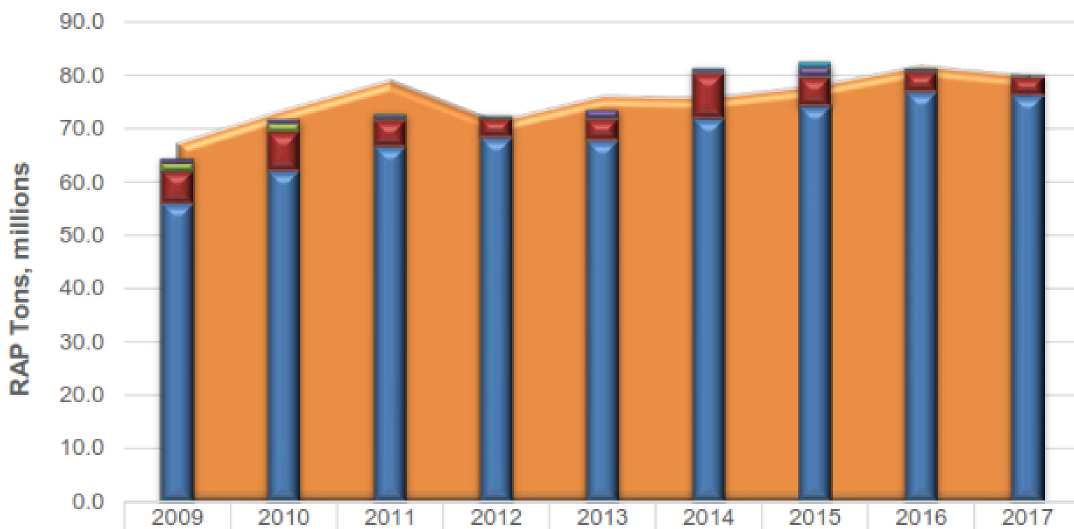
2 CHAPTER TWO – LITERATURE REVIEW

2.1 RAP Usage in Asphalt Mixtures

The use of RAP in asphalt mix has grown popular over the years due to its potential to reduce the carbon footprint and maintain the drive for sustainable practices in asphalt production. Generally, RAP is the material generated from the milling of old asphalt pavement during rehabilitation or reconstruction operation, rejected paving mixtures, asphalt plant waste, etc. The quality and properties of a RAP depend on the source it was milled or extracted from. Asphalt mixtures containing high RAP content, typically above 25% RAP, is referred to as "high-RAP asphalt mixtures" (Haghshenas et al., 2016).

The construction season survey, from 2009 – 2017, of the estimated tons of RAP used in asphalt mixtures, aggregates, cold-mix asphalt, other uses, and the amount landfilled is presented in Figure 1. As shown, the largest portion of RAP is used in hot-mix asphalt (HMA) or warm-mix asphalt (WMA) mixtures, which is the most productive use of RAP. Further, the use of RAP in HMA/WMA rose from an estimated 56 million tons in 2009 to an estimated 76.2 million tons in 2017 (NAPA, 2018).

The consistent increase in the use of RAP in pavement construction may push for a need to adopt a higher percentage of RAP in asphalt mixtures. However, to make the resulting high-RAP mixtures perform similar or superior to a virgin mix, it is necessary to engineer the high-RAP mixtures.



	2009	2010	2011	2012	2013	2014	2015	2016	2017
Accepted	67.2	73.5	79.1	71.3	76.1	75.8	78.0	81.8	79.9
Landfilled	0.1	0.0	0.3	0.2	0.1	0.2	1.0	0.1	0.0
Used in Other	0.7	0.8	0.7	0.2	1.5	0.6	1.6	0.4	0.2
Used in Cold Mix	1.5	1.6	0.2	0.2	0.2	0.2	0.2	0.2	0.3
Used in Aggregate	6.2	7.3	4.9	3.6	4.0	8.5	5.5	3.7	3.4
Used in HMA/WMA	56.0	62.1	66.7	68.3	67.8	71.9	74.2	76.9	76.2

Figure 1. Comparison of Estimated RAP Tons Usage from 2009 – 2017 (Williams et al., 2018)

Based on the outcome of previous research studies, the use of RAP in an asphalt mixture changes the original mechanical properties of asphalt mixtures. The mechanical changes in an asphalt mixture containing RAP can either improve or reduce the performance of the asphalt mixture, depending on the percentage and source of RAP in the asphalt mixture. Haghshenas et al. (2016) reviewed the four mechanical properties of asphalt mixtures containing RAP, including; stiffness, moisture resistance, permanent deformation or rutting resistance, and cracking resistance.

The stiffness in asphalt mixtures containing RAP can be attributed to the aged asphalt binder present in RAP. Such an increase in stiffness makes the RAP mixture less resistance to fatigue and low-temperature brittleness. Among the existing asphalt mixture performance tests, the dynamic modulus test is the most convenient test for estimating the stiffness in asphalt mixtures

(Haghshenas et al., 2016). According to Al-Qadi et al. (2009), the result of dynamic modulus test on 0%, 20%, and 40% RAP mixtures revealed that there wasn't a significant difference between the stiffness of 0% and 20% RAP mixtures. However, the asphalt mixture containing 40% RAP had a significantly higher dynamic modulus as compared to the 0% and 20% RAP mixture.

On moisture resistance of asphalt mixtures, no consensus is yet to be reached among researchers on the moisture susceptibility of asphalt mixtures containing RAP. However, according to Tran et al. (2012), some researchers claimed that asphalt mixtures with high RAP content are more resistant to moisture damage as compared to asphalt mixtures without RAP.

In terms of rutting resistance, an evaluation of the rutting resistance of asphalt mixtures containing different percentage of RAP (i.e., 0%, 10%, 20%, and 40%) was carried out by McDaniel et al. (2011), and the result of the experiment revealed that the higher the RAP content of the asphalt mixture, the stiffer and more resistant the asphalt mixture is to permanent deformation. An evaluation of the field performance of asphalt mixtures containing 50% RAP was conducted by West et al. (2010) at the NCAT accelerated loading test track facility. The result of the three years field investigation showed that asphalt mixtures containing RAP perform better in rutting as compared to asphalt mixtures without RAP. A summary of the past investigations by different researchers on the rutting resistance of virgin and RAP asphalt mixtures is as shown in Table 1.

Table 1. Rutting Resistance of RAP Mixtures (Bonaquist, 2013)

Study	Mixture Type	RAP Content	Test: Hamburg Wheel Track	Flow Number	Pavement Test Track	Superpave Shear Tester	Asphalt Pavement Analyzer
McDaniel et al. (2000)	Lab HMA	0, 10, 20, 40				Increases	
Al-Qadi et al. (2012)	Lab HMA	0-50	Increases	Increases			
Xiao et al. (2007)	Lab HMA	0, 15, 25, 30, 38					Increases
Mogawer et al. (2012)	Plant HMA	0-40	Increases				
West et al. (2012)	Lab WMA	0, 50			Slightly Decrease		
	Lab HMA	0, 50			Increases		

The cracking resistance of asphalt mixtures is generally improved when a soft asphalt binder is used. However, the presence of RAP makes the asphalt mixture becomes stiff due to the long-term aged asphalt binder of the RAP. Stiffness of the mix makes it susceptible to cracking. Over the years, various researchers have worked on the cracking behavior of RAP mixtures using different cracking performance tests. The summary of the past research on the resistance of asphalt mixtures to load and non-loaded related cracking is as presented in Table 2 and 3, respectively (Haghshenas et al., 2016).

Table 2. Load-Related Cracking Resistance of RAP Mixtures (Bonaquist, 2013)

Study	Mixture Type	RAP Content	Test				
			Flexural Fatigue	Energy Ratio	Overlay Tester	Cyclic Direct Tension	Indirect Tension Fracture Energy
McDaniel et al. (2000)	Lab HMA	0, 10, 20, 40	Decreases				
Shu et al. (2008)	Lab HMA	0, 10, 20, 30	Decreases	Decreases			
Hajj et al. (2009)	Lab HMA	0, 15, 30	Decreases				
Mogawer et al. (2012)	Plant HMA	0 - 40			Decreases		
Zhao et al. (2012)	Lab WMA	0, 30, 40, 50	Increases	Increases			
	Lab HMA	0, 30	Decreases	Increases			
West et al. (2013)	Lab HMA	0, 25, 40, 55					Decreases
Li and Gibson (2013)	Lab HMA	0, 20, 40				Decreases	

Table 3. Resistance of RAP Mixtures to Low-Temperature Cracking (Bonaquist, 2013)

Study	Mixture Type	RAP Content	Low-Temperature Cracking			
			Disc- Shaped Compact Tension	Semicircular Bend	Indirect Tension	Thermal Stress Restrained Specimen Test
Li et al. (2008)	Lab HMA	0, 20,40		Lower fracture energy for 40%		
McDaniel et al. (2012)	Plant HMA	0, 15, 25, and 40			Lower cracking temperature for 40%	
Hajj et al. (2011)	Plant and Lab HMA	0, 15, 50				Higher fracture temperature for 50%
Behnia et al. (2010)	Lab HMA	0, 30	Lower fracture energy for 30%			
West et al. (2013)	Lab HMA	0, 25, 40, 55		Mixture and temperature dependent		

2.2 Asphalt Binder Composition and Aging Mechanism

While there are natural asphalt binder deposits in some places around the world, it is not as popular as asphalt binder extracted from crude oil. The asphalt obtained from crude oil is composed of asphaltenes and maltenes. Asphaltenes consist of saturates, naphthene aromatics, and polar aromatics, while maltenes consist of resins and oils. Asphaltene is the dispersed phase while maltene is the dispersion medium of the colloidal solution (Thyrion, 2000; Tran et al., 2012). Over time, asphalt binder ages, which leads to the transformation of some of the maltenes to asphaltenes. The shortage of maltenes results in the flocculation of the asphaltenes, which leads to higher viscosity, lower ductility, and consequently, decrease in the fatigue resistance of the asphalt binder (Corbett, 1975; Tran et al., 2012).

The physical and chemical properties of asphalt binder change due to aging. The aging process causes a decrease in the fatigue resistance of the asphalt binder, and consequently, the loss of durability of the constructed asphalt pavement. During asphalt mixing and compaction, oxidation of the asphalt binder occurs due to the reaction with atmospheric oxygen and high mixing temperatures (evaporation of oils and resins in asphalt binder). Subsequently, the prevailing climatic condition influences the aging of the compacted asphalt mixtures (Vargas and Reyes, 2010).

In the Superpave asphalt mix design developed during the strategic highway research program (SHRP) in 1993, asphalt aging is simulated for both the short-term and long-term using the rolling-thin film oven (RTFO) test and the pressure aging vessel (PAV) test, respectively. The short-term aging is mainly attributed to the absorption and/or volatilization of the oily components of maltenes during mixing at the plant and construction at the project location. On the other hand, long-term aging over the service life of the pavement can be attributed to oxidation of the asphalt constituents, polymerization, and thixotropy i.e., the formation of a structure within the asphalt binder (Khandal et al., 1996; Tran et al., 2012).

Summarily, asphalt aging evolved from being considered a physical hardening process to getting recognized as a more complex phenomenon. The complexity lies in the rheological study of the viscoelastic properties of asphalt binders. The analysis of the chemical composition of asphalt to estimate the changes before and after aging and material modification is carried out using advanced techniques. Some of the techniques include; Fourier transform infrared spectroscopy (FTIR), ultraviolet and visible diffraction, nuclear magnetic resonance, liquid chromatography, and thermogravimetric analysis, etc. (Vargas and Reyes, 2010).

2.3 Rejuvenation of Asphalt Binders

As reflected in past studies, recycling agents (rejuvenators or softening agents) have proved to be effective in improving the cracking resistance of mixtures with high recycled binder ratios (RBRs). However, surveys suggested that there are still concerns among State DOTs on the use of recycling agents, such as rejuvenators, in asphalt mix due to lack of experience, but more importantly, the absence of standard test procedures to determine the optimum dosage of rejuvenator and/or to evaluate the performance of asphalt mixtures incorporating the recycling agents (Martin et al., 2018).

According to Roberts et al. (1996), the rheological properties of an aged asphalt binder can be restored by adding a recycling agent. The recycling agent may be a softening agent, such as lube stock, flux oil, and slurry oil for lowering the viscosity of the aged asphalt binder, or a rejuvenator which helps in restoring the physical and chemical properties. In most cases, a rejuvenator comprises of a lubricating oil extracts and extender oils which are highly rich in maltene constituents (naphthenic or polar aromatic fractions). The high maltene constituents in rejuvenators helps in restoring some maltenes lost during the construction and service life of the asphalt pavement (Terrel and Epps, 1989). The aromatic fraction of maltenes should be high enough to keep the asphaltenes dispersed, however, the saturates should be low since they are incompatible with asphaltenes (Bullin et al., 1997; Dunning and Mendenhall, 1978). The five major categories of rejuvenators and a summarized description of each rejuvenator category is as presented in Table 4.

Table 4. Categories of Rejuvenators (NCAT, 2014)

Category	Examples	Description
Paraffinic Oils	Waste Engine Oil (WEO)	Refined used lubricating oils
	Waste Engine Oil Bottoms	
	(WEOB)	
	Valero VP 165	
	Storbit	
Aromatic Extracts	Hydrolene	Refined crude oil products with polar aromatic oil components
	Reclamite	
	Cyclogen L	
	ValAro 130A	
Naphthenic Oils	SonneWarmix RJ™	Engineered hydrocarbons for asphalt modification
	Ergon HyPrene	
Triglycerides & Fatty Acids Waste Vegetable Oil	Waste Vegetable Grease	Derived from vegetable oils, and it has other key chemical elements in addition to triglycerides and fatty acids
	Brown Grease	
Tall Oils	Sylvaroad RP1000	Paper industry byproducts, same chemical family as liquid antistrip agents
	Hydrogreen	

The actualization of the potentials of rejuvenators in recycled asphalt mixtures or aged asphalt binders depends on the uniform dispersion of the rejuvenator within the recycled asphalt mixtures, and the diffusion of the rejuvenator into the aged asphalt binder that coats the aggregates in the asphalt mixture (Tran et al., 2012). By mixing dyes with rejuvenators, Lee et al. (1983) conducted a visual investigation of how rejuvenators are dispersed in the recycled asphalt mixtures at different asphalt plants. The main finding of the research is that the uniform distribution of rejuvenators in recycled asphalt mixtures could be achieved through mechanical mixing.

Regarding the diffusion of rejuvenators into an aged binder, Carpenter and Wolosick (1980) summarized the diffusion process into four stages. At first, the rejuvenator forms a very low viscous layer around the asphalt-coated aggregates. Then, the rejuvenator begins to penetrate the thick layer of the aged asphalt binder, which results in the softening of the aged asphalt binder and reduction in the available raw rejuvenator. At the third stage, the raw rejuvenator is completely exhausted due to its continuous penetration into the aged asphalt binder, which results in the decrease in the viscosity of the inner layer, while the viscosity of the outer layer gradually increases. At the fourth stage, the inner and outer layer of the aged binder would have attained a certain degree of equilibrium.

The outcome of the rejuvenation diffusion test conducted by Carpenter and Wolosick (1980) suggested that there are challenges associated with the characterization of recycled asphalt binder that contains a rejuvenator. Factors such as sample handling, the time frame between mixing and testing, and the variation in the diffusion rate of different modifiers at different concentration level influence the accurate characterization of recycled asphalt binders containing rejuvenators. Karlsson and Isacsson (2003) suggested that the diffusion rate of rejuvenators into an aged binder is determined by the maltene phase of the recycled asphalt binder.

2.4 Anova Asphalt Rejuvenator

Anova asphalt is a bio-based rejuvenator that was formulated by Cargill. The product was designed to restore the rheological properties of aged asphalt binder, which facilitates the increase in the RAP and Reclaimed Asphalt Shingles (RAS) content of asphalt mixtures without compromising the short- or long-term performance of the mix. Anova rejuvenator is applicable in high RAP and RAS mixtures while lowering compaction temperature by approximately 20^0C in high RAP pavements. It is also applicable to surface seals, cold-in-place recycling, hot-in-place recycling, and pavement surface rejuvenating applications (Cargill, 2019).

2.5 Effects of Bio-based Rejuvenators on the Properties of Aged Asphalt Binders

Porot and Grady (2016) investigated the effectiveness of bio-based rejuvenators in restoring the mechanistic properties of aged asphalt binder. In the first part of the study, the researchers evaluated the effect of the rejuvenating agent at various dosage levels using basic asphalt binder properties, including; penetration value at 25 ^{0}C, temperature at softening point, and viscosity at different temperatures. The dosage of the rejuvenator ranges from 0 % to 15 % by weight of the aged asphalt binder. The outcome of the study is as delineated in Figure 2 and 3.

Figure 2 depicts the impact of different dosage level of rejuvenator on the penetration value of the blend and the softening point temperature. The boxes represent the penetration grade specification by following EN 12591. From Figure 2, at 0 % dosage level, the penetration value is approximately 0.5 inch (12.5mm), which suggests a stiff asphalt binder. As the dosage level increases, a corresponding increase in penetration value was recorded, which suggests that bio-based rejuvenators soften aged asphalt binders. Similar trend can be observed with softening point temperature data. Holistically, over a tight range of 3 % to 10 % dosage of rejuvenator in a blend, penetration grade between 20/30 and 70/100 can be achieved. On average, a 7 % dosage level can restore the properties of the aged asphalt by 3 grades (Porot and Grady, 2016).

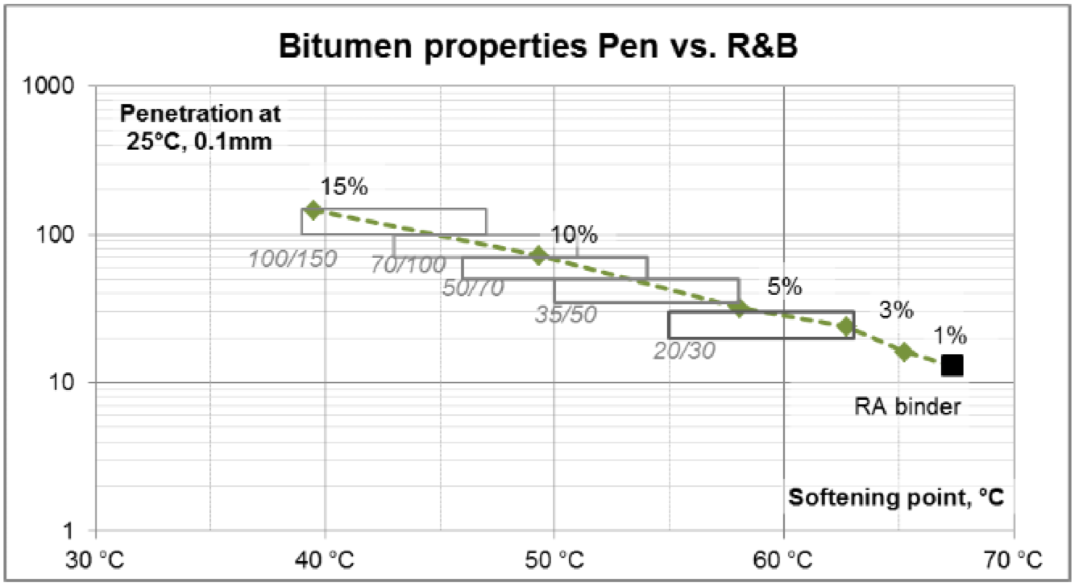


Figure 2. Effect of Rejuvenator on Pen. and Softening Point (Porot and Grady, 2016)

As shown in Figure 3, the dashed line indicates the maximum allowable viscosity for effective mixing of the recycled asphalt binder at the asphalt plant. As the rejuvenator dosage level increases, the required mixing temperature decreases. From Figure 3, the recycled asphalt binder containing 0% dosage level can be mixed at a minimum temperature of 180 ^{0}C, however, if the dosage level is increased to 10%, the mixing can be conducted at a lower temperature of 160 ^{0}C with effective mixing of the recycled asphalt binder at the plant.

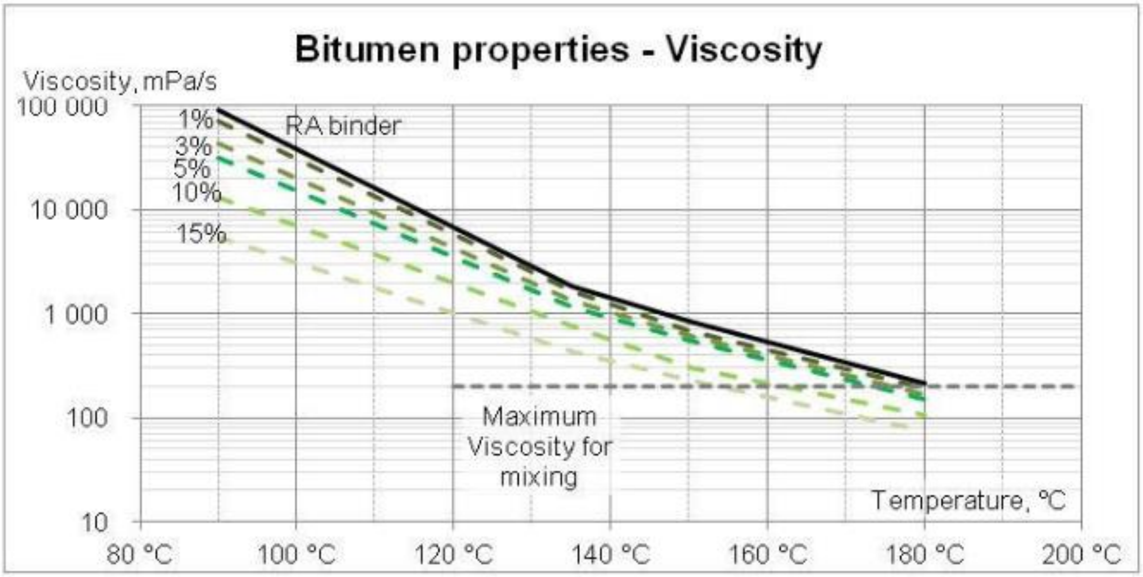


Figure 3. Effect of Rejuvenator on Viscosity (Porot and Grady, 2016)

Previous analysis of the effects of bio-based rejuvenators on asphalt binder by Porot and Grady (2016) considered a single recycled asphalt binder. However, in the second part of the study, the researchers went further by evaluating the effect of a constant 5% rejuvenator dosage on varying recycled asphalt binder of different quality and source. Four different recycled asphalt binders, one pressure aging vessel (PAV) aged asphalt binder, and one unaged asphalt binder were analyzed. The outcome of the analysis is as shown in Figure 4. In summary, the rejuvenator consistently improved the penetration value and softening point of the different recycled and virgin asphalt binders considered in the experiment.

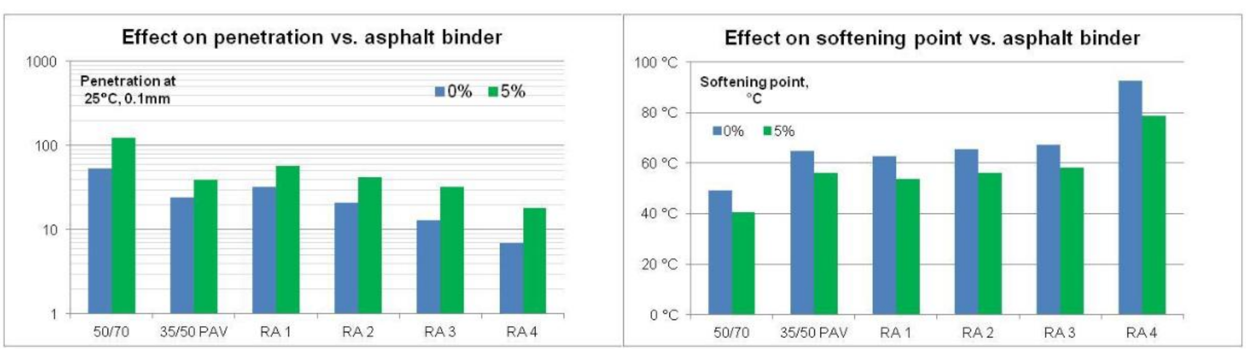


Figure 4. Effect of 5% Rejuvenator on Various Binders (Porot and Grady, 2016)

Cavalli et al. (2018) also evaluated the effects of bio-based rejuvenators on RAP asphalt binders. From the master curves shown in Figure 5, the results of the research showed that the addition of 5% bio-based rejuvenators soften the RAP asphalt binder. At the high-frequency range, the rejuvenated asphalt binder is softer than the virgin asphalt binder, but at low-frequency range, the virgin asphalt binder is slightly softer than the rejuvenated RAP asphalt binder.

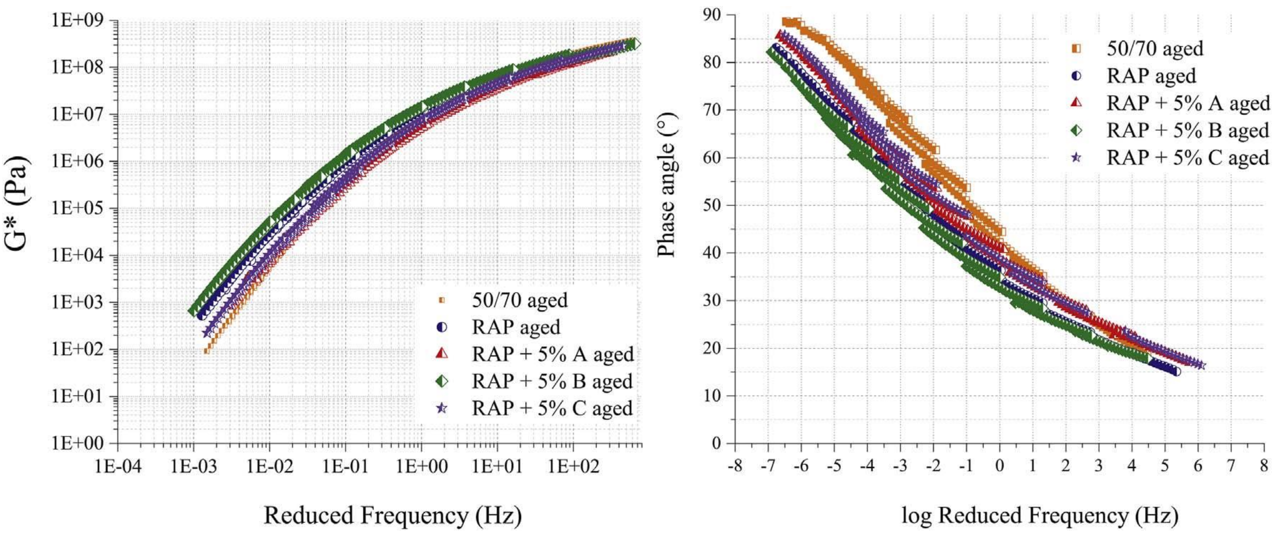


Figure 5. Dynamic shear moduli at 20°C (left) & phase angle at frequency sweeps (0.1 – 20 Hz) (Cavalli et al., 2018)

2.6 Performance Evaluation of Rejuvenated RAP Asphalt Mixtures

In the following subsections, a brief review of some selected works of literature on the laboratory performance properties of rejuvenated RAP asphalt mixtures was evaluated, then a concise summary of each performance properties was presented in Table 5 – 9. Further, a review of the field performance of rejuvenated RAP asphalt mixtures was conducted.

2.6.1 Effects of Rejuvenators on RAP Asphalt Mixtures

Elkashef et al. (2018) investigated the fatigue and thermal cracking behavior of rejuvenated RAP asphalt binders and mixtures. The rejuvenator used in the research is a soybean-derived rejuvenator, which is a bio-based rejuvenator like Anova asphalt rejuvenator. 12% dose of the rejuvenator was added to PG 58-28 asphalt binder used in the experiment before mixing the modified asphalt binder with an extracted RAP binder at a ratio 1:5. Dynamic shear rheometer (DSR) and bending beam rheometer (BBR) tests conducted on the rejuvenated RAP asphalt binder showed that the critical high and low temperatures were significantly lowered. Also, the BBR master curves showed a significant improvement in the creep compliance of the rejuvenated RAP asphalt binder, and the fatigue and Glover-Rowe parameters suggested an improvement in the fatigue cracking of the RAP asphalt binder. DCT test of specimens comprising of 100% RAP mixed with the modified PG 58-28 had higher fracture energy than specimens fabricated from 100% RAP and a neat PG 58-28 asphalt binder, when tested at -6^{0}C.

Xie et al. (2017) conducted a similar study to evaluate the effect of two different rejuvenators on the mechanistic and performance properties of recycled asphalt binders and mixtures, containing 25% RAP and 5% RAS. The research utilized a control mixture comprising 20% RAP and no rejuvenator, and two experimental mixes with 25% RAP and 5% RAS containing different rejuvenators. While the control mix was produced at the conventional hot-mix asphalt (HMA) temperature of 300^{o}F, the experimental mixtures were produced as warm-mix asphalt (WMA) at a temperature of 265^{o}F. Laboratory tests such as tensile strength ratio (TSR), asphalt pavement analyzer (APA), dynamic modulus, OT, etc. were conducted to examine and compare the performance of the control and experimental mixtures. TSR test results indicated that the experimental asphalt mixtures are less resistant to moisture damage, as compared to the control asphalt mixtures. The result of the APA rutting test suggests that for both the manual and automatic rut depths, the control mix has lower rutting resistance as compared to the experimental mixes, while the OT cracking result suggested that the experimental asphalt mixtures are less resistant to reflection cracking as compared to the control asphalt mixtures.

Tran et al. (2016) investigated the effects of a rejuvenator (Hydrogreen) on the performance of high RAP and RAS asphalt mixture. The three asphalt mixtures considered in the study includes: a control mix that comprises of 30% RAP and a SBS-modified PG 70-22 asphalt binder with no rejuvenator; a 40% RAP mix using the same PG 70-22 asphalt binder with a recycling agent; and a 25% RAP and 5% RAS mix using an unmodified PG 64-22 with a recycling agent. Laboratory performance tests such as Hamburg wheel-track (HWT), TSR, dynamic modulus, OT, etc. were carried out to evaluate and compare the moisture susceptibility, rutting resistance, and cracking resistance of the three asphalt mixtures. HWTT results showed that the 40% RAP and the control asphalt mixture yielded the lowest and highest rut depths, respectively. However, a one-way ANOVA statistical test with Tukey-Kramer statistical groupings that was conducted on the rutting results at a significance level (α) of 0.05, suggested that there is not enough evidence to show that a significant difference exists in the rutting resistance of the asphalt mixtures. For the cracking test results, the Texas OT results of the three asphalt mixtures showed similar average cycles to failure. However, the control mix recorded the highest number of cycles to failure.

Kodippily et al. (2016) examined the effects of different rejuvenators on the performance of 11 different RAP asphalt mixtures, which contains RAP in proportions of 15% and 30%. The results of the rutting tests showed that the rejuvenated RAP asphalt mixtures retained the high deformation resistance of RAP asphalt mixtures, especially the mixtures containing a higher proportion of RAP. On the other hand, the cracking resistance of the mixtures containing a higher proportion of RAP has lower cracking resistance. However, a significant improvement in the cracking resistance of the RAP mixes was recorded when rejuvenating agents were used. Further, the results of the research show that the use of a maltene fraction for RAP rejuvenation had a marginally better fatigue performance than a chemical rejuvenator.

The summaries of more studies conducted on the effect of rejuvenators on the stiffness, rutting resistance, cracking resistance, and moisture susceptibility of RAP mixtures are as presented in Table 5 – 8, respectively. From Table 5, the results of the research conducted on the influence of rejuvenators on RAP mixtures largely suggested that rejuvenators lower the stiffness of RAP mixtures.

Table 5. Effect of Rejuvenators on the Stiffness of RAP Mixtures (Martin et al., 2018)

Author(s) and Year of Pub.	Laboratory Test(s)	Main Findings
Mallick et al. (2010)	Dynamic Modulus	The addition of rejuvenators dropped the stiffness of 100% RAP mixture at high loading frequencies (5 and 10 Hz) but increased the stiffness at lower loading frequencies (1 and 0.1 Hz), at the highest testing temperature (54.4°C).
Uzarowski et al. (2010)	Dynamic Modulus	The rejuvenator effectively decreased the stiffness of the RAP mixtures.
O'Sullivan (2011)	Dynamic Modulus	The rejuvenator decreased the stiffness value of 80, 90, and 100% RAP mixtures to a stiffness value below the virgin mixture.
Tran et al. (2012)	Dynamic Modulus	• After short-term aging, the rejuvenator reduced the stiffness of the RAP/RAS mixtures to a stiffness close to that of the virgin mixture. • After long-term aging, the rejuvenated mixtures seem to age faster than the RAP/RAS mixtures without rejuvenator.
Mogawer et al. (2013)	Dynamic Modulus	The stiffness of the rejuvenated mixtures is close to that of the virgin mixture. Rejuvenated mixtures with RAS and RAP/RAS showed less significant reduction in stiffness after incorporating a rejuvenator, as compared to a RAP only rejuvenated mixtures.

Im et al. (2014)	Dynamic Modulus	Reduction in the stiffness of the rejuvenated mixtures at high testing temperature (40°C) and low frequency ranges but did no significant effect on the stiffness at lower temperatures (4 and 20°C).
Alavi et al. (2015a)	Dynamic Modulus	The addition of a petroleum-based rejuvenator decreased the stiffness of fine aggregate mixtures (FAM) with 25 and 40% RAP and 15% RAS for five different base binders.
Haghshenas et al. (2016)	Dynamic Modulus	The petroleum-based rejuvenator is more effective than soybean oil and tall oil in reducing stiffness.

From Table 6, the rutting resistance results of some researchers suggested that moisture susceptibility of RAP mixtures increases when a rejuvenator is added, while some say otherwise. Im et al. (2014), however, attributed the rutting susceptibility of rejuvenated RAP mixtures to the type and dosage of the rejuvenator.

Table 6. Effect of Rejuvenators on the Rutting Resistance of RAP Mixtures (Martin et al., 2018)

Author(s) and Year	Laboratory Test(s)	Main Findings
Uzarowski et al. (2010)	APA	Significant decrease in rutting resistance.
Lin et al. (2011)	Marshall Stability	As the rejuvenator dosage increases from 10 to 40%, the reduction in rejuvenated mixture stability ranged from 25% to 55% depending on the type of rejuvenator.
Tran et al. (2012)	APA	Recycling agent addition increased mixture susceptibility to rutting, but with rut depths less than 5.5 mm to withstand at least 10 million ESALs.
Mogawer et al. (2013)	Hamburg Wheel Tracking Test (HWTT)	Decrease in rutting resistance of rejuvenated RAP/RAS mixtures.
Im et al. (2014)	HWTT	Rutting and moisture susceptibility of the rejuvenated mixtures RAP/RAS mixes depend on the type and dose of rejuvenating agent.
Tran et al. (2016)	HWTT	The rejuvenated RAP mixture was more rut-resistant than the RAP mixture without rejuvenator.
Kodippily et al. (2016)	HWTT	The rejuvenated RAP mixture was more resistant to rutting than the RAP mixture without a rejuvenating agent.

From Table 7, it can be observed that the use of rejuvenators in RAP mixture generally yields better cracking performance. Some of the researchers also noted the significance of the type of rejuvenator used in RAP mixes.

Table 7. Effect of Rejuvenators on the Cracking Resistance of RAP Mixtures (Martin et al., 2018)

Author(s) and Year of Pub.	Laboratory Test(s)	Main Findings
Lin et al. (2011)	Indirect Tensile (IDT)	Rejuvenator improved the cracking resistance of the RAP mixture, depending on the type of rejuvenator.
Tran et al. (2012)	Energy Ratio Test	Improvement in the fracture energy of rejuvenated RAP mixtures.
	Texas Overlay (OT)	The rejuvenator improved the average number of cycles to failure of the RAP mixture.
Mogawer et al. (2013)	OT	Improvement in the cracking performance of rejuvenated RAP/RAS mixtures, depending on the type of rejuvenator.
Im et al. (2014)	OT	Addition of rejuvenator increased the average OT number of cycles to failure, from approximately 110% to 300%, depending on the type of rejuvenator.
Tran et al. (2016)	OT	The mixture containing 30% RAP with no rejuvenator was more resistant to fatigue than the mixture containing 45% RAP and rejuvenator.
Nabizadeh et al. (2017)	Illinois Flexibility Index Test (I-FIT)	Recycling agent addition increased the flexibility index (FI). Aromatic extract was more effective than tall oil and soybean oil.
Elkashef et al. (2018)	Disc-Shaped Compact Tension (DCT) Test	Fracture energy of the rejuvenated mixture containing 100% RAP was higher than the mix without rejuvenator.

The summary of the past researches on the effect of rejuvenators on the moisture susceptibility, as presented in Table 8, suggests that no consensus exists, yet.

Table 8. Effect of Rejuvenators on the Moisture Resistance of RAP Mixtures (Martin et al., 2018)

Author(s) and Year of Pub.	Laboratory Test(s)	Main Findings
Shen et al. (2007)	Tensile Strength Ratio (TSR) Test AASHTO T283	Mixtures containing RAP, combined with a soft asphalt binder or a rejuvenator, had the same level of moisture susceptibility as the asphalt mixture without rejuvenator.
Tran et al. (2012)	TSR	The addition of the recycling agents to the RAP mixtures did not negatively affect the TSR values but slightly increased them.
Yan et al. (2014)	TSR	TSR value decreased with increasing RAP contents. Rejuvenator type is a factor that has a significant effect on the moisture susceptibility of the rejuvenated RAP mixture.

2.7 Field Investigation of the Performance of Rejuvenated RAP Asphalt Mixtures

Tran et al. (2016) investigated the effects of a rejuvenator (Hydrogreen) on the performance of three mixes, including: a control mix that comprises of 30% RAP and a SBS-modified PG 70-22 asphalt binder with no rejuvenator; a 40% RAP mix using the same PG 70-22 asphalt binder with a recycling agent; and a 25% RAP and 5% RAS mix using an unmodified PG 64-22 with a recycling agent. The three RAP mixtures were constructed as part of NCAT's sixth research cycle. The early field performance (10 months after construction) of the three mixtures indicated no significant rutting. However, low-severity reflective cracking was observed in all three Sections, which is made up of the control mix and the rejuvenated RAP mixes. The international roughness index (IRI) value of the rejuvenated RAP mixes was classified as "good," and the observed rutting was low after two years in service. The control mixture had good field cracking performance, while the two experimental mixes exhibited premature field cracking.

The premature field cracking failure may be partially attributed to the Superpave mix design approach implemented in the research study, where performance verification of the final mix design was not conducted. To that effect, this thesis used the BMD method for the design of the control and experimental RAP mixes. The methodology of the research is fully explained in the next chapter of this report.

3 CHAPTER THREE – METHODOLOGY

This chapter covers the experimental plan of the research study, materials used, mix design, quality control of asphalt mix production, laboratory performance testing of the reheated plant-produced mixes, and the field performance data.

3.1 Experimental Plan

The experimental plan of this research study is structured into four tasks, which includes; mix design, mix production and construction, laboratory performance testing, and early field performance. The sequential arrangement of the experimental plan is illustrated in Figure 6. The execution of the mix design was done by following the Virginia Department of Transportation (VDOT) provisional BMD specifications, as summarized in Table 9.

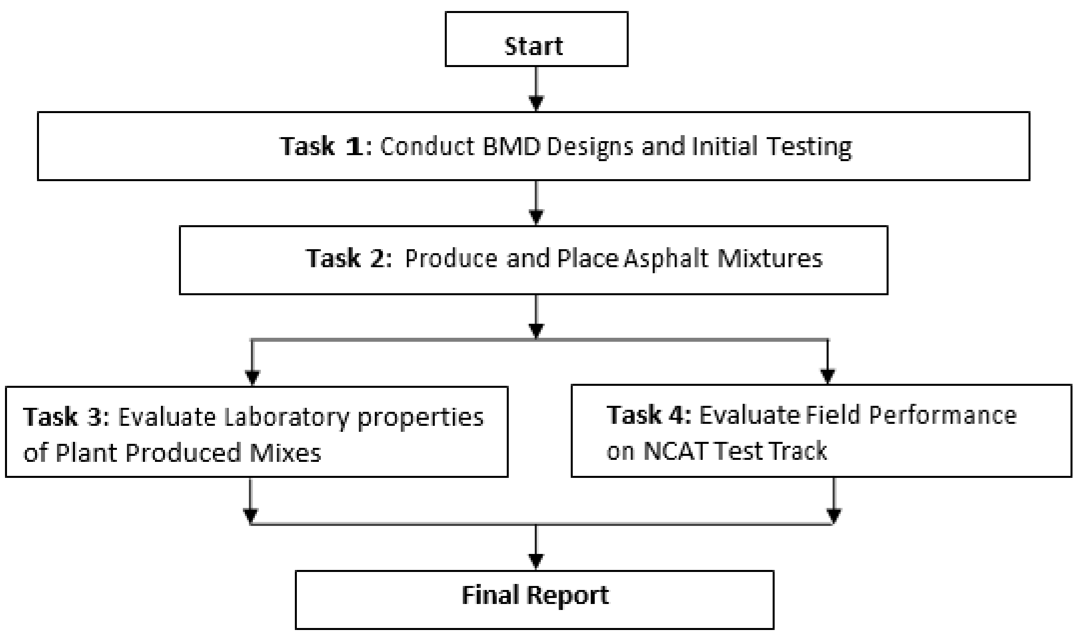


Figure 6. Experimental Plan

Table 9. Performance Testing Requirements by VDOT for High-RAP Surface Mixtures

Test	**Procedure**	**Specimens**	**Criteria**
APA rutting	8,000 cycles; 64ºC; 120lb; 120psi	2 replicates of 2 pills (APA Jr), 150 mm dia., 75 ± 2 mm ht., and 7 ± 0.5% air voids [Note: T340 recommends avoidance of reheating plant mix]	Rutting ≤ 8.0mm
Cantabro	300 rotations, 30-33 rotations/min.	3 replicates, 150 mm dia., 115 ± 5 mm ht., compact to N_{design}, report air voids. [Lab-produced mix – subject loose mix to 4 hours at 135 ºC STOA prior to compacting]	Mass loss ≤ 7.5%
CT_{index}	Condition specimens 25 ± 1°C for 2 ± 0.5 hr; Apply contact load of 0.1 ± 0.02 kN with loading rate of 0.05 kN/s; After contact reached, load using load-line displacement control at 50 mm/min.; record load to peak and through failure.	3 replicates, 150mm dia., 62 ± 2mm ht., and 7±0.5% air voids [Lab-produced mix – subject loose mix to 4 hours at 135 ºC STOA prior to compacting]	CT_{index} ≥ 70

3.2 Materials

The materials utilized in this research study are asphalt binder, aggregates, RAP, anti-strip additive, and Anova asphalt rejuvenator. The combination of some of the materials forms the building block of the control and experimental mixtures.

3.2.1 Asphalt Binder

The selected virgin asphalt binder grade of the control and experimental mixes in this research study is PG 64 – 22. The selected asphalt binder grade was recommended by VDOT based on the historical performance data of asphalt mixes in Virginia.

It is considered a good practice to extract, recover, and grade asphalt binder when the RAP content in the asphalt mixture is intermediate or high. In this research study, the use of 30% RAP in the control mix and 45% RAP in the experimental mixes made it necessary to extract, recover, and grade the recovered asphalt binder.

The extraction of the asphalt binder was done by following the ASTM D2172 test procedure (centrifuge method). ASTM D2172 test procedure utilizes a solvent-based extraction technique where solvents such as trichloroethylene (TCE) is used to extract asphalt binder from the RAP mixes.

The extracted asphalt binder was recovered by following the ASTM D1856 test procedure, which covers the recovery of asphalt from a solution by the Abson method. The recovered asphalt binder was subjected to a two-level aging procedure. The first level is the standard aging procedure that follows the AASHTO M320 specification. The second level aging procedure is an extended aging process where the recovered asphalt binder was subjected to twice the standard PAV aging (2 x PAV aging). The recovered asphalt binder was subjected to the extended aging process because a research study by Swiertz et al. (2011) suggested that 2 x PAV aging reveals the properties of aged RAP binder. The aging procedure of the recovered asphalt binder is summarized in Table 10. The recovered asphalt binder was graded by following the standard test procedure in ASTM D7643 and the guidelines provided in the NCHRP 452 report by McDaniel & Anderson (2001).

Table 10. Aging Procedure

Aging Level	Description
HTPG: As Extracted LTPG: As Extracted + TFO	Standard Aging Method, calibrated to correspond to standard M320 grades
HTPG: As Extracted + TFO + PAV LTPG: As Extracted + TFO + PAV	Additional PAV testing, reflecting an extended (2 x PAV) aging of asphalt binder

3.2.2 Aggregates

Chemung Trap Rock (TR) aggregate stockpiles, which include #8s', cleaned #10s', and #10s' (dust) were mixed with a single source of RAP, supplied by a contractor in Virginia, to produce a 30% RAP mixture (control mix) and two 45% RAP mixtures (experimental mixes). The NMAS of the final aggregate blend of both the control and experimental mixtures is 9.5mm. The aggregate gradation charts of the 30% and 45% blends are illustrated in Figure 7 & 8, respectively. It is worthy of note that the mix design of the first experimental mix (45% RAP mix with rejuvenator) was implemented in the second experimental mix (45% RAP mix without rejuvenator).

The RAP used in the experimental mix was fractionated into coarse RAP (CRAP) and fine RAP (FRAP), while the RAP in the control mix was not fractionated. The fractionation of the RAP in the experimental mix was necessary to achieve a design gradation that is AASHTO M323-07 compliant, as shown in Figure 8. Aggregate gradation and the final aggregate blend of the control and experimental mixtures are presented in Table 11 and 12, respectively.

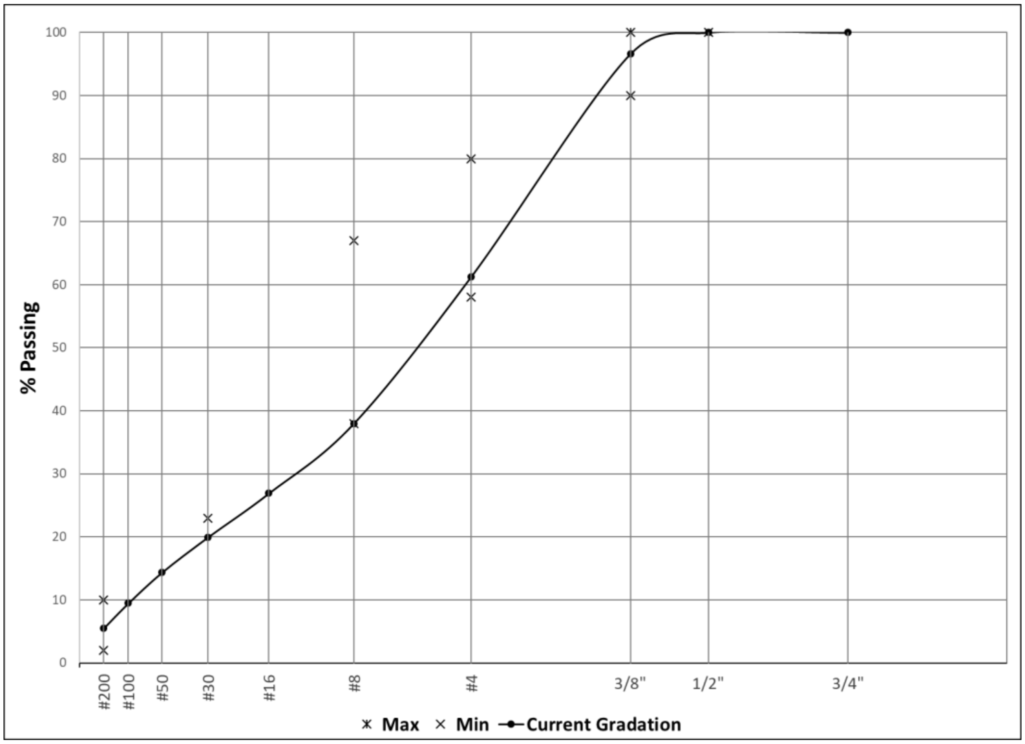


Figure 7. 0.45 Power Chart of Control Mix

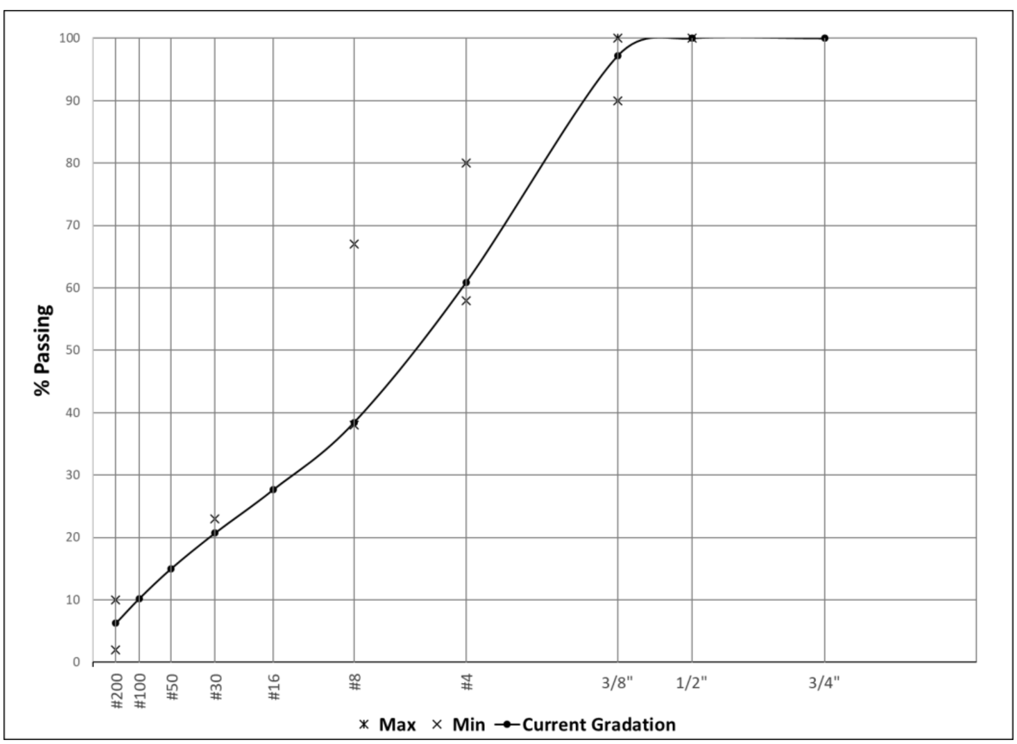


Figure 8. 0.45 Power Chart of Experimental Mix

Table 11. Aggregate Gradation of the Control Mix

Material	Aggregate Blend	Aggregate Gradation (% Passing Each Sieve)									
		3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
TR #8	33%	100.0	100.0	90.8	27.7	8.0	6.1	5.7	5.2	4.4	3.1
TR #10 (Clean)	36%	100.0	100.0	100.0	95.0	64.2	42.7	28.9	18.7	9.6	2.9
TR #10's Dust	1%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
RAP	30%	100.0	100.0	98.8	56.5	37.5	28.5	22.2	16.4	11.9	8.2
Blend	100%	100.0	100.0	96.6	61.3	38.0	26.9	19.9	14.4	9.5	5.5

Table 12. Aggregate Gradation of the Experimental Mix

Material	Aggregate Blend	Aggregate Gradation (% Passing on Each Sieve)									
		3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
TR #8	25%	100.0	100.0	90.8	27.7	8.0	6.1	5.7	5.2	4.4	3.1
TR #10 (Clean)	29%	100.0	100.0	100.0	94.9	64.0	42.4	28.5	18.3	9.1	2.4
TR #10's Dust	1%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
FRAP	25.4%	100.0	100.0	100.0	100.0	66.3	50.2	39.0	28.8	21.0	14.3
CRAP	19.6%	100.0	100.0	97.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Blend	100%	100.0	100.0	97.2	60.9	38.4	27.6	20.7	15.0	10.2	6.3

3.2.3 Rejuvenators and Anti-Strip Additive

The rejuvenator used in this research study is the bio based Anova 1815 rejuvenator that was formulated by Cargill. Anova 1815 rejuvenator is a chemically modified vegetable oil-based recycling agent. It improves the low-temperature performance of aged asphalt binders, thereby allowing the use of more RAP in asphalt mixtures. In the laboratory, Anova 1815 was added directly to the asphalt binder, followed by a low shear blending for 3 – 5 minutes to achieve homogeneity. At the asphalt plant, the manufacturer recommended the injection of the rejuvenator into the asphalt binder tank before mixing.

As previously mentioned, the control mix has no rejuvenator. However, it contains a non-toxic warm-mix additive named Anova 1501, which was also produced by Cargill. The additive enhances asphalt mixture resistance to moisture damage, workability, compactability, and facilitates compaction at lower temperatures. Cargill recommends a dosage rate in the range of 0.2 – 0.7% by total weight of the asphalt binder.

3.3 Mixture Design

In this research project, the basic performance tests used in the BMD are APA, IDEAL-CT, and Cantabro abrasion test, as specified in VDOT's provisional BMD specification for high-RAP mixes. The three main approaches to implementing a BMD includes (1) volumetric design with performance verification; (2) performance-modified volumetric design; and (3) performance design. A flowchart of how each BMD approach is executed is illustrated in Figure 9. The BMD approach used in this research project is the "performance-modified volumetric design." In this approach, volumetric analysis is performed to determine the initial binder content, then performance tests such as APA rutting test, IDEAL-CT, and Cantabro abrasion test are conducted. If the mix passed the performance tests, moisture susceptibility of the mix is evaluated, and the volumetric properties are verified. However, if the mix failed the performance tests, the mix and or asphalt binder content is adjusted before running another set of a performance test. The final volumetric properties may be allowed to drift outside existing AASHTO M323 limits.

In addition to the three performance tests specified by VDOT for BMD, other asphalt mixture performance tests, as summarized in Table 13, are required for evaluating the unaged and aged laboratory performance of the control and experimental mixes. However, the scope of this report is limited to the laboratory performance evaluation of the unaged or reheated plant-produced control and experimental mixtures.

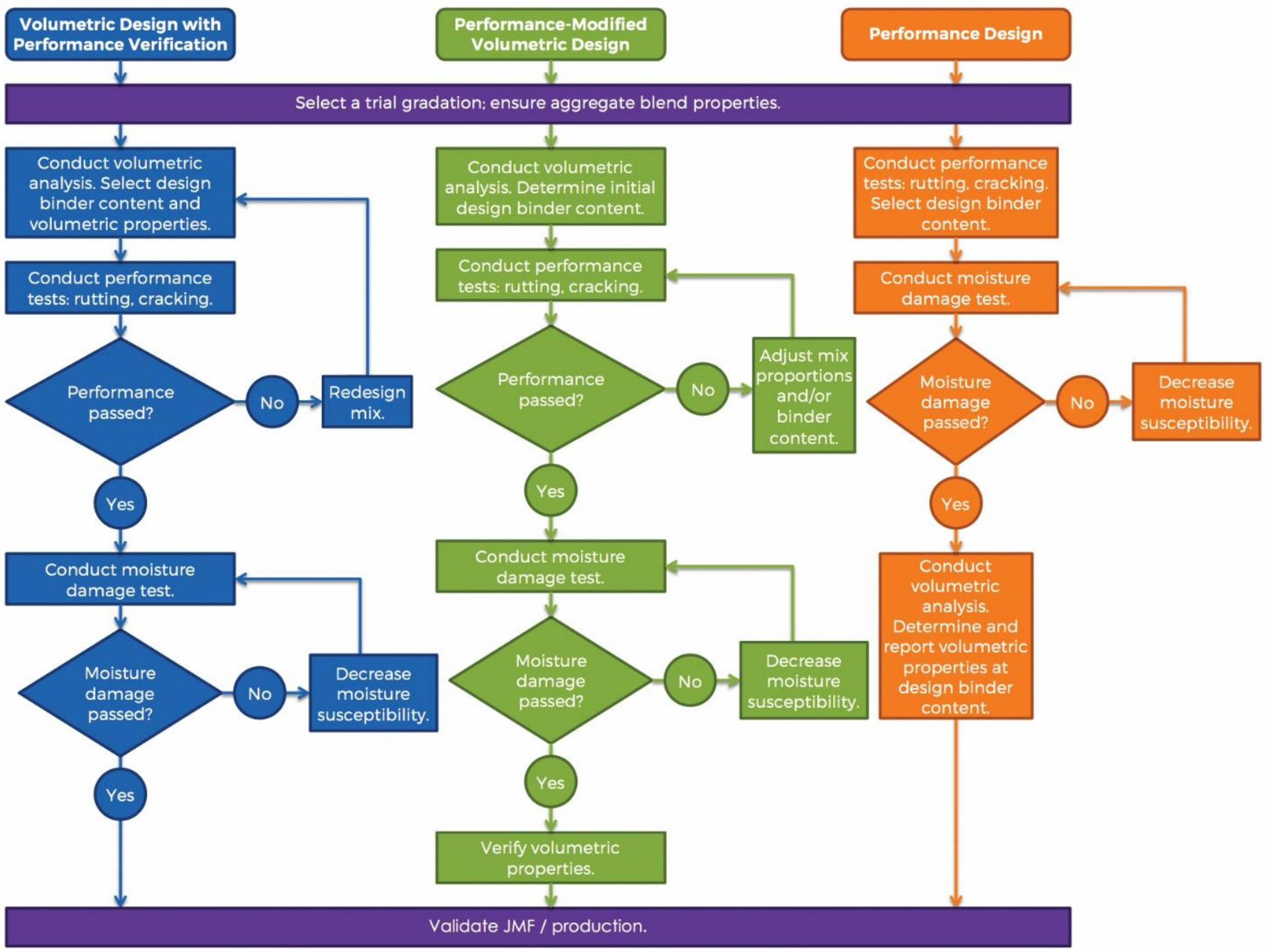


Figure 9. BMD Approaches

Table 13. Testing Schedule

Test	30% RAP		45% RAP Anova		45% RAP no Anova	
	Unaged	Aged	Unaged	Aged	Unaged	Aged
Mixture Tests (by NCAT)						
Virginia Specifications						
IDEAL (VDOT)	x	x	x	x	x	x
APA (VDOT)	x		x			
Cantabro	x		x		x	
Other States Specifications						
OT (NJ)	x	x	x	x	x	x
I-FIT (AASHTO TP 124)	x	x	x	x	x	x
DCT (ASTM D7313)	x	x	x	x	x	x
TSR (AASHTO T283)	x		x			

Hamburg (AASHTO T 324)	x		x			
Binder Tests (by Cargill)						
Extract/recovery/PG*	x	x	x	x	x	x

*Includes RAP and base asphalt binder sampled during construction.

3.4 Quality Control of Asphalt Mix Production and Construction

Quality Control (QC) is an integral component of asphalt pavement construction that serves as a tool for tracking the compliance of the production mix with the job mix formula (JMF). In this research study, QC tests were conducted on randomly sampled control mix and the experimental mix, which contains 45% RAP and Anova 1815 rejuvenator. The control and experimental mixes were laid in Sections N3A and N3B of the NCAT Test Track, respectively. Asphalt binder was extracted and separated from the aggregates of each sampled mix. Then, the aggregates of each mix were graded, and the volumetric properties of each recovered asphalt binder were determined.

3.5 Performance Tests

3.5.1 Asphalt Binder Test

As previously described, asphalt binders were extracted and recovered from each of the three plant-produced RAP mixes. The extracted asphalt binders were recovered and graded using the asphalt binder performance tests specified in AASHTO M320. Since the recovered asphalt binders were subjected to a two-level aging procedure (standard aging and 2 x PAV aging), the performance grade (PG) of the three plant-produced RAP mixes are expected to be different at each aging level.

The high and low-temperature performance grades of the recovered asphalt binders, at the standard and extended aging levels, were selected by following the recommendations provided in NCHRP 452 report. The selected critical high-temperature PG of the recovered asphalt binder is the lower of the original DSR and the RTFO DSR critical temperatures, while the selected critical low-temperature PG is the higher of the BBR stiffness and m-value critical temperatures. The difference between the critical temperatures of the BBR stiffness and m-value yields the parameter ΔT_C, which was first proposed by Anderson (2011) as a measure of evaluating the non-load related cracking resistance of asphalt binders. The minimum ΔT_C criterion for asphalt mixtures containing RAS is -5^0C, but the minimum criterion for asphalt mixtures containing RAP is yet to be established.

3.5.2 Moisture Susceptibility Test

Moisture susceptibility test was conducted for this research study by following the AASHTO T283 procedure. The objective of this test is to evaluate the effects of saturation and accelerated water condition, with a freeze-thaw cycle, on compacted asphalt mixtures. In other words, it is a test of the moisture susceptibility of compacted asphalt mixtures.

The six specimens used in this experiment are Superpave Gyratory Compacted (SGC) specimens, which were compacted to a height of 95mm and an air void of 7 ± 0.5%. There is no loose mix curing since the control and experimental mixtures are plant produced reheated asphalt mixtures. The 6 SGC specimens are divided into two subsets, with each subset containing three specimens. One subset is tested in a dry condition for indirect tensile strength (ITS), while the other subset is tested in wet condition after moisture conditioning, as mentioned in AASHTO T283 and illustrated in Figure 10. The tensile strength ratio (TSR), which is a measure of moisture susceptibility of the asphalt mix, is calculated using equation 1.

$$TSR = \frac{S_2}{S_1} \qquad \text{Equation (1)}$$

Where: S_1 = Average tensile strength of the dry subset, psi (kPa)

S_2 = Average tensile strength of the conditioned subset, psi (kPa)

Figure 10. TSR Test Setup

3.5.3 Asphalt Pavement Analyzer (APA)

Figure 11 shows the APA laboratory testing device for measuring the permanent deformation or rutting resistance of compacted asphalt mixtures in dry condition. The APA run a loaded aluminum wheel back and forth across a pressurized linear hose over an SGC specimen. The standard protocol for the test method can be found in AASHTO T340. However, for this research study, the AASHTO T340 APA testing procedure and criterion were modified to match Virginia DOT (VDOT) requirements for both the BMD and re-heated mixes, as summarized in Table 9. VDOT's test procedure for APA requires 8,000 passes for each test, using a 120 lb. load and a 120 psi hose pressure at a temperature of 64^{0}C. Six cylindrical specimens with a diameter of 150 mm (6 in.), 75 ± 2 mm thick, and an air void of 7 ± 0.5% are required to conduct the test. VDOT's APA testing criterion is that rut depth must not exceed 8 mm upon the completion of the test (i.e., rutting ≤ 8 mm).

Figure 11. APA Testing Equipment and Specimen

3.5.4 Hamburg Wheel Tracking Test (HWTT)

The HWTT is a performance testing device for measuring the susceptibility of an asphalt mix to both rutting and moisture damage. The Hamburg Wheel Tracking Device (HWTD), depicted in Figure 12, was developed using the concept of a similar British device which uses rubber instead of 158 lb. (705 N) steel wheels (Rahman & Hossain 2014).

The standard procedure for the HWTT is the AASHTO T324. In this experiment, four SGC specimens were fabricated for testing, where two of the SGC specimens are reheated control asphalt mixture (i.e., N3A-RH), and the remaining two are reheated experimental asphalt mixture with Anova rejuvenator (i.e., N3B-RH). The SGC specimens have a diameter of 150 mm (6 in.), a height of 63 mm (approx. 2.5 in), and placed side by side, while fully submerged in water at a temperature between 104^0F (40^0C) and 140^0F (60^0C). The SGC specimens were subjected to a wheel speed of 52 passes per minute. Linear variable differential transformers (LVDTs) measure the rut depth at 11 points along the length of each SGC specimen to an accuracy of 0.01 mm.

The HWTD automatically comes to a halt when the default number of wheel passes, usually, 20,000 passes, is completed or a rut depth of 20 mm (approx. 0.8 in.) is recorded, depending on whichever occurs first. The testing time is about 6.5 hours, excluding the initial wait time of 30 minutes.

Figure 12. HWTD

3.5.5 Indirect Tension Asphalt Cracking Test (IDEAL – CT)

Zhou et al. (2017) came up with a cracking test, IDEAL – CT, which addresses the difficulties associated with most existing cracking tests procedure. The IDEAL – CT does not require cutting, drilling, notching, instrumentation, excessive temperature conditioning and testing time. The test utilizes the concept of indirect tension test, as illustrated in Figure 13. The only instrumentation required in the IDEAL – CT is a load cell and displacement transducer for monitoring the force and movement of the loading crosshead, as delineated in Figure 14 (Newcomb and Zhou, 2018).

In this research study, the IDEAL – CT test was utilized extensively in the mix design and the performance evaluation of the reheated plant-produced asphalt mixtures. From the VDOT BMD provisional specification, the IDEAL – CT procedure involves the conditioning of SGC specimens compacted to a height of 62 ± 2 mm, a diameter of 150 mm, and an air void of 7 ± 0.5%, for 2 ± 0.5 hr. at a temperature of 25 ± 1^0C before testing.

Figure 13. IDEAL – CT Setup

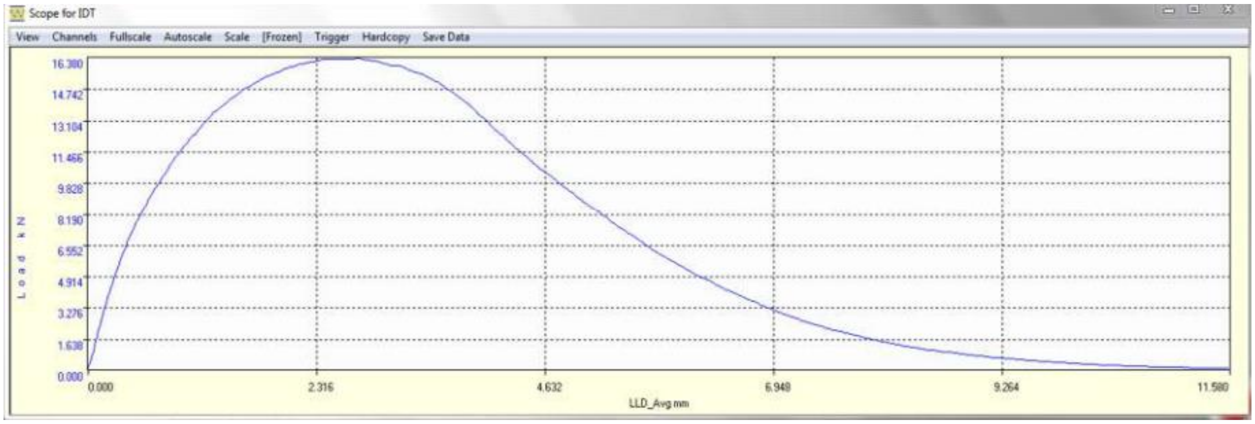


Figure 14. Force-Displacement (F-D) Curve for IDEAL – CT

In the conventional Indirect Tension (IDT) test, the maximum stress is used in analyzing the test results. In IDEAL – CT, however, both the maximum stress and the F-D curve are used for the analysis. As presented by Zhou et al. (2017), the derivation of the fracture mechanics principles used in developing the CT_{index} is a function of the SGC specimen thickness, fracture energy (G_f), displacement at 75 percent of the peak load, and the diameter of the SGC specimen. The analysis simply requires the computation of the area under the F-D curve to determine G_f and other parameters. An Excel spreadsheet template is available for automating the CT_{index} calculation. The larger the CT_{index}, the better the cracking resistance of the mix (Newcomb and Zhou, 2018).

3.5.6 Illinois Flexibility Index Test (I-FIT)

The I-FIT is a variation of the Semi-Circular Bend (SCB) test for thermal cracking, which was proposed by the University of Illinois (Al-Qadi et al., 2015). Unlike the SCB test, the I-FIT uses the vertical displacement of the loading head instead of the load-line displacement (LLD), and the test is conducted at a temperature of 25^0c and a vertical crosshead speed of 50mm/min (Newcomb and Zhou, 2018).

The I-FIT equation is given as:

$$G_f = \frac{W_f}{a_{lig}} \qquad \text{Equation (2)}$$

$$FI = A(\frac{G_f}{abs(m)}) \qquad \text{Equation (3)}$$

Where; FI = Flexibility Index

A = Calibration coefficient (0.01 for unaged mixture).

G_f = Work of the fracture energy (W_f) to peak load (J/m^2)

W_f = Work of fracture (J)

A_{lig} = Ligament area (mm^2)

Abs (m) = Absolute value of the post-peak slope of the inflection point.

Figure 15 is a typical graphical illustration of the I-FIT result. A high FI, especially in polymer modified asphalt mix, indicates a high cracking resistance in the mix. The FI can be related to failure modes of the SGC asphalt specimens, with high FI suggesting a ductile failure, while a low FI suggests a brittle failure. According to Al-Qadi et al. (2015), coefficient of variation (COV) in I-FIT test results ranges from 4.2 – 21.3 percent with an average of approximately, 10 percent. The comprehensive description of the I-FIT standard procedure can be found in AASHTO TP 124. The I-FIT was utilized in both the BMD performance verification test and the performance evaluation of the reheated asphalt mixtures.

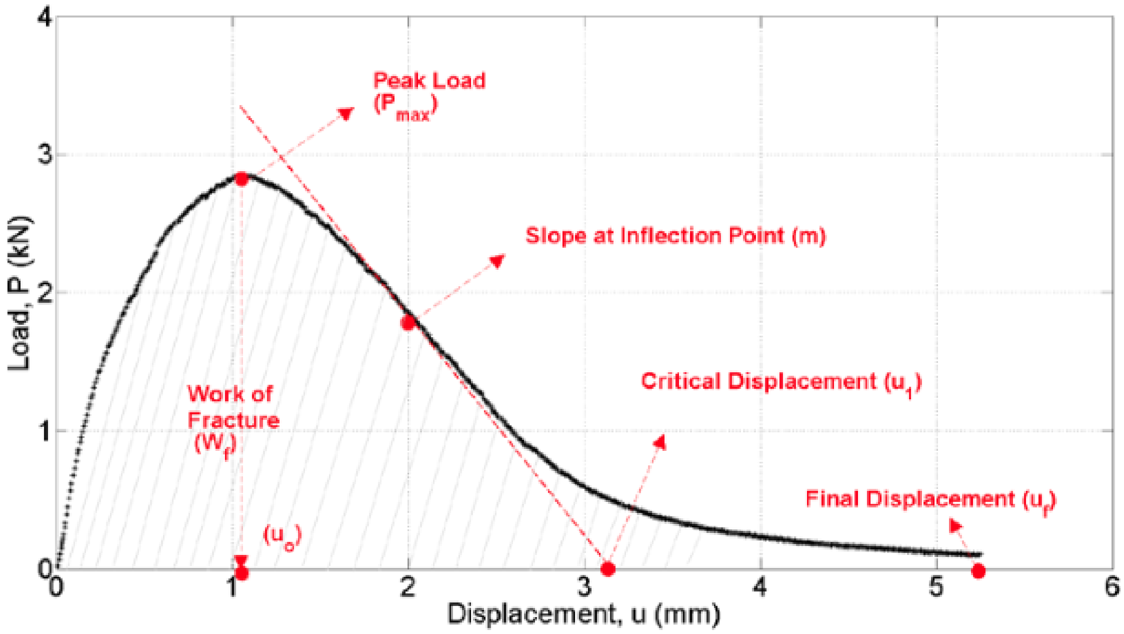


Figure 15. Graphical Illustration of the I-FIT Parameters (Newcomb and Zhou, 2018)

3.5.7 Texas Overlay Test

The Texas Overlay Test (OT) is commonly used to estimate traffic-related top-down cracking, and the resistance of an asphalt mix to reflective cracking, especially in overlays. For a given OT, the higher the number of cycles to failure, the higher the resistance of the asphalt mixture to cracking.

The OT was originally developed by Germann and Lytton (1979), to simulate the contraction and expansion mechanism of joints or cracks, which are responsible for inducing reflective crack initiation and propagation. The original concept of the OT was eventually modified by Zhou and Scullion (2005), to make it applicable as a cracking test in asphalt overlays.

The OT is a fully computer-controlled system with special programs. The test data including time, displacement, and force are automatically recorded and saved as an Excel file. The sample size is 6in. (150 mm) long by 3in. (75 mm) wide by 1.5 – 2 in. (38 – 50 mm) high. The OT specimens using in this research study were glued to the aluminum plates using two-part epoxy, and a minimum of four specimens was tested for the control and experimental asphalt mixtures.

Figure 16 & 17 show the schematic diagram and photos of the new Texas Transportation Institute (TTI) OT, respectively. The OT test procedure can be found in the TxDOT Tex-248-F, which has been in use since 2009, revised in 2014, and currently in the process of acquiring an ASTM standardization.

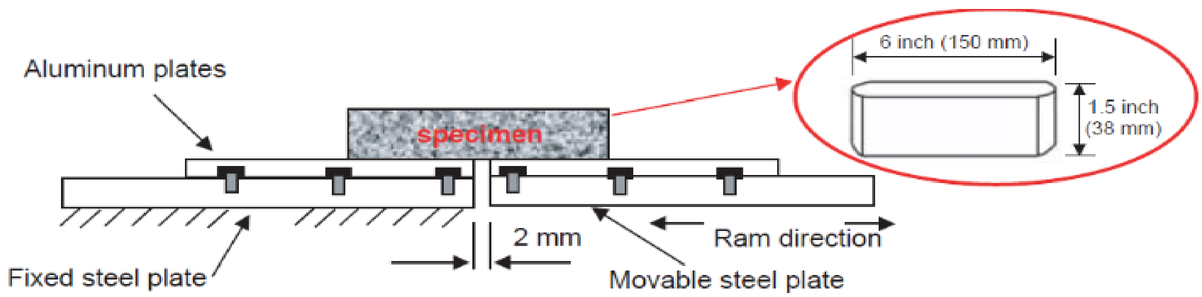


Figure 16. Schematic of TTI OT (Newcomb and Zhou, 2018)

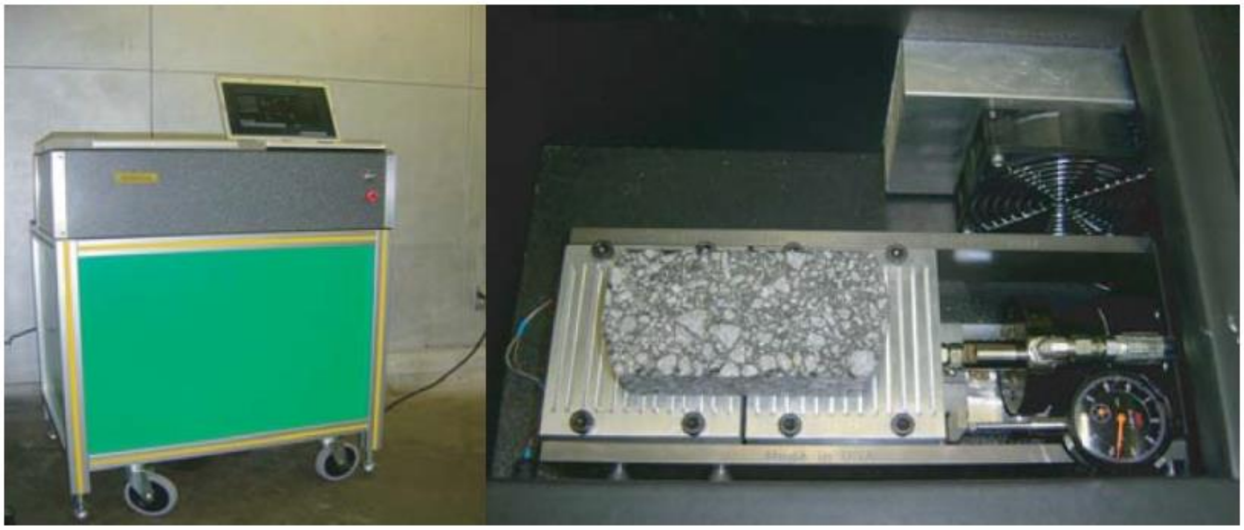


Figure 17. Testing Equipment and Setup of the OT (Newcomb and Zhou, 2018)

3.5.8 Disc-Shaped Compact Tension (DCT) Test

Wagoner et al. (2006) developed the DCT test for characterizing cracking resistance of asphalt mixtures at low temperatures. The DCT test procedure can be found in ASTM D7313. The geometry of the DCT specimen is a 6in. (150mm) diameter, 2in. (50mm) thick overall dimension with two 1in. (25mm) holes on either side of a 2.46in. (62.5mm) notch cut into a flattened portion of the circumference, as shown in Figure 18. The DCT test is often conducted at 18^0F (10^0C) warmer than the PG low temperature grade in a crack-mouth opening displacement (CMOD) controlled mode with an opening rate of 0.04in/min (1mm/min) (Newcomb and Zhou, 2018). However, it is worthy of note that a major concern in the DCT test is the sample fabrication.

Figure 19 shows a typical DCT test curve. The fracture energy (Gf) is calculated by determining the area under the Load-CMOD curve normalized by the initial ligament length and thickness. The bigger the Gf, the better the cracking resistance of the asphalt mixture.

It is relatively easy to calculate fracture energy. However, a data analysis program or Excel Macro is required since the integration of the test result curve is involved. Presently, DCT test results, in terms of fracture energy, is interpreted by comparing the results obtained with an established pass/fail criterion for thermal cracking (Newcomb and Zhou, 2018).

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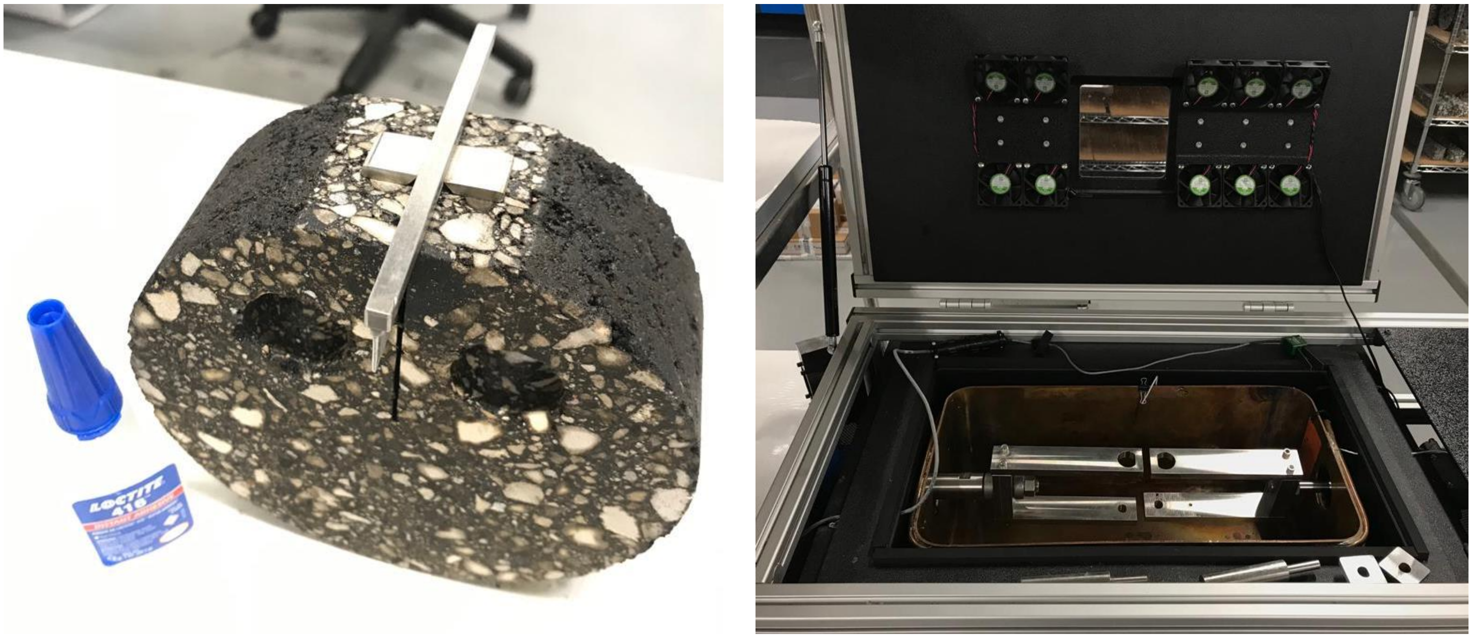

Figure 18. DCT Test Specimen and Equipment

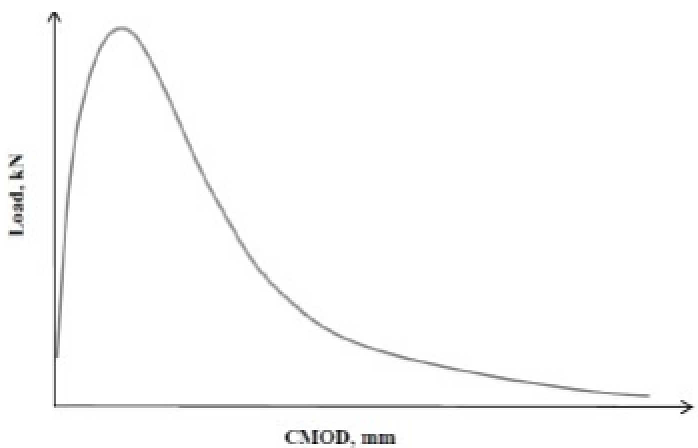

Figure 19. Typical DCT Test Curve (Marasteanu et al. 2012)

3.5.9 Cantabro Abrasion Test

Cantabro abrasion test is necessary for addressing raveling, which is a common distress in rejuvenated asphalt mixtures (Pan et al., 2018). In this research, Cantabro abrasion test was conducted on SGC specimens of the control and experimental mixes, which were fabricated and tested by following the VDOT's BMD provisional standard specification. The mass loss ratio of each SGC specimen, before and after the test, was utilized in evaluating the resistance of the control and experimental asphalt mixtures to particle loss, as presented in Equation 4. The standard specification for the Cantabro abrasion test is ASTM D7064-08. However, in this research study, the standard specification was slightly modified to fit the VDOT's BMD provisional standard specification, as summarized in Table 10. Figure 20 shows the Cantabro abrasion SGC specimens of the control and experimental mixes after testing.

$$mass\ loss = \left(1 - \frac{weight\ after\ test}{weight\ before\ test}\right) \times 100 \qquad \text{Equation (4)}$$

Figure 20. Cantabro SGC Specimens

3.6 Early Field Performance Data

As previously stated, the control mix with 30% RAP and no rejuvenator, and the first experimental mix with 45% RAP and Anova asphalt rejuvenator were produced by east Alabama paving (EAP) company in Opelika, AL. The asphalt mixtures were placed in Sections N3A and N3B of the NCAT Test Track as a part of the seventh research cycle. The NCAT Test Track is a 1.7 miles closed-loop accelerated pavement testing facility that comprises of 46 test sections. Each test section is 200ft long, and a fleet of five (5) Trucks that weigh 156,995lb each, on the average, runs throughout the entire Test Track round the clock. An estimated 10 million ESAL is to be applied on the Test Track by the end of the 3 years research cycle.

Traffic is suspended on the Test Track every Monday to facilitate the collection of rutting, ride quality, and surface crack mapping data. The standard metric for measuring the ride quality of a pavement surface is the international roughness index (IRI). IRI is measured at the Test Track using the Dynatest Mark IV inertial profiler shown in Figure 21. Rutting is measured at the Test Track using the ALDOT beam procedure, as explained in the ALDOT T-392 standard specification. Figure 22 depicts the ALDOT beam, where the testing procedure involves the utilization of a 4-foot beam with a dial gauge to measure rut depths along the wheel path at predetermined locations in each test section of the Test Track. The accuracy of the rut depths obtained using the ALDOT beam method is estimated at ± 2.5mm.

Figure 21. Inertial Profiler (Giler, 2017)

Figure 22. ALDOT Beam (Giler, 2017)

The cracking performance data is obtained by initially carrying out visual inspection of the test section, then the observed surface cracks are mapped and measured, as illustrated in Figure 23. Area of the cracked section is determined by conducting a linear measurement of the cracks within the test section. The measurements obtained are used to facilitate the calculation of the percentage of cracked section (Giler, 2017).

Figure 23. Crack Mapping on the Test Track (Giler, 2017)

4 CHAPTER 4 – RESULTS AND DISCUSSIONS

This chapter of the report covers a clear description of the research study results. The results comprise of the outcome of the mix design, QC of mix production and construction, laboratory evaluation of reheated plant-produced mixes, and the early field performance. Also, statistical methods were used to make sense of the test results.

4.1 Mix Design

As previously stated, the mix design principle implemented in this research study is the BMD. Before implementing BMD, the control and experimental mixtures were designed by following VDOT's Volumetric Mix Design (VMD) criteria.

Starting with the control mix, the VMD yielded a total asphalt content of 5.20%. Then, APA and IDEAL – CT samples were prepared in compliance with the sample fabrication procedure prescribed by VDOT's BMD provisional standard specification to establish the performance properties of the control mix. The result of the APA and IDEAL – CT tests are summarized in Table 14 and 15, respectively. From Table 14, the average APA rut depth of the control mix is 2.72mm, which is less than the VDOT threshold value of 8mm after 8000 cycles. However, the CT_{index}, which is a parameter used in IDEAL – CT for evaluating cracking resistance in asphalt mix showed that the control mix failed to meet the minimum CT_{index} criterion of 70.

Table 14. APA Test Result of the Control Mix (VMD)

AC (%)	Rutting (mm)	Avg. Rut Depth (mm)	St. Dev.
5.2	2.46	2.72	0.36
5.2	2.80		
5.2	3.19		
5.2	2.42		

*St. Dev. = Standard Deviation

Table 15 .IDEAL -CT Result of the Control Mix (VMD)

Sample No.	AC (%)	CT_{Index}	Average CT_{Index}	St. Dev.
1	5.2	48.6		
2	5.2	58.1		
3	5.2	49.6		
4	5.2	33.6	44.9	8.76
5	5.2	46.1		
6	5.2	33.9		
7	5.2	44.4		

The volumetric analysis of the experimental mix produced a total asphalt content of 5.25%. Before fabricating the APA and IDEAL – CT samples, the rejuvenator dosage must be determined. To get the appropriate rejuvenator dosage, the APA and IDEAL – CT samples of the experimental mix were fabricated at a constant asphalt content of 5.25% but different dosages. The dosages, without disclosing the exact percentage, are low, medium, and high. The APA and IDEAL – CT results of the experimental mix are illustrated in Figure 24 and 25, respectively.

From Figure 24, the average rut depth of the experimental mix at low, medium, and high dosage of rejuvenator are numerically different, but none of the average rut depth neither came close nor exceeded the threshold rut depth value of 8mm. The highest average rut depth was recorded in the experimental mix at a medium dosage of rejuvenator, while the experimental mix containing a low dosage of rejuvenator had the lowest average rut depth.

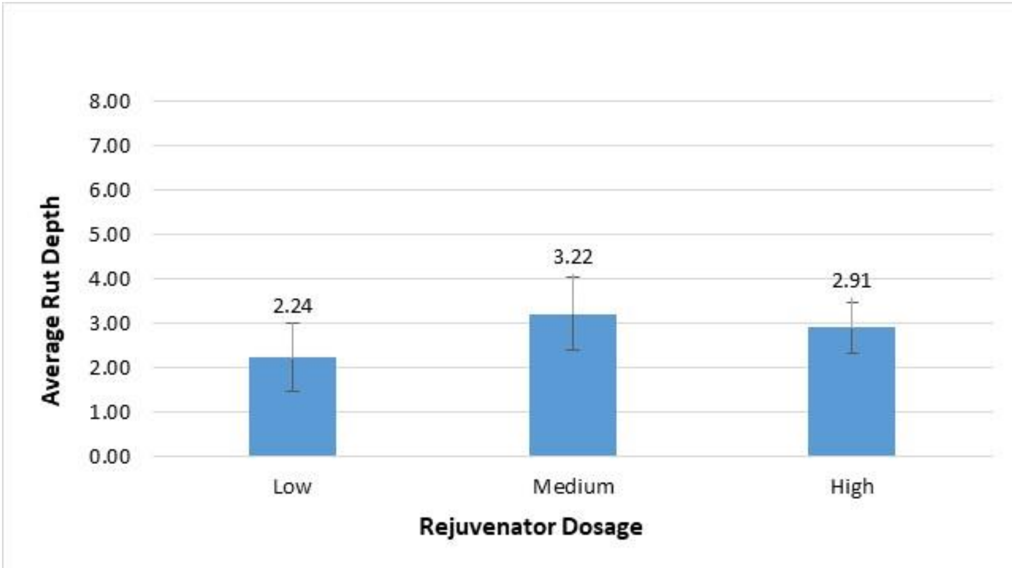

Figure 24 .APA Test Result of the Experiment Mix (VMD)

Figure 25 shows the IDEAL – CT results of the experimental mix at the three dosage levels. Numerically, the best cracking resistance, expressed with the CT_{index} values, was exhibited by the experimental mix containing a medium dosage of rejuvenator.

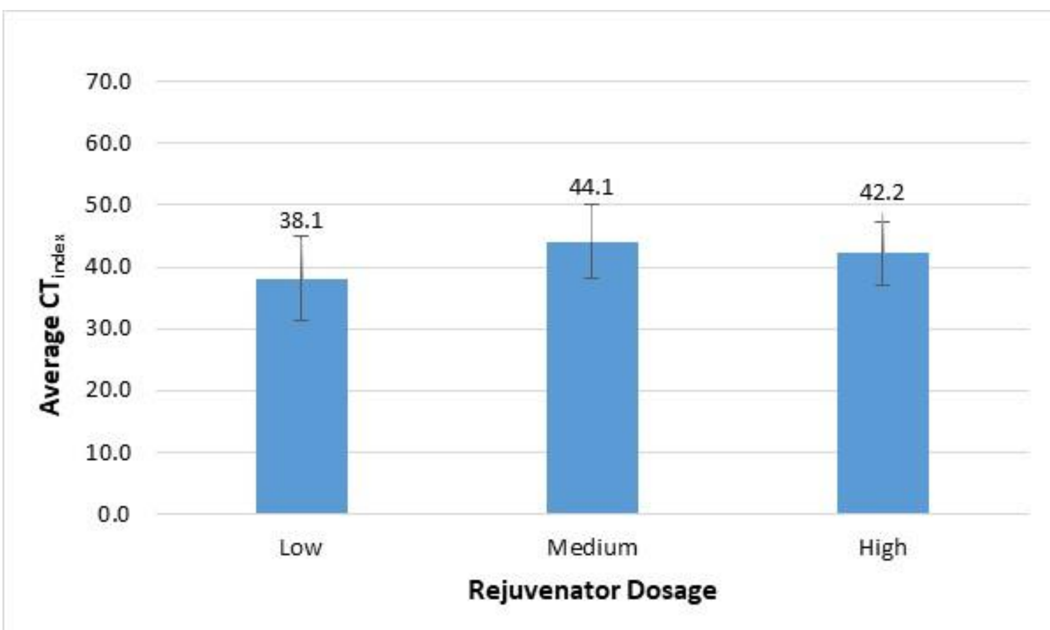

Figure 25. IDEAL – CT Result of the Experimental Mix (VMD)

Since the average rut depth of the experimental mix at a medium dosage of rejuvenator is satisfactory, and the cracking resistance is highest at the same dosage level, then a medium dosage of Anova asphalt rejuvenator was adopted. However, at the three different dosage levels, the experimental mix did not satisfy the minimum CT_{index} criterion of 70 specified in the VDOT's BMD provisional standard specification.

Table 16 summarizes the volumetric properties of the control and experimental mixes designed with VMD. The poor cracking performance of the control and experimental mixes designed with VMD led to the need for the implementation of BMD by using a performance-modified volumetric mix design approach. The BMD results of the control and experimental mixes are explained in the next section.

Table 16. Volumetric Properties of the Control and Experimental Asphalt Mixtures

Mix. Properties	**Control Mix**		**Experimental Mix**	
		Criteria		**Criteria**
NMAS, mm	9.5	--	9.5	--
Ndes	50	--	50	--
Total AC, %	5.19	--	5.25	--
Gmm	2.729	--	2.717	--
Gmb	2.620	--	2.608	--
Va	4.0	4.0	4.0	4.0
VMA	16.3	Min. 16.0	16.7	Min. 16.0
VFA	75.9	70 - 85	77.7	70 - 85
DP	1.07	0.7 - 1.3	1.20	0.7 - 1.3
Pbe, %	4.89	--	4.96	--
Pba, %	0.31	--	0.31	--
Anova 1501*, (% of Total A.C)	0.50	--	--	--
Anova 1815**, (% of Total A.C)	--	--	Medium	--
RAP AC, %	1.33	--	2.21	--

Virgin Binder, %	3.86	--	3.04	--

*Anova 1501 = Anti-Strip / Warm-Mix Additive; ** Anova 1815 = Recycling Agent / Rejuvenator

4.2 BMD

The failure of the VMD test samples to produce a control mix and an experimental mix with satisfactory cracking performance makes BMD an attractive alternative design procedure. The APA, IDEAL – CT, and Cantabro abrasion test results of the control and experimental mixes are explained in the subsequent sections. As previously established, the experimental mix was designed at a medium dosage of rejuvenator.

4.2.1 APA Test

The APA test samples were prepared and tested by following the VDOT's BMD provisional standard specification, as summarized in Table 10. The APA test results of both the control and experimental mixes at varying asphalt content are illustrated in Figure 26 and 27, respectively. From Figure 26, the linear regression line suggests that there is a corresponding increase in the rut depth of the control mix as the asphalt content increases. However, since the highest rut depth (5.64 mm) recorded at 6.2% asphalt content is below the 8.00mm threshold specified by VDOT (see Table 10), selecting an asphalt content within the range of 5.2% – 6.2% will satisfy the rutting criterion of the design control mix.

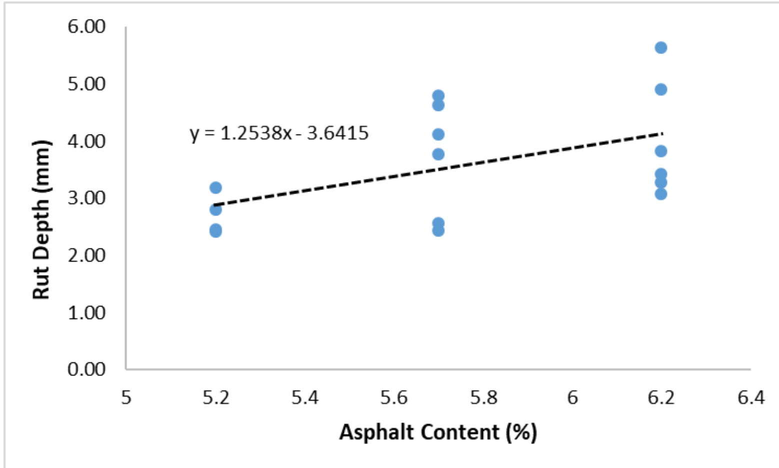

Figure 26. APA Test Result of the Control Mix (BMD)

The linear regression line of the APA test results of the experimental mix, as depicted in Figure 27, suggests that an increase in asphalt content yields a corresponding increase in rut depth, with some degree of variability. Also, the highest rut depth (4.87mm) recorded at 6.2% asphalt content when the asphalt content is varied between 5.2% and 6.2%, is below the threshold specified by VDOT. Therefore, selecting an asphalt content within the range of 5.2% – 6.2% will satisfy VDOT's BMD rutting criterion.

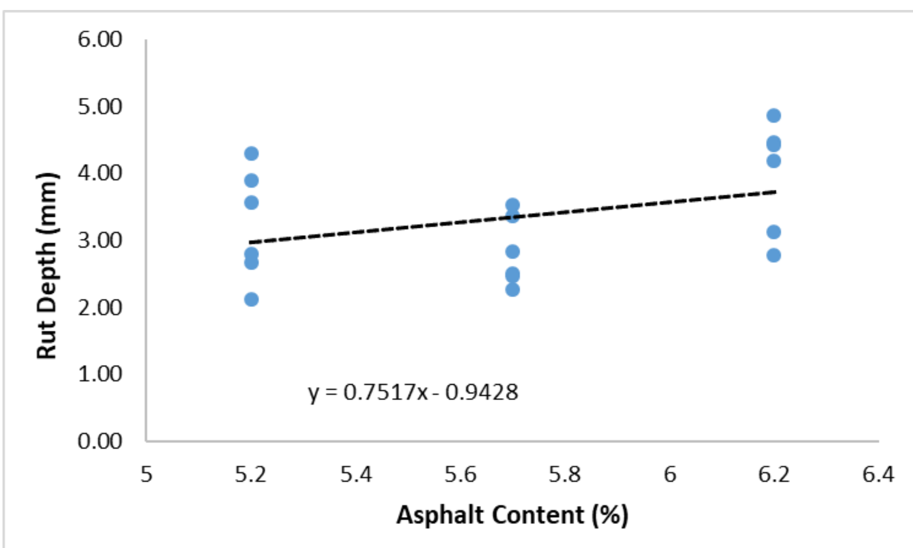

Figure 27. APA Test Result of the Experimental Mix (BMD)

4.2.2 IDEAL – CT

As shown in Figure 28, the IDEAL – CT result of the control mix, based on the average CT_{index}, shows that the cracking resistance of the mix improves as the asphalt content increases. However, since VDOT's BMD criterion for IDEAL – CT requires a minimum CT_{index} of 70 (i.e., $CT_{index} \geq 70$), then at 5.5% asphalt content ($CT_{index} = 74.6$), the control mix satisfies VDOT's cracking criterion. Further, at 5.5% asphalt content, VDOT's BMD criterion for APA rutting test is also satisfied (see Figure 26).

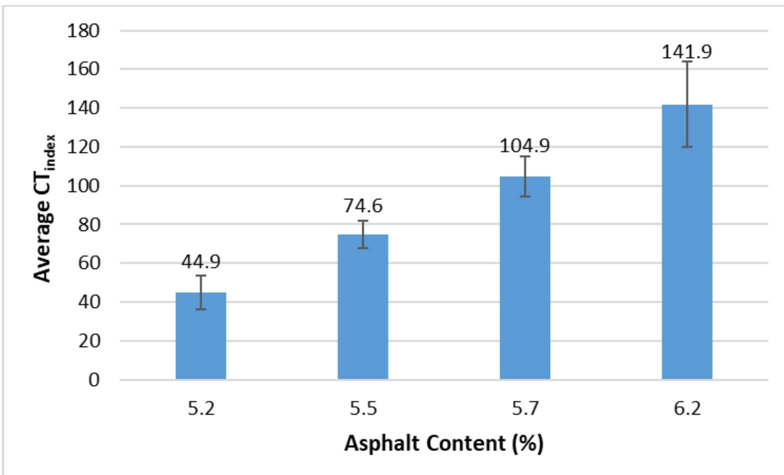

Figure 28. IDEAL – CT Result of the Control Mix (BMD)

Figure 29 illustrates the IDEAL – CT result of the experimental mix, which also exhibited a similar trend as compared to the control mix. The experimental mix was tested at different asphalt content at a range of 5.2% - 6.2%. At 5.8% asphalt content, the CT_{index} of the experimental mix is 75.6, which satisfies the minimum requirement by VDOT. Also, at 5.80% asphalt content, the rut depth of the experimental mix is below the 8mm rut depth threshold specified by VDOT.

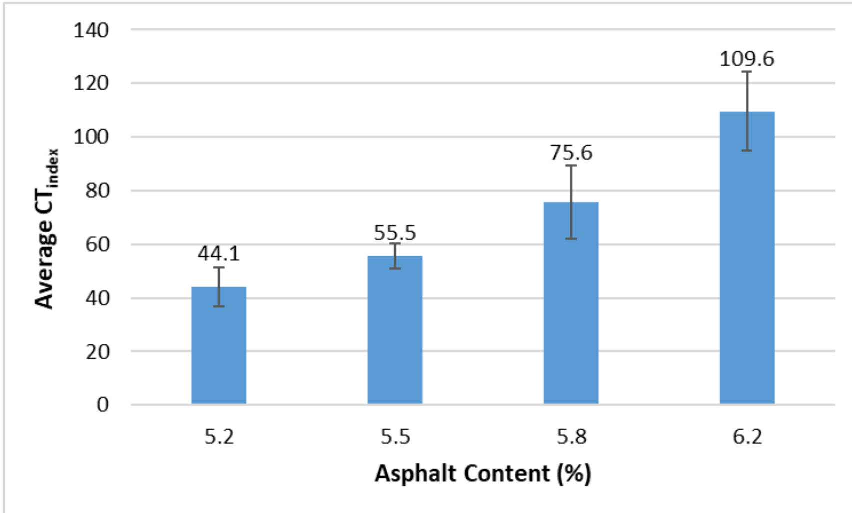

Figure 29. IDEAL – CT Result of the Experimental Mix (BMD)

Based on the APA and IDEAL – CT result of the control and experimental mixes designed using the BMD procedure, the selected asphalt content of the control and experimental mixes are 5.50% and 5.80%, respectively. Table 17 summarizes the volumetric properties of the control and experimental mixes designed using the BMD protocol. From Table 17, it can be observed that the design air voids of the control and experimental mixes did not meet the 4.0 Superpave VMD minimum requirement. Such an outcome is expected since the lowered design air voids of the control and experimental mixtures facilitate the increase in the asphalt content of the mixtures. Implementation of the performance-modified volumetric mix design approach of BMD also affects other volumetric factors such as VMA, VFA, G_{mm}, etc.

Table 17. Vol. Properties of the Control and Experimental Mix with BMD

Mix. Properties	**Control Mix**		**Experimental Mix**	
		Criteria		**Criteria**
NMAS, mm	9.5	--	9.5	--
Ndes	50	--	50	--
Total AC, %	5.50	--	5.80	--
Gmm	2.715	--	2.691	--

Gmb	2.636	--	2.628	--
Va	2.9	4.0	2.4	4.0
VMA	15.8	Min. 16.0	16.2	Min. 16.0
VFA	82.4	70 - 85	85.8	70 - 85
DP	1.0	0.7 - 1.3	1.1	0.7 - 1.3
Pbe, %	5.21	--	5.512	--
Pba, %	0.31	--	0.308	--
Anova 1501*, (% of Total A.C)	0.50	--	--	--
Anova 1815**, (% of Total A.C)	--	--	Medium	--
RAP AC, %	1.33	--	2.21	--
Virgin Binder, %	4.17	--	3.59	--

Before adopting an asphalt content of 5.50% for the control mix and 5.80% for the experimental mix, VDOT's BMD provisional standard specification requires the simulation of mass loss in SGC specimens by using the Cantabro abrasion test procedure, which is summarized in Table 10.

Figure 30 shows that the mass loss in the control mix reduces as the asphalt content increases. A similar trend is also observed in the air void versus asphalt content relationship. The selected asphalt content of 5.5% with a corresponding average mass loss of 4.7% satisfies the VDOT's Cantabro test criterion, which specifies a maximum mass loss of 7.5% (i.e., mass loss ≤ 7.5%).

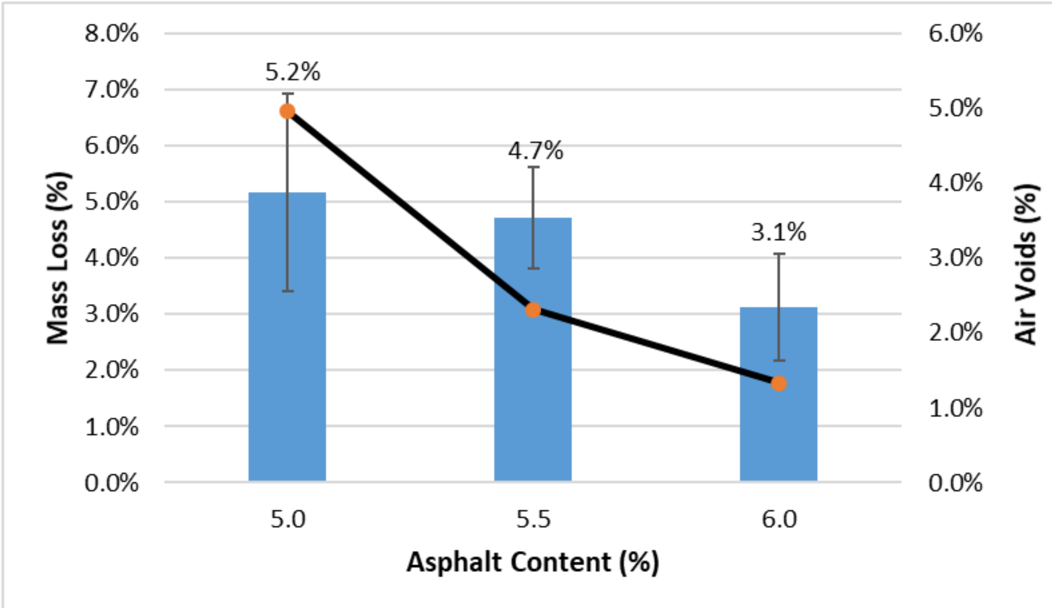

Figure 30. Cantabro Test Result of the Control Mix (BMD)

Figure 31 shows the Cantabro test result of the experimental mix. From Figure 31, it was observed that the mass loss in the SGC specimens decreases as the asphalt content increases. Since the lowest asphalt content (4.7%) considered in the test satisfies the VDOT's Cantabro test criterion, it can be deduced that the adopted asphalt content (5.8%) of the experimental mix satisfies VDOT's Cantabro test criterion.

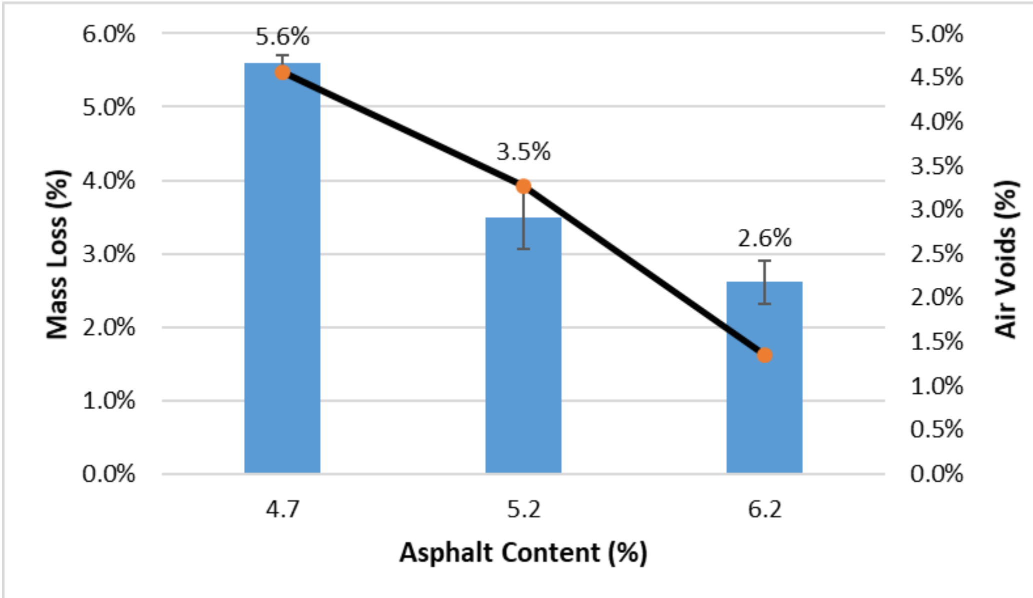

Figure 31. Design Experimental Mix Cantabro Test Result

Other asphalt mixtures performance test such as IFIT, OT, TSR, etc. were conducted to further verify the selected asphalt content of the control and experimental mixes. However, only the three basic performance tests (APA, IDEAL – CT, and Cantabro) required by VDOT were reported and discussed in this section of the report. In the next section, the QC results of the plant-produced control and experimental mixtures were presented and discussed.

4.3 QC Results

Before delving into the QC results of the control and experimental mixes, it is necessary to reiterate that the control and experimental mixes were laid in Section N3A and N3B of the NCAT Test Track, respectively. Table 18 shows the QC results of the aggregate gradation of the control mix and how it compares with the target JMF, while Table 19 shows the comparison between the QC and target JMF results of the volumetric properties of the control mix. Figure 32 portrays a comparison of the aggregate gradation of the QC and target JMF results of the control mix.

Table 18. QC Result of the Control Mix (Aggregate Gradation)

Sieve Size	% Passing	
	Target JMF	QC
25 mm (1")	100	100
19 mm (3/4")	100	100
12.5 mm (1/2")	100	100
9.5 mm (3/8")	97	95
4.75 mm (#4)	61	56
2.36 mm (#8)	38	36
1.18 mm (#16)	27	26
0.60 mm (#30)	20	18
0.30 mm (#50)	14	11
0.15 mm (#100)	9	7
0.075 mm (#200)	5.5	4.9

A close observation of the QC results of the volumetric properties of the control mix suggests that there are some slight deviations in the QC results from the target JMF value. The QC results of the binder content and air void are good examples of such deviations. However, the use of QC results to determine the payment schedule or rating of the construction quality is beyond the scope of this report.

Table 19. QC Result of the Control Mix (Volumetric Properties)

Volumetric Properties	Target JMF	QC
Binder Content (Pb)	5.5	6.0
Eff. Binder Content (Pbe)	5.2	5.7
Dust-to-Eff. Binder Ratio	1.1	0.9
RAP Binder Replacement (%)	24	25
RAS Binder Replacement (%)	0	0
Total Binder Replacement (%)	24	25
Rice Gravity (Gmm)	2.715	2.679
Bulk Gravity (Gmb)	2.636	2.608
Air Voids (Va)	2.9	2.7
Aggregate Gravity (Gsb or Gse)	2.973	2.966
VMA	16.2	17.3
VFA	82	85

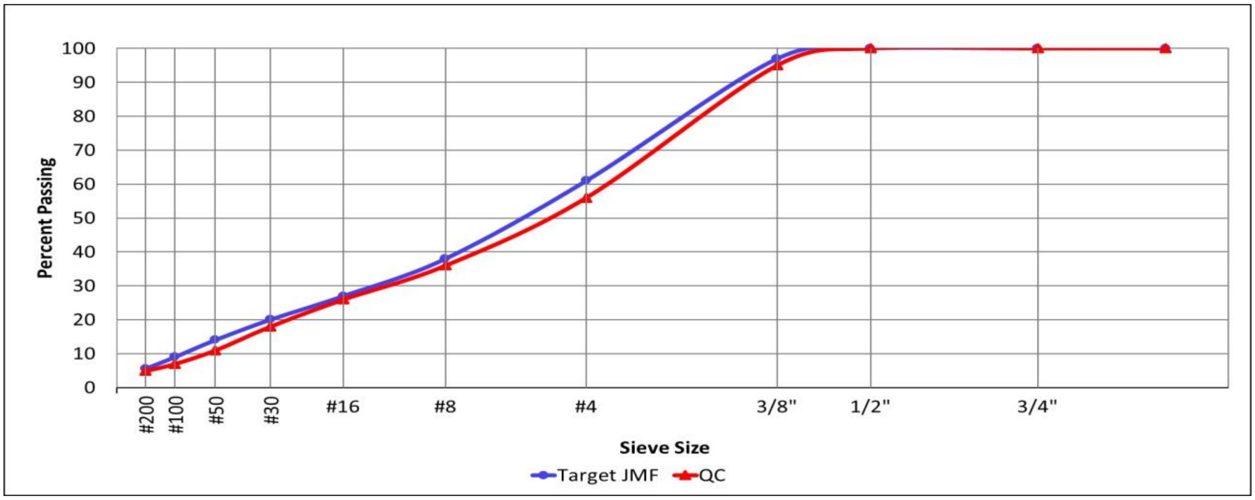

Figure 32. Graph of QC Results of the Control Mix (Aggregate Gradation)

The QC results of the aggregate gradation of the experimental mix and how it compares with the target JMF is summarized in Table 20 and illustrated in Figure 33. Table 21 presents the comparison between the QC and target JMF results of the volumetric properties of the experimental mix.

Table 20. QC Results of the Experimental Mix (Aggregate Gradation)

	% Passing	
Sieve Size	**Target JMF**	**QC**
25 mm (1")	100	100
19 mm (3/4")	100	100
12.5 mm (1/2")	100	100
9.5 mm (3/8")	97	96
4.75 mm (#4)	61	56
2.36 mm (#8)	38	38
1.18 mm (#16)	28	27
0.60 mm (#30)	21	20
0.30 mm (#50)	15	13
0.15 mm (#100)	10	9
0.075 mm (#200)	6.3	6.1

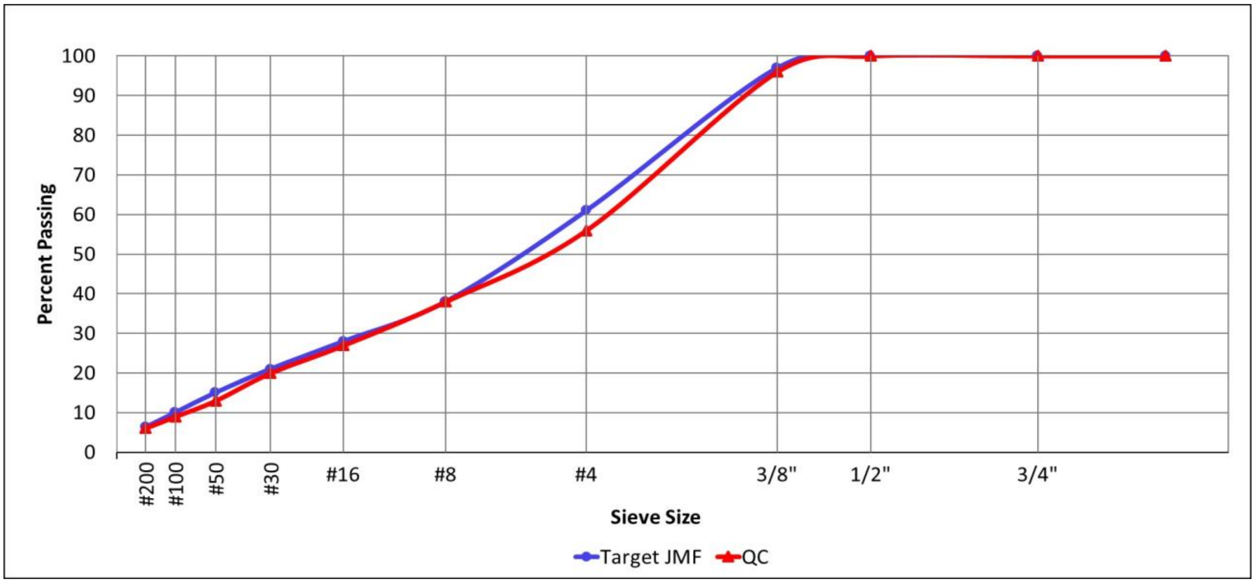

Figure 33. Graph of QC Results of the Experimental Mix (Aggregate Gradation)

Table 21. QC Results of the Experimental Mix (Volumetric Properties)

Volumetrics	Target JMF	QC
Binder Content (Pb)	5.8	6.0
Eff. Binder Content (Pbe)	5.5	5.7
Dust-to-Eff. Binder Ratio	1.1	1.1
RAP Binder Replacement (%)	38	38
RAS Binder Replacement (%)	0	0
Total Binder Replacement (%)	38	38
Rice Gravity (Gmm)	2.691	2.664
Bulk Gravity (Gmb)	2.628	2.624
Air Voids (Va)	2.3	1.5
Aggregate Gravity (Gsb or Gse)	2.963	2.949
VMA	16.5	16.3
VFA	86	91

4.4 Performance Test Results

In this section, the performance results of the extracted asphalt binders and the reheated plant-produced mixes are presented and discussed.

4.4.1 Asphalt Binder

Asphalt binders were extracted from the three reheated plant-produced mixes, which includes: (1) control mix with 30% RAP and no rejuvenator (N3A-RH); (2) first experimental mix with 45% RAP and Anova rejuvenator (N3B-RH); and (3) Second experimental mix with 45% RAP and no Anova rejuvenator (N3C-RH). The results of the binder tests are summarized in Table 22. As previously stated in the methodology of the research study, a two-level aging procedure was implemented in the binder tests. The first aging level is the same as the standard aging level described in AASHTO M320 standard specification. The second aging level is an extended aging procedure where the recovered asphalt binder was subjected to twice the standard PAV aging time of 20 hours (2 x PAV aging).

The High-Temperature Performance Grade (HTPG) is the lower of the original DSR and RTFO DSR. The Low-Temperature Performance Grade (LTPG) is the higher of the S-BBR and m-BBR. The HTPG and LTPG selections are based on the recommendations in the NCHRP 452 report. ΔT_C is the difference between the S-BBR and m-BBR, where a high ΔT_C suggests a low resistance of the asphalt mixtures to non-load related cracking. Using the ΔT_C value in Table 22, it can be deduced that at both aging levels, the first experimental mix is more resistant to non-load related cracking than the control mix.

Table 22 .Extracted Asphalt Binder Test Result

Aging Level	Mix ID	HTPG (°C)	S-BBR (°C)	m-BBR (°C)	ΔTc (°C)
HTPG: As Extracted LTPG: As Extracted + TFO	N3A - RH	76.7	-23.8	-14.6	-9.2
	N3B - RH	75.9	-24.0	-22.0	-2.0
	N3C - RH	78.8	-21.5	-18.6	-2.9
HTPG: As Extracted + TFO + PAV LTPG: As Extracted + TFO + PAV	N3A - RH	91.2	-20.7	-12.8	-7.9
	N3B - RH	90.5	-22.8	-17.0	-5.9
	N3C - RH	94.3	-18.6	-12.9	-5.7

Table 23 shows the performance grading of the extracted asphalt binder at both the standard and extended aging level. At the standard asphalt binder aging level, the LTPG of both experimental mixtures is higher than the LTPG of the control mixture, which suggests that the experimental mixtures have a superior low temperature cracking performance. However, at the extended asphalt binder aging level, the LTPG of all the extracted binders are the same.

Table 23. Performance Grading of Extracted Asphalt Binder

Aging Level	Mix ID	Extracted Binder PG
HTPG: As Extracted LTPG: As Extracted + TFO	N3A - RH	76 - 22
	N3B - RH	70 - 28
	N3C - RH	76 - 28
HTPG: As Extracted + TFO + PAV LTPG: As Extracted + TFO + PAV	N3A - RH	88 - 22
	N3B - RH	88 - 22
	N3C - RH	94 - 22

4.4.2 Mixture Resistance to Moisture Damage

The moisture susceptibility of the three reheated plant – produced mixes were examined using the AASHTO T283 procedure, as explained in the previous chapter. The outcome of the moisture susceptibility test is illustrated in Figure 34. From Figure 34, it can be observed that all the three mixes satisfied the 0.80 minimum TSR value requirement for moisture susceptibility, with the control mix having the highest moisture damage resistance.

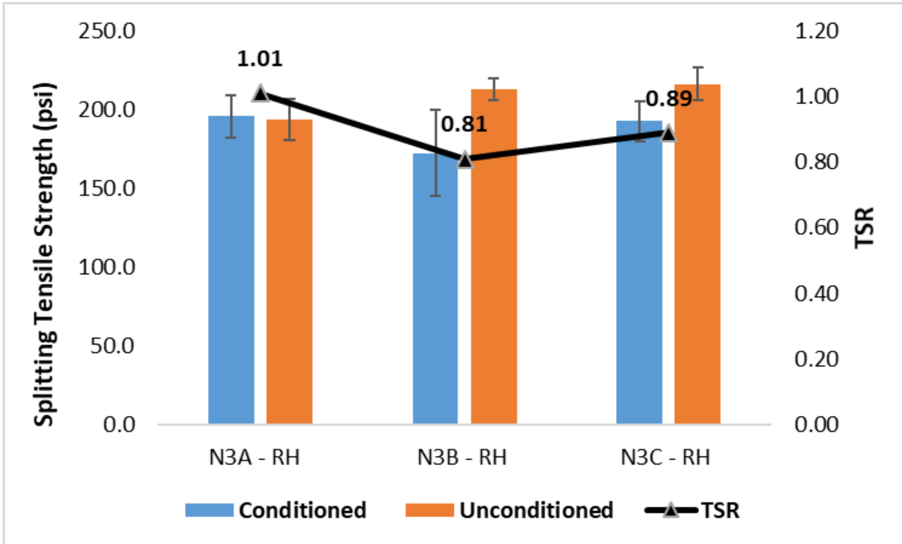

Figure 34. TSR Test Results

4.4.3 APA Rutting Result

The APA rutting test procedure, as previously described in chapter 3 of this report, was conducted on SGC specimens of N3A-RH and N3B-RH and the result of the experiment is summarized in Table 24. The rut depth was measured using both the manual and automatic methods. Numerically, regardless of manual or automated measurement, N3B-RH recorded a higher rut depth. Also, the Coefficient of Variation (COV) of N3B-RH was higher in both the manual and automated average rut depth measurements.

The APA rutting result was further analyzed by deploying descriptive statistics. Since the comparison is between two test results, a Student t-test or a one-way Analysis of Variance (ANOVA) test will suffice. It is worthy of note that the Student t-test, which is the analysis of two levels of a nominal variable, is just a special example of a one-way ANOVA test. So, executing the analysis of the means of the APA rut depth results with a Student t-test will produce the same result as a one-way ANOVA test.

The one-way ANOVA (α = 0.05) test results of both the manual and automated average rut depths of N3A-RH and N3B-RH are summarized in Table 24. The null hypothesis of the one–way ANOVA (α = 0.05) test states that the average rut depth of N3A-RH and N3B-RH are equal. Since the P-values of both the manual and automated average rut depths result in each exceeding the significance level of the test (α = 0.05), then there is not enough evidence to reject the null hypothesis of the tests.

Table 24. APA Rutting Test Results

Parameter	Mix Identifier	Average	COV	P-Value
Manual rut depth (mm)	N3A-RH	2.97	0.22	0.267
	N3B-RH	3.44	0.69	
Automated rut depth (mm)	N3A-RH	2.97	0.23	0.382
	N3B-RH	3.37	0.84	

4.4.4 HWTT Result

To further examine the rutting resistance of the SGC specimens of N3A-RH and N3B-RH, HWTT was done, and the result is summarized in Table 25. From the one-way ANOVA (α = 0.05) test result, it can be concluded that there is not enough evidence to reject the null hypothesis that states that the rut depth of N3A-RH and N3B-RH, at either 10,000 or 20,000 passes, are equivalent. The HWTT result obtained is in tandem with the APA result.

Table 25. HWTT Result with Tukey – Kramer Statistical Groupings

Parameter	Mix Identifier	Average	COV	P-Value
10,000 Passes	N3A-RH	2.51	0.07	0.760
	N3B-RH	2.55	0.02	
20,000 Passes	N3A-RH	3.15	0.11	0.875
	N3B-RH	3.10	0.03	

4.4.5 I-FIT Result

Table 26 summarizes the I-FIT result of SGC specimens of N3A-RH, N3B-RH, and N3C-RH. Numerically, N3B–RH recorded the highest average flexibility index (FI), which suggests that the mix has the highest cracking resistance. To examine the overall differences in the average FI of the three mixes, a one-way ANOVA ($\alpha = 0.05$) statistical test was conducted on the I-FIT results. Before conducting the one-way ANOVA test, an outlier analysis, as specified in ASTM E178 (Standard Practice for Dealing with Outlying Observations), was conducted on the raw I-FIT data. The outlier test was conducted at 5% significance level. After eliminating the outlying observation(s), the one-way ANOVA test was ran on the cleaned-up data. The p-value (0.007) of the statistical test suggested that significant differences exist among the average FI of the three mixes. However, the p-value does not specifically tell where the significant difference lies. Thus, a post hoc test is required to tell where the differences occurred between the groups.

The post hoc test used is the Tukey-Kramer test because it accounts for unequal sample sizes. The result of the post hoc test is represented with statistical grouping (letters) without any order of arrangement. If any of the groups share the same letter, then the average flexibility indexes are not significantly different. However, if the letters are different, then a significant difference exists.

The Tukey-Kramer statistical grouping of the I-FIT result, as presented in Table 26, suggested that significant differences exist between the average FI of N3A-RH and N3C-RH as they do not share a letter. However, N3B-RH shares a letter with both N3A-RH and N3C-RH. The high cracking resistance observed in N3B-RH may be attributed to the effect of the Anova asphalt rejuvenator.

Table 26. I-FIT Result with Tukey – Kramer Statistical Groupings

Parameter	Mix Identifier	Average FI	COV	P-Value	Grouping
FI	N3A-RH	6.6	0.27	0.007	A
	N3B-RH	7.5	0.35		A, B
	N3C-RH	3.7	0.33		B

Figure 35 illustrates the I-FIT results of the three reheated mixes. The error bar represents the inherent variation in the FI results of each mix, the numbers on the error bars are the average FI of each mix, and the letters represent the Tukey-Kramer statistical groupings.

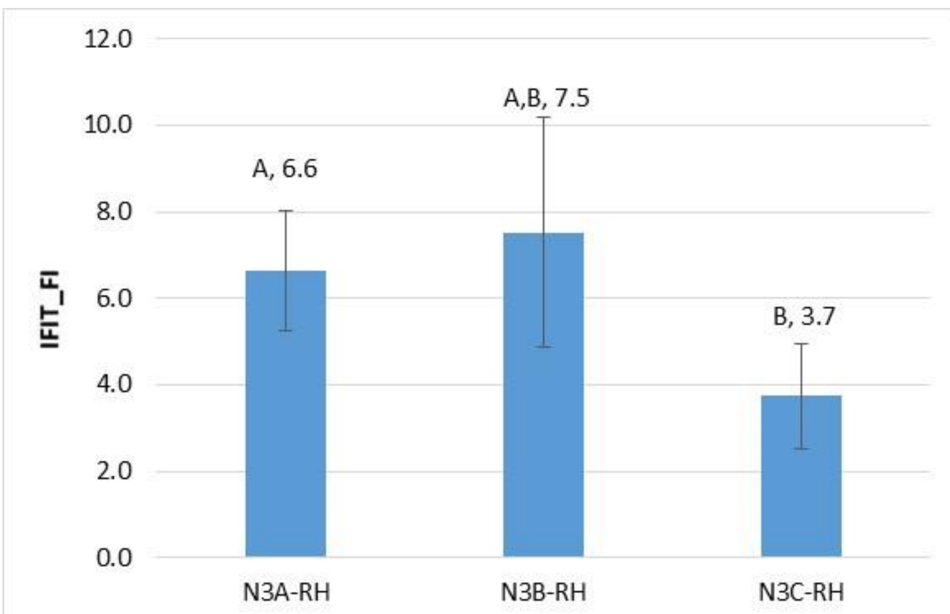

Figure 35. I-FIT Result

4.4.6 Overlay Test (OT) Result

The resistance of each reheated plant – produced asphalt mixtures to reflective cracking was evaluated using the OT procedure, as described in the previous chapter. The outcome of the test is summarized in Table 27 and illustrated in Figure 36. The average number of cycles to failure (N_f), which is an index that represents the reflective resistance of an asphalt mixture to cracking, is 297, 326, and 73 for N3A–RH, N3B–RH, and N3C–RH, respectively. The higher the N_f, the better the resistance of the asphalt mixture to reflective cracking. Going by the N_f of the three mixes, N3B–RH produced the highest resistance to reflective cracking.

An outlier test (ASTM E178) was conducted on the raw OT data before running a one-way ANOVA test. The result of a one-way ANOVA ($\alpha = 0.05$) statistical test, based on the P-value (<0.05), suggested that there is a significant overall difference in the N_f of the three mixes. However, the Tukey-Kramer statistical groupings of the OT results showed that the only significant difference in the OT results of the three mixes is the N_f of N3C–RH.

Table 27. OT Result with Tukey – Kramer Statistical Groupings

Parameter	Mix Identifier	Average N_f	COV	P-Value	Grouping
N_F	N3A-RH	297	0.24	0.000	A
	N3B-RH	326	0.09		A
	N3C-RH	73	0.32		B

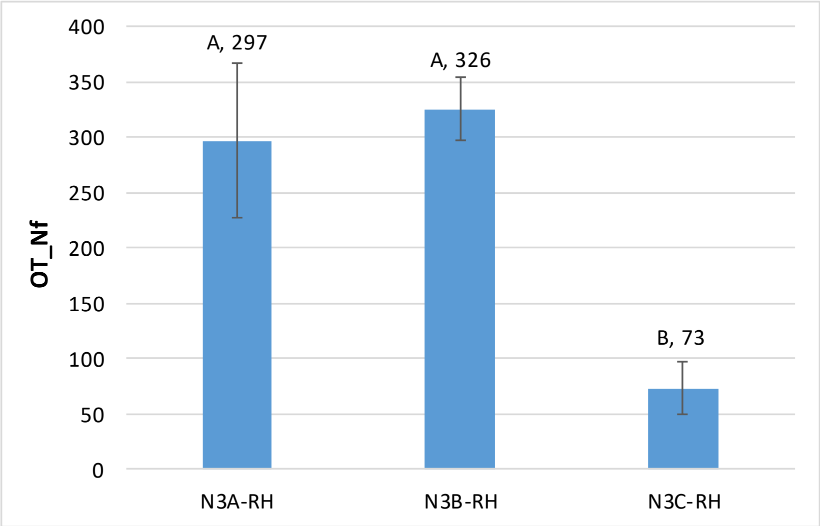

Figure 36. OT Result

4.4.7 DCT Test Result

The DCT test was conducted to evaluate the cracking resistance of each reheated plant – produced asphalt mixtures at low temperature. The result of the test is summarized in Table 28 and illustrated in Figure 37. Numerically, N3B – RH had the highest average fracture energy (FE), followed by N3A – RH, and then N3C – RH. A one-way ANOVA ($\alpha = 0.05$) statistical test, after running the outlier test, suggested that there is not enough evidence to reject the null hypothesis that states that the average FE of N3A-RH, N3B-RH, and N3C-RH are equal. It is worthy of note that the post hoc test is only required if the p-value of the one-way ANOVA test falls below the significance level of the test.

Table 28. DCT Test Result with Tukey – Kramer Statistical Groupings

Parameter	Mix Identifier	Average FE	COV	P-Value
FE	N3A-RH	529	0.11	0.136
	N3B-RH	562	0.09	
	N3C-RH	494	0.12	

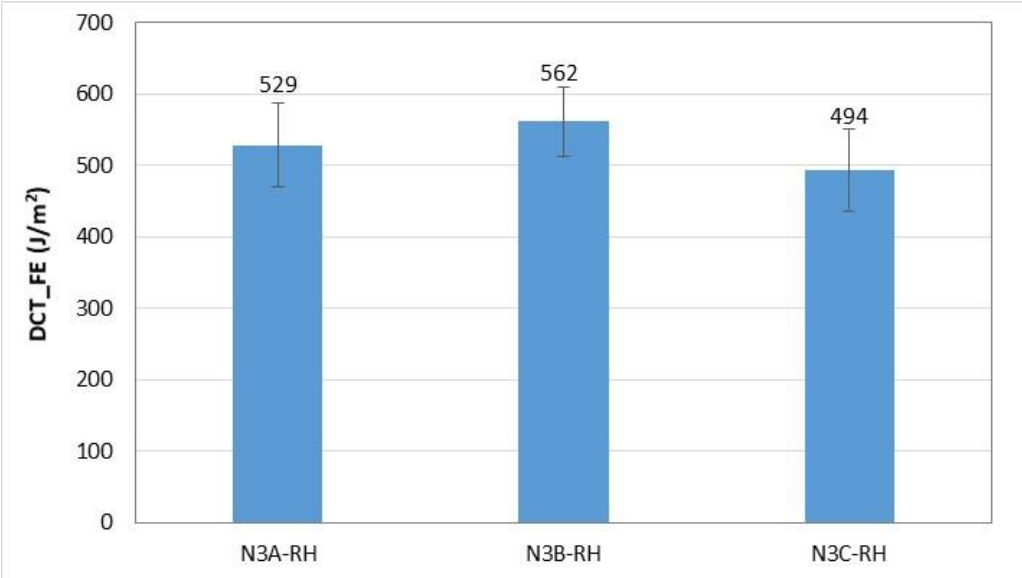

Figure 37. DCT Test Result

4.4.8 IDEAL – CT Result

Table 29 shows the result of the IDEAL – CT of the three mixes. The cracking resistance of the mixes are measured based on the CT_{index}, and VDOT requires a minimum CT_{index} of 70 (i.e., $CT_{index} \geq 70$). From Table 29, only N3A–RH satisfied VDOT's BMD IDEAL – CT criterion. The p-value (<0.05) of a one-way ANOVA ($\alpha = 0.05$) statistical test suggested that there is a significant overall difference in the CT_{index} of the three mixes. Further, the Tukey-Kramer statistical groupings of the IDEAL – CT results showed that significant differences exist between the CT_{index} of the three mixes. The outlier test was conducted on the raw CT_{index} data of the three mixes.

Based on the IDEAL – CT result of the reheated plant – produced mixes summarized in Table 29 and illustrated in Figure 36, it can be deduced that the result is not in tandem with the previous cracking test results.

Table 29. IDEAL – CT Result with Tukey – Kramer Statistical Groupings

Parameter	Mix Identifier	Average	COV	P-Value	Grouping
Nf	N3A-RH	102	0.15	0.000	A
	N3B-RH	64	0.18		B
	N3C-RH	45	0.20		C

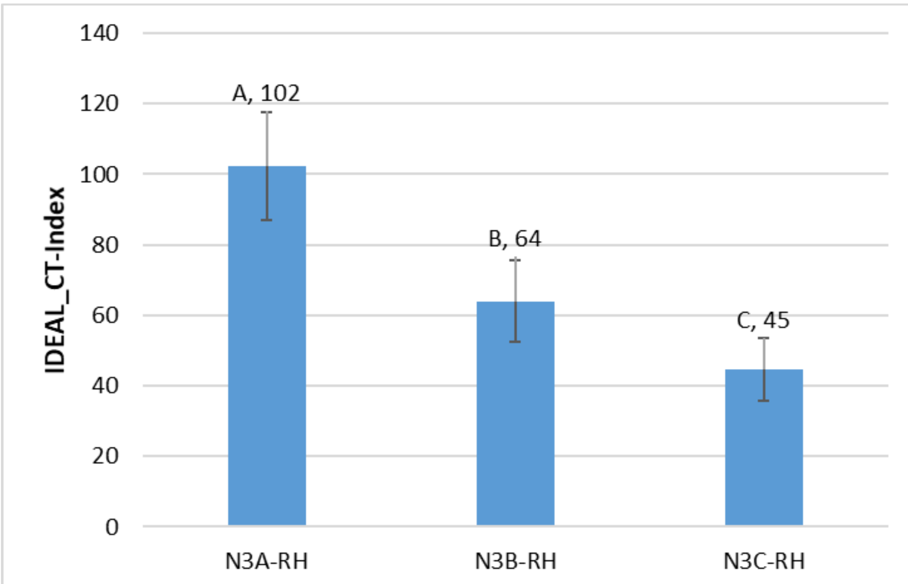

Figure 38. IDEAL – CT Result

4.4.9 Cantabro Abrasion Test Result

The Cantabro abrasion test was conducted on SGC specimens of the three reheated plant – produced asphalt mixtures, as previously described, and the outcome of the test is illustrated in Figure 39. The three mixes failed VDOT's BMD Cantabro test criterion (mass loss ≤ 7.5%). However, the rejuvenated mix (N3B-RH) recorded the lowest average mass loss.

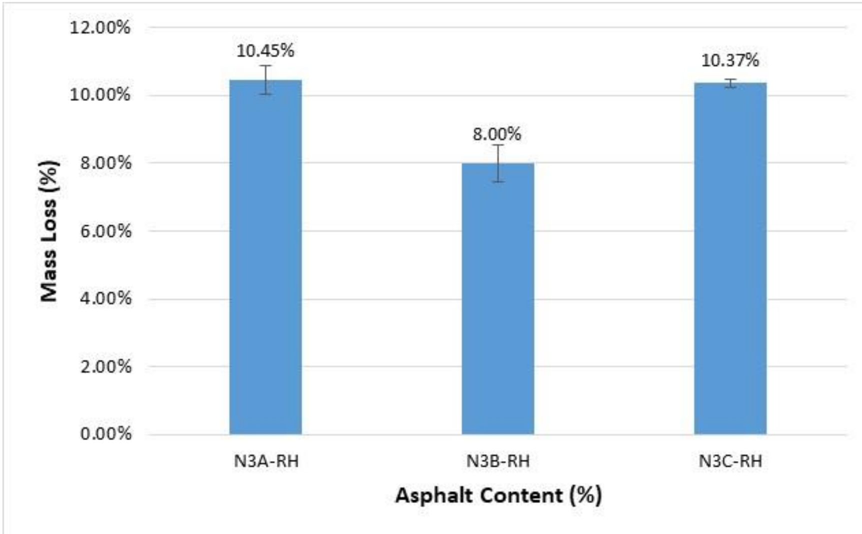

Figure 39. Cantabro Abrasion Test Result

4.5 Early Field Performance

The early field performance data collected from Sections N3A and N3B of the NCAT Test Track include rutting, IRI, and cracking. At the time of data collection (April 2019), approximately 2 million ESALs of traffic have been applied on the Test Track. An estimated total of 10 million ESALs of traffic is expected to have been applied on the Test Track by the end of the seventh research cycle.

4.5.1 Rutting

The rutting data of N3A and N3B, from the end of construction in September 2018 to April 2019, is illustrated in Figure 40. From Figure 40, it can be observed that the rut depths of both N3A and N3B at the end of construction or the beginning of traffic application are relatively high. The high rut depths can be attributed to the initial densification under traffic loads and the prevailing weather condition. The weather condition is a significant factor because traffic operation commenced in late summer when the surface temperature of the pavement is generally high. Overall, the rut depths of both N3A and N3B are within the low severity rutting classification, according to

ALDOT–392 standard specifications. VDOT only uses IRI, a measure of ride quality, to rate their pavement condition.

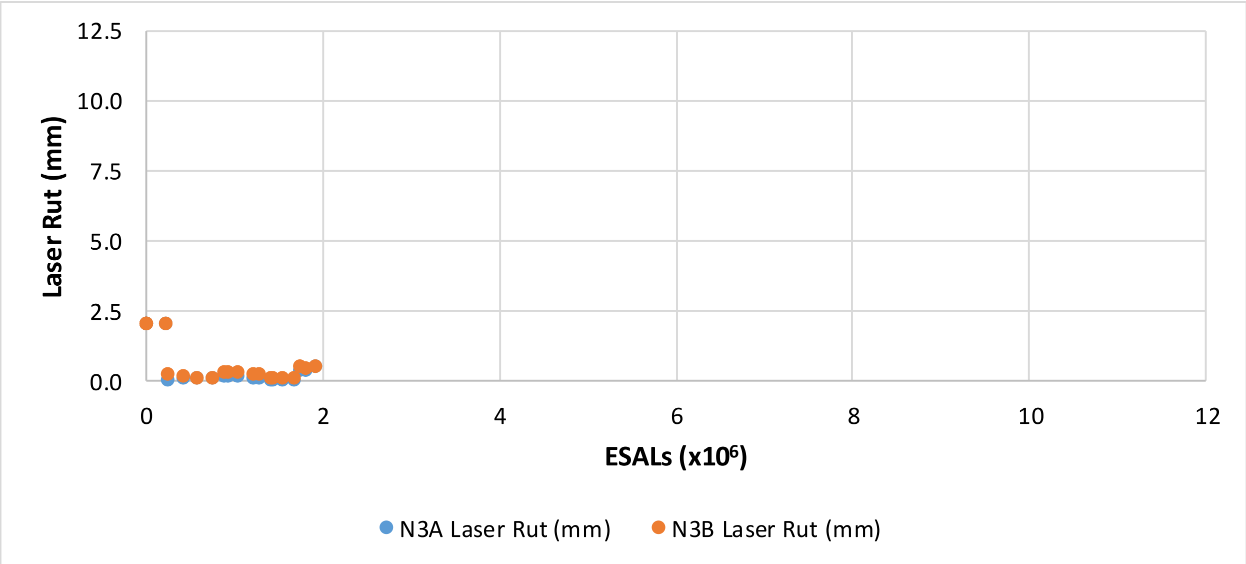

Figure 40. Field Rut Depth Measurements

4.5.2 Ride Quality

Ride quality, expressed as IRI, was measured for both N3A and N3B using the Dynatest inertial profiler, and the result is shown in Figure 41. According to VDOT's IRI classification (see Table 30), IRI of N3A and N3B indicates "good" ride quality. However, the ride quality of N3A and N3B may change over the lifespan of the current research cycle.

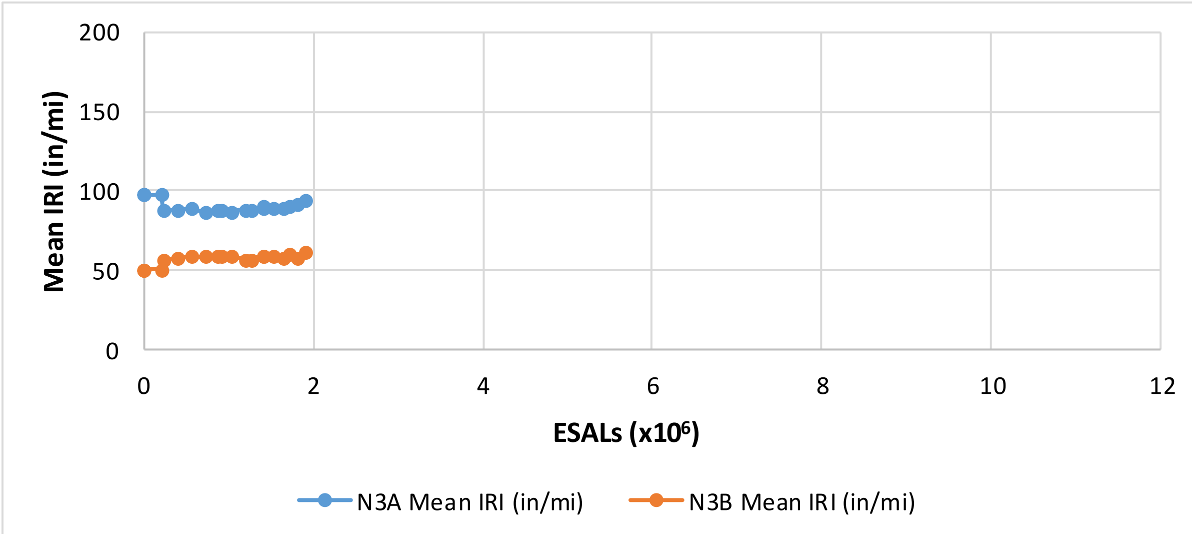

Figure 41. Ride Quality Measurements

Table 30. Pavement Ride Quality Based on IRI (VDOT, 2018)

Ride Quality	IRI Rating (inch/mile)	
	Interstate & Primary Roads	Secondary Roads
Excellent	< 60	< 95
Good	60 - 99	96 - 169
Fair	100 - 139	170 - 219
Poor	140 -199	220 -279
Very Poor	≥ 200	≥ 280

4.5.3 Cracking

Field data collected from both Sections N3A and N3B in April 2019 showed that no visible crack is present in either Section of the NCAT Test Track, as illustrated in Figure 42. However, cracks may develop, subsequently over the life span of the current research cycle.

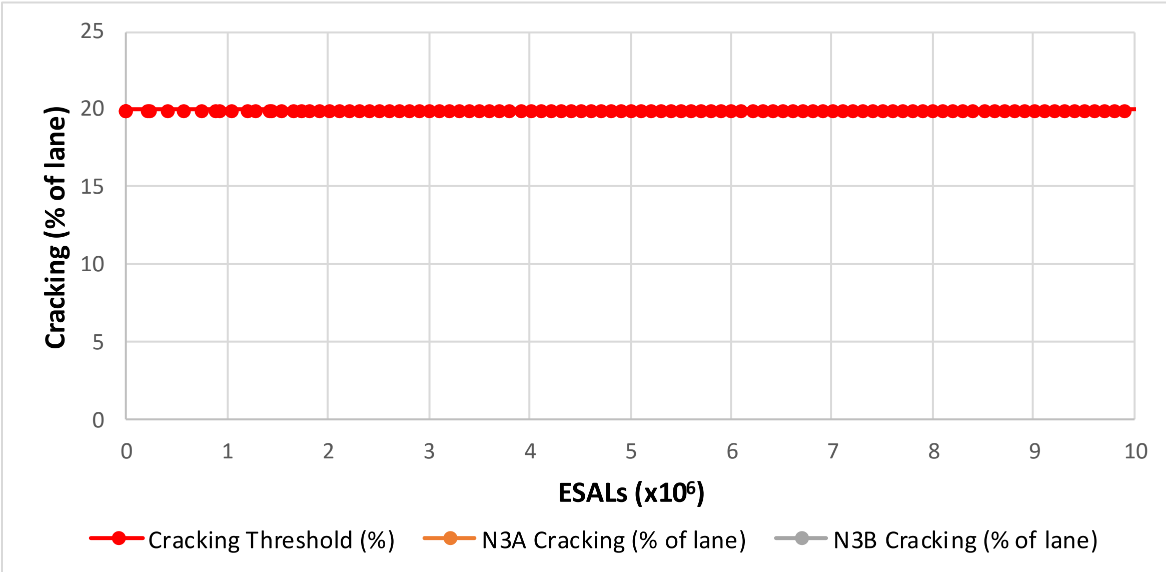

Figure 42. Field Cracking Measurements

4.6 Summary of Results

Testing was conducted by following the experimental plan of this research study (See Figure 6). The result of the Superpave VMD was reported before progressing to the outcome of the BMD. Table 31 summarizes the results of both the mix design approach.

Table 31. Summarized Performance Test Results of BMD and VMD

	CTindex	APA
Control VMD	45.0	2.72
Experimental VMD	44.0	3.22
Control BMD	74.6	3.28
Experimental BMD	75.6	3.42

Figure 43 depicts the relationship between the cracking and rutting potential of the control and experimental asphalt mixtures when either VMD or BMD is implemented. By implementing

BMD, appropriate asphalt contents were selected for the control and experimental asphalt mixtures, which satisfy VDOT's rutting, cracking, and mass loss criteria.

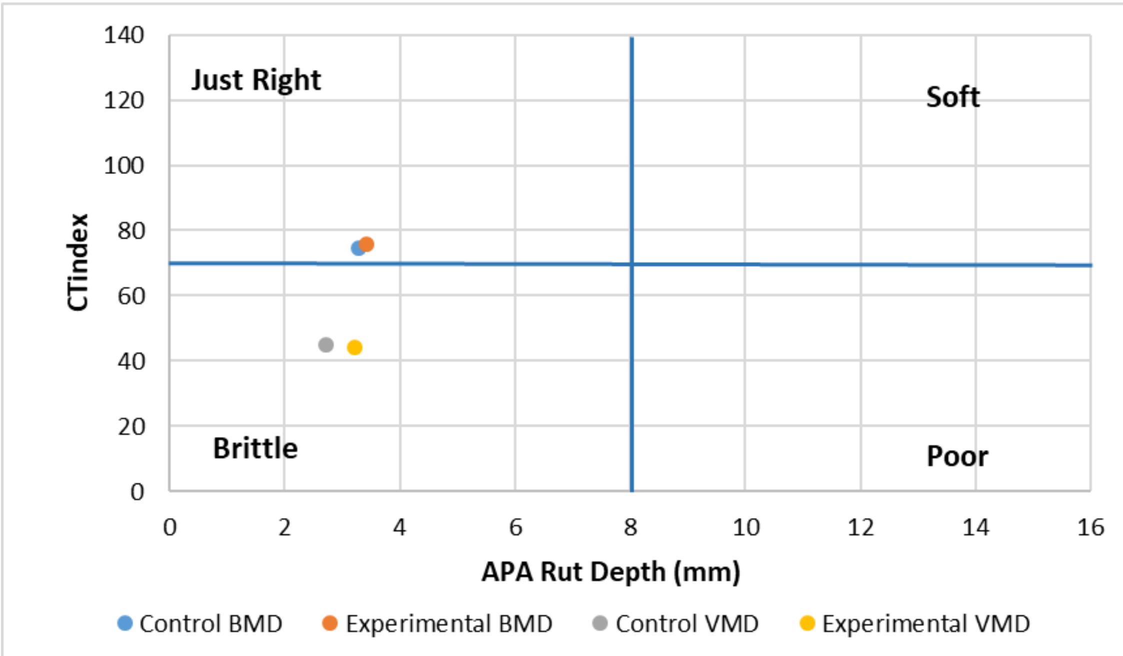

Figure 43. VMD and BMD of the Control and Experimental Asphalt Mixtures

As part of the mixture design, Cantabro abrasion test was conducted on the control and experimental mixes. The control and experimental mixes satisfied the mass loss criterion (mass loss $\leq$ 7.5%) at the design asphalt contents. After selecting the appropriate asphalt contents for the control and experimental mixes, QC test result of the plant – produced mixes laid in Sections N3A and N3B of the NCAT Test Track were presented.

The reheated plant – produced mixes were subjected to a series of performance tests (see Table 13), and the results of the tests are summarized in Table 32. The letter next to each test results in Table 32 represents the Tukey-Kramer statistical groupings of each performance test conducted. The asphalt binder PG of each mix includes the PG at both the standard and extended aging level, respectively. The APA test result mix includes both the manual and automatic rut depths, respectively. Also, the HWTT result of each reheated asphalt mixture comprises of the rut depth at both 10,000 and 20,000 passes, respectively.

Table 32. Summarized Lab. Performance Test Results of Reheated Mixtures

Asphalt Mixture	Laboratory Tests								
	Binder PG	**TSR**	**APA (mm)**	**HWTT (mm)**	**I-FIT (10^{-1})**	**OT**	**DCT**	**IDEAL - CT**	**Cantabro (%)**
N3A-RH	76 – 22	1.01	2.97	2.51	62 (A, B)	297 (A)	529	102 (A)	10.4
	88 – 22		3.44	3.15					
N3B-RH	70 – 28	0.81	2.97	2.55	85 (A)	326 (A)	562	64 (B)	8.0
	88 – 22		3.37	3.10					
N3C-RH	76 – 28	0.89	N/A	N/A	37 (B)	73 (B)	494	45 (C)	10.4
	94 – 22		N/A	N/A					

Figure 44 represents the results of all the cracking tests considered in this research study. All the cracking test results exhibited a similar trend except the IDEAL – CT result. Further investigation may be required to understand the reason behind the deviation of the IDEAL – CT results from the observed trends in OT, DCT, and IFIT results.

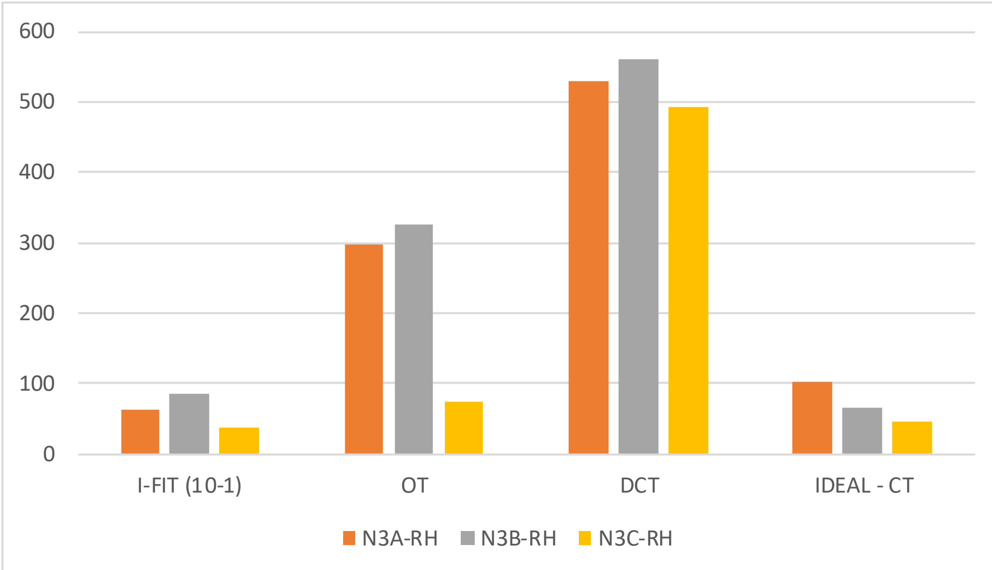

Figure 44. Lab. Cracking Performance of the Reheated Asphalt Mixtures

The field investigation data of IRI, Rutting, and crack mapping were analyzed, and the results are summarized in Table 33. No visible cracking was observed in N3A and N3B, which is why the percentage of cracked area is 0% in both cases. The average rut depth of N3A and N3B are the same, which agrees with the APA and HWTT results of the reheated plant – produced mixes. Last, the mean IRI of N3A and N3B indicates "good" ride quality by VDOT standard (Table 23).

Table 33. Summary of Early Field Performance Data Analysis

Performance Parameters	**N3A**	**N3B**
Mean IRI (in/mi)	94.7	61.4
Laser Rut (mm)	0.6	0.6
Mean Texture Depth (mm)	0.38	0.37
Cracking (%)	0.0	0.0

5 CHAPTER 5 – CONCLUSION AND RECOMMENDATIONS

This research study involves the use of BMD approach to evaluate the performance of a high-RAP asphalt mixture, which contains a specially formulated bio-based asphalt rejuvenator (Anova) by Cargill, in relative to a control asphalt mixture with lower RAP content and no Anova asphalt rejuvenator. The control asphalt mixture contains 30% RAP, PG 64-22 asphalt binder, and no Anova asphalt rejuvenator (N3A-RH). The experimental asphalt mixture contains 45% RAP, PG 64-22 asphalt binder, and Anova asphalt rejuvenator (N3B-RH). For a more robust laboratory performance comparison, a similar experimental asphalt mixture with 45% RAP, PG 64-22 asphalt binder, and no Anova asphalt rejuvenator was also evaluated (N3C-RH). The laboratory performance tests were conducted by reheating the plant – produced asphalt mixtures before compaction and testing. The control asphalt mixture and the experimental asphalt mixture that contains Anova asphalt rejuvenator were produced at the asphalt plant and laid on both Sections N3A and N3B, respectively, of the NCAT Test Track for field assessment.

Based on the laboratory performance results of the control and experimental asphalt mixtures, and the early field performance of N3A and N3B, the following inferences can be drawn:

- Moisture susceptibility test result of the three mixes suggested that the use of Anova asphalt rejuvenator does not compromise the moisture damage resistance of the RAP mix.
- The rutting performance of N3B-RH and N3A-RH are almost the same.
- I-FIT, OT, and DCT cracking test results produced a similar trend, where N3B-RH performed better than N3A-RH and N3C-RH.

- IDEAL-CT result suggested that N3A-RH has the best cracking resistance, contrary to the result of the other three cracking tests. Also, only N3A-RH satisfied VDOT's BMD cracking criterion ($CT_{index} \geq 70$).
- Cantabro abrasion test result of N3A-RH, N3B-RH, and N3C-RH did not meet VDOT's BMD criterion (Mass loss ≤ 7.5%). However, N3B-RH test result yielded the lowest average mass loss (8%).
- Early field performance of the control and experimental asphalt mixtures suggested that there is neither visible cracking nor significant rutting in Section N3A and N3B of the NCAT Test Track. Also, mean IRI value of N3A and N3B suggests "good" ride quality by VDOT's standard.

Based on the investigation carried out so far, the following recommendations will be useful for future implementation of Anova asphalt rejuvenator in high-RAP asphalt mixtures:

- The reason behind the unique result of the IDEAL – CT needs to be further researched on.
- The performance of the three mixes, after undergoing an approved aging protocol, should be evaluated.
- More performance tests may be required to evaluate and compare the performance of the control and experimental mixes before high-RAP mixes containing bio-based rejuvenator could be adopted by some State DOT.

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