RAP Reset — Responsibly Optimizing Recycled Materials Use in Asphalt Concrete and Pavement Performance Life

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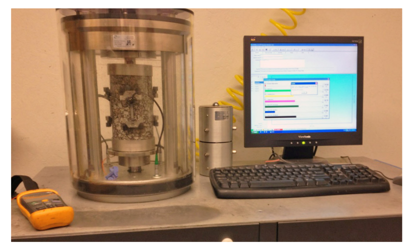

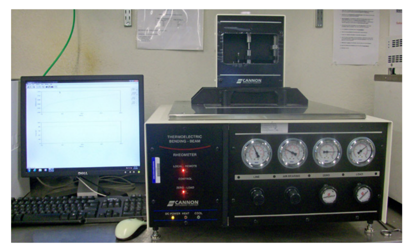

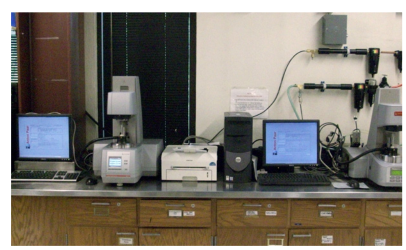

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RAP RESET – Responsibly Optimizing Recycled Materials Use in Asphalt Concrete and Pavement Performance Life

FINAL REPORT

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16. Abstract

The durability performance of Asphalt Concrete (AC) with various percentages of Reclaimed Asphalt Material (RAM) has been a focal point of the asphalt industry leading it to embrace the Balanced Mix Design (BMD) concept. The goal of this research was to review and enhance Washington State Department of Transportation (WSDOT) AC materials selection, mix design, and standard specifications for optimized use of RAM, based on readily implementable technology, in collaboration with industry stakeholders for improved durability performance. The scope included a literature review; an assessment of RAM supply in the state; a statewide comparison of low and high RAM pavement performance data to determine if differences were observed; evaluation of raw materials and field mixtures; evaluation of laboratory mixed-laboratory compacted (LMLC), field mixed-laboratory compacted (FMLC), and field cored (FMFC) samples; statistical analysis of results and preparing this report. The laboratory analysis included short-term and long-term aging of binders and mixtures with rheological and cracking tests performed on them. Primary recommendations of this study include: integrating volumetric parameters along with further performance testing in a BMD approach to increase effective binder content; implementing all of the volumetric criteria in AASHTO M323 during mix design, test section and acceptance; using ΔTc as an aging parameter in binder specifications; including RAM in all mix designs regardless of doses; maintaining an IDT strength specification and transitioning to CT-Index; adding long-term aging for IDT/CT-Index test specimens in the future, maintaining the current Hamburg Wheel Track (HWT) rutting test and criteria; short- and long-term standard specification revisions; and re-evaluating the performance of RAM pavements with time.

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EXECUTIVE SUMMARY

Durability of Asphalt Concrete (AC) containing reclaimed asphalt materials (RAM), mainly Reclaimed Asphalt Pavement (RAP) and Reclaimed Asphalt Singles (RAS), has become a focal point of the asphalt industry. RAM has been used in asphalt mixtures for 50 years due to its sustainable benefits. The Washington Department of Transportation (WSDOT) has successfully been using RAP since the mid 1970's and introduced a contractor's option to use 0 to 20 % RAP (low RAP/no RAS) in dense graded mixtures without the need to adjust binder properties or limit RAP to meet project specific Performance Grade (PG) requirements. The WSDOT specification revisions in 2013 allowed up to 40% binder replacement from RAM (high RAP/any RAS) while ensuring that blended binders and mixture volumetric specifications can be met. The goal of this research was to review and enhance WSDOT AC materials selection, mix design, and standard specifications for optimized use of RAM, based on recent readily implementable technologies, in collaboration with industry stakeholders for improved durability performance.

A literature review indicated that good long-term performance of AC mixtures containing up to 30% RAP (by total weight of mix) was observed in several states. It also indicated that as RAM dose increases more diligence is needed to obtain desired performance. Several practices identified to be used by DOTs for improving RAP AC performance, were using softer virgin binders or recycling agents, including ΔTc specification, reducing mix design air voids, increasing Voids in Mineral Aggregates (VMA), and implementing a Balanced Mix Design (BMD) approach.

For this study, laboratory samples were prepared based on WSDOT approved mix designs using virgin and recycled materials obtained from multiple sources in the state. Several variations of the mix designs were made so the impact of virgin binder source, virgin binder grade, RAP dose (0 to 40%), and recycling agent on durability performance could be evaluated. Superpave performance grading (PG) was performed on virgin and RAP binder with ΔTc determination. The results showed that 40-hour Pressure Aging Vessel (PAV) aging identified these evaluated binders as m-controlled (more aging susceptible) binders. Comparison of ΔTc estimated from blending charts and ΔTc measured on physically blended binders (100% contribution from RAP) showed that blending charts underestimated ΔTc (in absolute value) for 90% of the evaluated binder blends. Laboratory Mixed-Laboratory Compacted (LMLC) samples, were subjected to Indirect Tensile (IDT) strength and Dynamic Modulus (E*) testing for performance evaluation. As RAP dose increased, IDT strengths increased while Cracking Tolerance Index (CT-Index) decreased. The IDT strength values were better correlated to the fracture energy than CT-Index and exhibited the lowest variability among the cracking test parameters.

Field Mixed-Laboratory Compacted (FMLC) samples and Field Mixed-Field Compacted (FMFC) samples (cores) were evaluated for mixture performance and extracted binder rheological properties. The extracted binder properties of the FMLC and FMFC samples demonstrated that 20-hour PAV aged the extracted binder and dropped the ΔTc more than the 5 days at 185°F aging protocol on compacted specimens. Despite high variability among field samples, lower variability was associated with IDT strength values compared to CT-Index and fracture energy results. The IDT strength parameter showed statistically significant differences between FMLC and LMLC samples, IDT strengths consistently being greater for field mixtures (FMLC).

A statewide analysis of the WSDOT Pavement Management System (PMS) data indicated that the performance of the low and high RAP pavements aged between 3 and 8 years is statistically similar, though it is noted that significant differences may be observed with long-term in-service performance. The performance analysis of the field cores from 7 projects indicated the same, although significantly more cracking was observed on one high RAP project than the other low RAP and high RAP projects. It is also noted that relatively young high RAM pavements (3 to 8 years of service) used virgin binder grade dropping and/or recycling agent.

Primary recommendations of this study include: integrating volumetric parameters along with further performance testing in a BMD approach to increase effective binder content; implementing all of the volumetric criteria in AASHTO M323 during mix design, test section and acceptance; using ΔTc as an aging parameter in binder specifications; including RAM in all mix designs regardless of doses; maintaining an IDT strength specification and transitioning to CT-Index; adding long-term aging for IDT/CT-Index test specimens in the future, maintaining the current Hamburg Wheel Track (HWT) rutting test and criteria; short- and long-term standard specification revisions; and re-evaluating the performance of RAM pavements with time.

1 INTRODUCTION

1.1 Background

Performance of Asphalt Concrete (AC) mixture with various percentages of recycled asphalt materials became a focal point of the asphalt industry including State Department of Transportation (DOTs), Federal Highway Administration (FHWA), and National Asphalt Pavement Association (NAPA), again in the mid-2010's. The use of recycled materials in asphalt mixtures has been occurring for 50 years because of the sustainable benefits (economic, environmental, and societal) associated with it (NAPA IS-138). Recycled materials used in Washington AC pavements could include Reclaimed Asphalt Pavement (RAP), Reclaimed Asphalt Shingles (RAS) and Recycled Engine Oil Bottoms (REOB). In the late 2000's recycled materials use increased as the economy crashed, market competition increased/margins decreased, and at the same time virgin asphalt binder cost increased significantly. Collectively, impacts of this combination are beginning to be better understood and indicate that the durability of AC could be compromised in this process. Especially in cases where relatively low doses of RAP (around 20%) are used without adjusting virgin binder grade, RAS is used, high doses of RAP are used without appropriate virgin binder selection and/or when these are coupled with asphalt binders susceptible to rapid aging. This has led to DOTs making asphalt binder and mixture design specification changes including implementation of Balanced Mix Design (BMD) approaches. The Washington Department of Transportation (WSDOT) elected to review its recycled materials strategy based on national best practices, while considering Washington specific pavement performance, specifications, design, and test methods to ensure durable asphalt pavement mixtures are being used. WSDOT has been using recycled materials in asphalt pavements since the 1970's and has been monitoring their performance over time. Peters et al. investigated the laboratory and field pavement performance of RAP mixtures constructed by WSDOT in the 1970's and 1980's (Peters et al. 1986). The engineering properties of cores taken 1, 2, 4, and 6 years after construction were evaluated. How asphalt binder percent, core density, viscosity, penetration, and pavement condition varied over time was tracked. Despite promising results from cores, some regions in the state experienced performance and durability related issues associated with the RAP mixtures. Therefore, WSDOT included the standard specification for use of RAP in the 1988 edition of the *WSDOT Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT 1988). Since then, WSDOT standard specifications have allowed inclusion of up to 20% RAP in AC mixtures without any virgin binder grade changes or additional testing, relative to virgin asphalt mixtures (WSDOT 2020). WSDOT standard specification revisions in 2013 allowed for up to 40% binder replacement from RAM (high RAP/any RAS). This was prompted by national trends and industry desire to increase RAP use. Howell et al. recently indicated that the use of high RAM mixtures was still considered to be in its early stages (Howell et al. 2019).

However, Ashtiani et al. reported that RAP stockpile inventory increased by 190% from 2006 to 2017 in Washington State (Ashtiani et al. 2019). Based on these findings, there is continued interest in considering using high RAP doses in AC mixtures, to reduce the growth of RAP stockpile inventory. It is believed that there is a lack in the long-term performance data, particularly in cracking resistance of relatively high RAP AC pavements. Howell et al. analyzed WSDOT Pavement Management System (PMS) data from 2007 to 2017 in order to address the concerns regarding WSDOT pavement performance (Howell et al. 2019). The authors concluded that high RAP mixtures with comparable density values, performed similar to low RAP mixtures, in terms of pavement structural (i.e., cracking index) and rutting performance. Nevertheless, it is worth mentioning that the study included five years of high-RAP field and performance data, hence trends that take longer to develop may emerge later in the pavement service life.

1.2 Scope of Work

The goal of this research is to improve the performance of AC pavements containing recycled materials in Washington State. The primary objective is to enhance WSDOT AC materials selection, mix design, and standard specifications to responsibly optimize the use of recycled materials, based on recent readily implementable technologies, in collaboration with industry stakeholders for improved pavement performance. Accordingly, the following strategies were considered:

Literature Review: A review of the literature relative to the overall project objectives was performed and the key highlights were summarized in section 2.4 including historical RAM mixture performance, durability of recycled mixtures, implementation of the BMD approach and application of the ΔTc parameter.

WSDOT & Industry Recycling Practices, Goals and Collaboration: The Washington Asphalt Pavement Association (WAPA) worked with WSDOT on increasing the allowable amount of RAP and RAS in AC, up to 40% binder replacement, along with procedures to do so. This effort reviewed the changes related to similar alterations in other states for improvement, considering Washington specific conditions and how suggested changes could impact WSDOT and WAPA members. It also considered the proposed change to allow 25% RAP while dropping one Performance Grade (PG) without evaluating the binder properties. WSDOT and WAPA were surveyed, and staff were interviewed to determine perceptions on specifications, economics, and recycled mixture performance, which were compared to actual observations.

Performance of WSDOT AC containing Recycled Materials: Cracking, raveling, and/or stripping are all evidence of poor AC durability. The influence of low and high levels of recycled materials in WSDOT AC on pavement performance trends was interpreted. FMFC, FMLC, and LMLC samples and virgin materials were subjected to materials testing to evaluate their durability and identify if changes to current WSDOT specifications and/or design procedures could lead to improved durability.

Virgin Binder Aging Susceptibility: The effectiveness of American Association of State Highway and Transportation Officials (AASHTO) M320 *Standard Specification for PG Asphalt Binder* (AASHTO M320 2017), and AASHTO M332 *Standard Specification for Performance Graded Asphalt Binder Using Multiple Stress Creep Recovery (MSCR) Test* (AASHTO M332 2019) in identifying excessive aging susceptibility of asphalt binders that may result in cracking susceptible AC mixtures has been questioned. Therefore, the rheological parameter delta Tc "ΔTc" and its acceptance criteria of $\geq$-5°C on virgin binder was assessed for multiple binder sources in Washington after 20-hour and 40-hour Pressure Aging Vessel (PAV) conditioning.

RAP and RAS contribution to Binder Aging Susceptibility with Climate Impacts: The aged asphalt binders in RAP/RAS from Western versus Eastern Washington are considerably different, while the aged asphalt binder in RAS is significantly stiffer than in RAP. WSDOT Standard Specifications Section 5-04.2 (1), partially addresses this factor for high binder replacement mixtures (WSDOT 2020). The adoption of ΔTc criteria for virgin binder/recycled material blends, regardless of RAP and/or RAS content, to minimize WSDOT AC performance risk at the mix design stage was assessed for multiple sources in Washington.

Influence of Recycled Materials in Mix Design: There is national evidence that even relatively low percentages of RAP and/or RAS in AC mixtures can lead to significant reductions in performance life across different climatic zones. WSDOT standard specifications currently allow performing low RAP/no RAS mix designs without any change in specified virgin binder, or without including the RAP stockpile in the mix design (WSDOT 2020). The incorporation of RAP and/or RAS in all mix designs coupled with performance testing during the mix design process using the Hamburg Wheel Track test (HWTT) and Indirect Tensile (IDT) Strength test methods was evaluated in this study. The IDT strength testing included several parameters such as the tensile strength, Tensile Strength Ratio (TSR), fracture energy, and Cracking Tolerance Index (CT-Index).

2 REVIEW OF THE LITERATURE

2.1 Performance of AC mixtures with RAM

The literature review includes many examples of efforts made to improve the performance and long-term durability of AC mixtures with RAM. A Florida DOT project investigated the engineering and in-service performance properties of high RAP/RAS mixtures and concluded that field performance of 30% RAP mixtures was similar compared to virgin mixtures, however, the performance of mixtures containing over 30% RAP decreased remarkably (Nash et al. 2012). The properties of AC mixtures with different RAP percentages and 5% RAS were investigated by Williams et al. and the findings suggested that mixtures with PG58-22 virgin binder containing up to 35% RAP and 5% RAS provided adequate cracking resistance, while mixtures containing 40% RAP were overly susceptible to cracking failure (Williams et al. 2011). In 2011, FHWA reported that the performance and life of pavement containing up to 30% RAP was similar to virgin pavements based on a survey of Long-Term Pavement Performance (LTPP) sections containing at least 30% RAP located throughout the United States and Canada (Copeland 2011). The long-term performance of virgin and high RAP (35%) AC mixtures in LTPP sections in Texas, indicated that high RAP mixtures showed similar performance to virgin mixtures even after 17 years of service life (Hong et al. 2010). The authors indicated that despite the fact that properly designed RAP mixtures can perform very well, AC pavements containing RAP failed to perform as expected in many other cases across North America. Evidence elsewhere suggests that AC mixtures with RAM exhibit accelerated cracking when improperly designed. Poor performance of RAP mixtures was suggested to be driven by several factors such as traffic, climatic conditions, existing pavement conditions for overlays, pavement layer thickness and so on. Some of the recent approaches to improve RAP/RAS mixture performance include limiting the RAP/RAS content, using soft and modified asphalt binders, reducing design air voids or lowering N_{design}, as well as incorporating rejuvenators in mix design process to soften RAP/RAS binder (Zhou et al. 2013).

An evaluation of Texas mixtures concluded that rutting/moisture resistance were enhanced as RAP content increased within the mixtures at low levels (Zhou et al. 2011). However, for mixtures with 30% RAP or more, and those with RAP/RAS combinations, the relative cracking resistance reduced when increasing the recycled materials content. Cracking resistance and moisture/rutting resistance of the mixtures were not significantly affected when 10-15% of RAP were used even without lowering the virgin binder grade (Zhou et al. 2011).

The use of softer and modified asphalt binders has been shown to improve RAP/RAS mixture performance without jeopardizing relative resistance to rutting/moisture damage in laboratory testing (Zhou et al. 2013). The research team recommended implementing a balanced mix design system and performance evaluation system for project-specific service conditions while claiming that soft and modified binders can effectively improve cracking resistance. Several documents summarized the current best practices for using recycled materials such as RAP/RAS in AC mixtures, which include RAP/RAS processing, mix design, mix production, and construction process (Zhou et al. 2014, West 2015). The authors developed a six-step guideline for RAP/RAS processing and stockpile management. A brief stepwise summary of the RAP processing guideline is shown below:

- Reducing RAP variability by eliminating contamination.
- Separating RAP stockpiles obtained from different sources.
- Obtaining uniform RAP by blending or mixing the material properly before processing RAP stockpiles.
- Properly crushing or fractionating RAP stockpiles while avoiding excessive crushing.
- Using paved, sloped surface to store processed RAP and storing it under covering.
- Marking RAP stockpiles for better characterization of stockpiles.

2.2 Asphalt Mixture Durability

Durability of AC mixtures refers to the ability of the compacted AC to resist the deterioration by maintaining its structural integrity throughout its expected service life (Schaffer et al. 2008). The primary distresses related with AC mixtures durability issues are raveling and cracking. Mixture durability can be addressed at the mix design level by providing binder specifications, aggregate specifications, limiting the volumetric properties, and ensuring the mixture is not sensitive to the moisture damage. Whereas during construction, durability is addressed by in-place compaction specifications, which may help to reduce the permeability and hence mitigates the age hardening rate in the mixture.

Table 1 is a summary of multiple factors that may negatively impact AC mixture durability (Bonaquist 2014). Although, the review of this literature is focused on the effect of recycled materials on AC mixture performance, other influencing factors in the following table are listed just to provide a broad coverage of the topic.

Table 1: Summary of the Factors that Affects the Durability of AC Mixtures (Bonaquist 2014)

General Category	Specific Factors Influencing Durability
Environment	Temperature Moisture
Drainage	Surface drainage Subsurface drainage
Construction	Weather conditions Segregation Compaction Joints Bonding of the layers
Mixture composition	Aggregate properties Binder properties Gradation Volumetric properties

Several engineering properties and performance testing have been adopted by many researchers to investigate the effect of recycled materials on AC mixture durability. To evaluate RAP mixtures, researchers have measured volumetric properties, rutting resistance, moisture sensitivity, resistance to fatigue cracking, dynamic modulus, indirect tensile strength, resistance to reflective cracking, fracture energy, cracking tolerance index, as well as low temperature compliance and strength. A nationally accepted standard mix design method for AC mixtures containing high RAP is not available to date, however, various state agencies across the country are advocating for the application of the BMD approach. This is because the BMD approach does not only include mixture volumetric properties but performance evaluation as well, such as rutting, cracking, and moisture susceptibility.

The BMD approach was established to evaluate rutting and cracking, by defining the Optimum Binder Content (OBC) as in between the minimum binder content per cracking criterion and maximum binder content per rutting criterion. There are essentially three types of balanced mix design as per the National Cooperative Highway Research Program (NCHRP) report for the Development of a Framework for Balanced Mix Design (West et al. 2018):

1. Volumetric Design with Performance Verification
2. Performance-Modified Volumetric Mix Design
3. Performance Design

The following section summarizes several state practices with regard to the BMD (West et al. 2018). California currently uses performance-based specifications coupled with a mechanistic empirical design approach when performing mixture designs for high-volume roadways. In total, seven projects have been constructed with this approach. Tests used on these projects include AASHTO T320, *Standard Method of Test for Determining the Permanent Shear Strain and Stiffness of Asphalt Mixtures Using the Superpave Shear Tester (SST)*, AASHTO T321, *Standard Method of Test for Determining the Fatigue Life of Compacted Asphalt Mixtures Subjected to Repeated Flexural Bending,* and AASHTO T324, *Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Asphalt Mixtures* (AASHTO T320 2007, AASHTO T321 2017, AASHTO T324 2019). It should be noted that the performance testing is conducted on plant-mixture specimens subject to short-term aging and not on lab batched specimens.

Current Florida specifications require Asphalt Pavement Analyzer (APA) testing with a maximum rut depth of 4.5mm after 8,000 cycles. The APA test is performed during the mixture design phase and only on projects that are located in the Panhandle Region of the state. This is the case because the Panhandle Region of Florida experiences more rutting than any other region in Florida. Past research has also been done using the Flow Number (FN) and HWTT to evaluate rutting. Additionally, the IDT strength testing and the Texas Overlay Test (OT) have been conducted to evaluate cracking. These tests, however, are currently not implemented in the Florida DOT specification.

Current Georgia specifications require APA testing for moisture susceptibility. Based on climatic conditions and pavement location, the state of Georgia utilizes different test temperatures. Moisture susceptibility of asphalt mixtures is also monitored through the IDT test (TSR). This test is important to the region due to stripping issues associated with the state's aggregate source. Additionally, the HWTT can be conducted to evaluate stripping issues.

The current Illinois DOT specification uses a volumetric mixture design with performance verification. Mixtures are initially designed based on volumetric properties and tested for tensile strength. After completing the initial design, the mixtures are evaluated using the Illinois Flexibility Index Test (I-FIT) cracking and HWT rutting tests. At the start of production, a 300-ton test strip is constructed, and the listed performance tests are verified. Current research projects may dictate the future index test thresholds. To date, the I-FIT has a minimum criterion of 8.0 and the HWTT criterion varies based on virgin binder grade.

Iowa currently uses the Superpave volumetric approach when designing asphalt mixtures, while also moving forward with the BMD approach. During the design, asphalt mixtures are subject to volumetric and HWT testing, which varied based on the traffic level, binder grade, and additives within the mixture. Traffic levels that are high or very high require more passes until the stripping inflection point than low traffic pavements; and all samples must be short term aged prior to testing. Fatigue resistance of asphalt interlayers is also monitored with the Bending Beam Fatigue (BBF) test. Samples are subjected to 100,000 cycles with a micro strain of 2,000. This evaluates the resistance to bottom-up fatigue cracking within the pavement. The state of Iowa is also considering implementing a Disc Shape Compact Tension (DCT) test to evaluate thermal cracking potential of pavements.

Louisiana uses the HWTT to evaluate rutting resistance and the semi-circular bend test to evaluate cracking resistance. These tests are conducted along with conventional volumetric mixture design. The HWTT samples are short-term aged, while the Semi-Circular Bend (SCB) samples are long-term aged. Since the state of Louisiana typically does not encounter rutting issues, the BMD approach generally leads to mixtures containing higher asphalt contents. In 2016 the specification was updated to include high and low volume roads with differing levels of design gyrations. This was done to increase the Voids in Mineral Aggregate (VMA) and the Voids Filled with Asphalt (VFA) of the mixtures.

Minnesota currently uses the DCT test to evaluate low temperature cracking (thermal Cracking) of mixtures. Fracture energy requirements vary based on the pavement traffic level. Research is also underway to determine whether or not to conduct testing on plant-produced or laboratory-produced samples.

New Jersey uses the BMD approach for roughly ten percent of the total material produced each year. This design approach uses the traditional method of volumetric design, followed by performance testing. New Jersey conducts the APA, BBF, OT and IDT tests for performance. Each test has a different test temperature, but all specimens are conditioned for two hours at compaction temperature prior to being compacted. All testing is conducted by the New Jersey DOT. If the asphalt mixtures fail to meet the performance test requirements, the contractor must redesign the mixture.

New Mexico is currently using the BMD approach only for test sections of the existing projects. The New Mexico DOT wants to evaluate the performance of the mixtures prior to implementing any new mixture design approach. If implemented, the plan is to develop a performance specification for the HWTT. This test would be used to evaluate mixture stripping and rutting potential. The mixtures with different binder grades would likely be tested at different temperatures for the HWTT.

Current mixture designs in Ohio utilize the APA test to evaluate mixture rutting for mixtures that do not comply with the fine aggregate angularity criteria. These mixtures are short-term aged for two hours at compaction temperature prior to compaction. Depending on the stress level of the pavement, different mixtures are required to resist varying rut depths after 8,000 cycles. For mixtures being placed on bridge decks, the BBF test must be conducted in addition to the APA test.

Oklahoma currently follows the Superpave volumetric mix design process and additionally requires the HWTT and IDT test. Preliminary specifications are currently being drafted for the implementation of the BMD Performance-Modified Volumetric Design. Before the specification can be drafted, field mixture performance must first be monitored and validated. Oklahoma plans to monitor performance of both mixture design and production samples using the HWTT, I-FIT, Cantabro test, and IDT test. Short-term aging of the HWTT and IDT test will be conducted at compaction temperature for two hours. A long-term aging protocol for the I-FIT and Cantabro tests has yet to be determined.

South Dakota currently uses the Superpave volumetric mixture design method coupled with the APA and IDT tests. During production, the contractor will monitor performance though these tests and the agency will verify their results. APA testing is conducted at the PG high temperature of the binder and the TSR criteria is a minimum of 80%. Research is ongoing using the DCT and SCB tests to evaluate low temperature cracking.

Texas currently uses the volumetric mixture design with performance verification approach on a percentage of its premium mixtures. Once a volumetric mixture design is completed, performance verification using the HWTT, and OT is conducted. Regardless of the binder PG, HWTT are conducted at 122°F. Additionally, samples for both performance tests are aged for two hours at compaction temperature prior to compaction. After 3 binder contents are evaluated, the optimum binder is selected where both HWTT and OT requirements are satisfied.

Current Utah mixture design processes follow the Superpave volumetric approach. The HWTT is additionally run on short-term aged samples to evaluate rutting and moisture resistance of the asphalt mixture. HWTT temperature is dictated by the PG high temperature of the asphalt binder. Consideration to include a BMD approach is currently underway. Utah is investigating the use of mixture specimens in the bending beam rheometer for low temperature cracking. The DOT is also investigating the use of the I-FIT to evaluate mixture performance at intermediate temperatures. Thresholds for these tests are yet to be determined.

On pilot projects within Wisconsin, the State has developed specifications for the HWTT, SCB test, DCT test, IDT test, and extracted binder analysis. It is important to note the agency has implemented these mixtures that contain more than 25% RAP and lower design air void level of 3.5% on the pilot projects. Short-term aging of 4-hour at 275°F is done on HWTT samples and long-term aging of 12-hour at 275°F is done on the DCT test and SCB test loose mixture samples. These specifications have been implemented on some projects at the city level, as well as the pilot projects. Further testing is being done using the FN test and I-FIT, but no specifications have been developed yet. The HWTT is used to evaluate moisture susceptibility and rutting, while the DCT and SCB tests are used to evaluate low temperature and intermediate temperature cracking, respectively.

Washington State is currently using a form of balanced mix design method, based on WSDOT Standard Specifications, which includes volumetric mix design with the HWT rutting test, coupled with a maximum dry tensile strength of 175 psi as a cracking test. A recent NCHRP project focused on improving the durability of AC mixtures by determining quality control and acceptance testing framework of the BMD (Yin et al. 2020). The research team evaluated five BMD mixes including Laboratory Mixed-Laboratory Compacted (LMLC), Hot Compacted Plant Mixed-Laboratory Compacted (HC-PMLC), and Reheated Plant Mixed-Laboratory Compacted (RH-PMLC) form the 2018 National Center for Asphalt Technology (NCAT) Test Track research cycle. The mixtures were examined using following rutting tests: HWTT, High Temperature Compact Shear (HT-CS), High Temperature Indirect Tensile (HT-IDT) Strength Test. The cracking tests considered were I-FIT, Cracking Index (CT-Index), and the Texas Overlay Test. The researchers developed a procedure for Quality Control (QC) and acceptance testing for BMD.

The are many recycling agents available that can be considered to improve the performance of mixtures containing high RAP and RAS. There are literally hundreds of journal articles on this topic in the literature. The NCHRP Project 09-58 specifically focused on the selection and use of recycling agents to improve high RAP and RAS mixture performance (Martin et al. 2019). The key findings of this extensive project included:

- There are many recycling agent base stocks (i.e. petroleum, plant, animal, waste oils, etc.).
- Some recycling agents only soften the binder they are blended with, while others may soften and change the rheological properties of blended binders.
- Some recycling agents are much more susceptible to aging than others.
- The compatibility of virgin and recycled materials with recycling agents is critical.
- It is important to use performance tests to evaluate rutting and cracking performance of high RAP/RAS mixtures made with recycling agents.
- It is important to long-term age cracking performance test specimens.

- Good quality high RAP/RAS mixtures can be made with softer binders and recycling agents, though the required recycling agent doses may be higher than those traditionally used.

NAPA recently published a practical guide for using recycling agents in asphalt mixtures that summarizes many techniques for selecting and integrating recycling agents in mix designs, as well as evaluating them using different levels of rigor depending on potential performance risk (Hand and Martin 2020).

2.3 ΔTc Parameter

Several approaches have been made to date to improve the durability of AC mixtures in the mix design phase by increasing the effective binder content, using a smaller Nominal Maximum Aggregate Size (NMAS) mixture, polymer-modified (PM) asphalt binders, softer binder grade with recycled mixtures and implementing a BMD approach for mix design. In addition, there has been growing interest in the concept of ΔTc parameter, an asphalt binder property that can be useful for identifying binders that are particularly susceptible to aging, and thus potentially susceptible to cracking, especially non-load related (block) cracking.

The asphalt binder property ΔTc is gaining attention in the asphalt industry as it can help improve AC mixture performance. The ΔTc parameter is an indicator of the loss of relaxation properties of an asphalt binder (virgin asphalt binders, and binders that may contain RAP/RAS, Polyphosphoric Acid (PPA), REOB or other asphalt additive), which can be used to evaluate aging susceptibility and cracking potential. The concept of ΔTc was originally proposed to primarily investigate block cracking of airport pavements. Three asphalt binders were evaluated after being subjected to 20-hour, 40-hour, and 80-

hour of PAV aging using Dynamic Shear Rheometer (DSR) tests, a DSR monotonic binder fatigue test, the ductility test, the force ductility test, and Bending Beam Rheometer (BBR) test (Anderson et al. 2011). The laboratory study showed that asphalt binder properties such as DSR function ($G'/(\eta'/G')$) and ΔTc correlated well with asphalt binder ductility and showed promising results when identifying binders prone to block cracking. Additionally, the cracking warning limit for ΔTc of -2.5°C and ΔTc cracking limit of -5.0°C were proposed. Following this, the AASHTO PP78-17, *Standard Practice for Design Considerations When Using Reclaimed Asphalt Shingles (RAS) in Asphalt Mixtures* was revised to include a provision for using ΔTc with the -5.0°C criterion, but it was noted that this limit could be adjusted based on local experience (AASHTO PP78 2017). The intent of adding the provision was to access the effect of heavily oxidized RAS binder on blended (virgin with RAS) binder embrittlement.

To show the impact of RAP on ΔTc, the binder data from NCHRP Project 9-12 was analyzed and illustrated how the addition of RAP binder to virgin binder generated more negative ΔTc (Christensen et al. 2019). The relationship between the ΔTc parameter and the fatigue life of AC mixtures with RAP was investigated. It was determined that lower ΔTc values related to lower fatigue life of asphalt mixtures containing RAP.

The evaluation of a range of asphalt binders from Texas indicated that ΔTc values measured on the same binder grade from the same source, at different days of production could be significantly different (Karki and Zhou 2018). This observation suggested that binders of the same grade from the same source could vary during production from one day to another. Reinke conducted a study to compare ΔTc and other rheological properties with field performance data finding that ΔTc showed good correlation with field cracking after six years of service life (Reinke 2017). The correlation was slightly better for 40-hour PAV aging. Researchers have tried to correlate ΔTc with field performance data and mixture cracking test results, experiencing variable degrees of success. However, the general relationships between those listed parameters have been fair to good.

The ΔTc parameter has been shown related to block cracking, but sometimes ΔTc is indirectly related to the other types of asphalt pavement distress such as fatigue cracking, longitudinal cracking, transverse cracking, reflective cracking, edge cracking, raveling and potholes. Asphalt Institute published an extensive document titled, "State-of-the-Knowledge: Use of the ΔTc Parameter to Characterize Asphalt Binder Behavior" in 2019 (Asphalt Institute 2019). According to the Asphalt Institute, the ΔTc parameter has been implemented by ten State DOTs. Table 2 shows the summary of ΔTc specifications adopted by different State DOTs.

Table 2: ΔTc Specification Requirement Adopted by Different State DOT's (Asphalt Institute 2019)

Agency	ΔTc requirement, °C	PAV Aging Duration, hrs.	Status
Florida DOT	≥ -5.0	20	Current
Utah DOT	≥ -2.0	20	Current[2]
PANYNJ	≥ -5.0	40	Current
Vermont DOT	≥ -5.0	40	Current
Maryland DOT	≥ -5.0	40	Current
Kansas DOT	≥ -5.0	40	Current
Ontario MTO	≥ -5.0	20	Current
Texas DOT	≥ -6.0[4]	20	Current[4]
Oklahoma DOT	≥ -6.0	20	2020[3]
Delaware DOT	≥ -5.0	40	2020[3]

[1]Asphalt Institute web site for current asphalt binder specification database. (www.asphaltinstitute.org)
[2]Applies to binders with ≥ 92°C temperature spread; BBR creep stiffness ≥ 150 MPa
[3]Applies to project tendered for bid beginning
[4]Only applies to BMD projects. For comparison, TxDOT requirement is shown using ΔTc computed by ΔTc =Tc,s, - Tc,m ; actual requirement is ΔTc ≤ 6°C using the equation ΔTc =Tc,s, - Tc,m.

2.4 Key Literature Review Highlights

The literature review indicated that good long-term performance of mixtures containing RAP or RAS can be obtained if responsibly designed and properly constructed. It also indicated that RAP or RAS dose increases more diligence to obtain the desired performance. The literature review identified several practices used by DOTs for improving performance of AC containing recycled materials including:

- Periodically reviewing in-service pavement performance to assess the impact of recycled material types and levels on performance.
- Using softer binders or recycling agents when recycled materials are used.
- Including ΔTc when specifying virgin asphalt binders.
- Techniques to increase virgin binder content including:
  - Reducing the mix design compaction effort (e.g. number of design gyrations).
  - Using regressed design air voids (e.g. 3.0 to 3.5% rather than 4.0%).
  - Using the bulk dry specific gravity of virgin and RAP aggregates (not the effective specific gravity) for calculating VMA.
  - Increasing the mix design VMA criterion over the AASHTO M323, *Standard Specification for Superpave Volumetric Mix Design,* minimum by 0.5% or requiring the mix design minimum to be met in production.
  - Using a discounting technique for RAP/RAS binder.
- Use of mixture performance tests (rutting and cracking) for mix design.
- Implementation of a BMD methodology.
- Including VMA and Dust to Asphalt ratio as acceptance quality characteristics.

- Using mat and joint density specifications.
- Using Percent Within Limits (PWL) specifications to improve consistency.

3 EXPERIMENTAL PLAN

The scope of this effort focused on quantifying the current and planned use of recycled asphalt materials in AC, as well as evaluating recent materials technologies that could be integrated into WSDOT policy, specifications, and procedures to optimize the recycled materials amounts without compromising pavement performance. This effort involved the following collaboratively conducted by WSDOT, University of Washington (UW), and University of Nevada-Reno (UNR).

- Task 1: WSDOT & Industry Recycling Practices, Goals and Collaboration
- Task 2: AC Pavement Performance Review
- Task 3: Review of the Literature
- Task 4: Field Sampling and Laboratory Materials Testing/Evaluation
- Task 5: Recommended Improvements Documented including Draft WSDOT Pavement Policy, Specification, and SOP Language

Table 3 summarizes the overall experimental plan, performed by UNR for various specimen types under different contract numbers, including Field Mixed-Field Compacted (FMFC), Field Mixed-Laboratory Compacted (FMLC), and Laboratory Mixed-Laboratory Compacted (LMLC). Detailed mixture and binder testing matrices along with different material types and sources can be found in APPENDIX A: EXPERIMENTAL PLAN.

Table 3: UNR Laboratory Experimental Plan

Specimen Types		FMFC	FMLC	LMLC
ID	Contract No.	7706, 8438, 8128, 8624, 8433, 8441, and 8465	9145, 9262/9229, and 9231	9145, 9262/9229, and 9231
	RAP %	<20 - 45%	0 - 40%	0 - 40%
Binder Properties	Extraction and Recovery	✓	✓(1)	✓(2)
	Superpave PG	✓	✓(1)	✓(3)
	Blending Charts			✓
	Superpave PG for the Blended Binder			✓(4)
Aggregate Gradations	Verification of Virgin and RAP Aggregates with JMF			✓
Mix Design and Volumetrics	Replicate FMLC JMF (5)			✓
	Mix Design Tweaks (6)			✓
	Cores Dimensions Measurements	✓		
	Bulk Measurements	✓	✓	✓
Performance	IDT (Strength and CT-Index parameters at 3 conditions)	✓	✓	✓
	Prepare HWTT Specimens		✓	
	E* Testing			✓

(1) : Only for 9145 and 9231 contracts

(2) : Extraction and Recovery performed for the RAP stockpile.

(3) : PG for virgin binder and recovered binder from RAP stockpile.

(4) : Physical Blending for virgin binder, recovered binder, and recycling agent if any.

(5) : Replicating Job Mix Formula (JMF) provided by WSDOT with same virgin and recycled materials.

(6) : Mixture tweaks include RAP%, virgin binder grade, virgin binder source while conserving same JMF final gradation and binder content %.

Field Mixed Field Compacted (FMFC): Existing performance was evaluated by obtaining field cores from seven typical WSDOT low RAP/no RAS (≤20% RAP) and high RAP (>20% RAP and/or RAS) contracts: 7706, 8438, 8128, 8624, 8433, 8441, and 8465. WSDOT obtained the cores and UW observed the sampling and pavement performance at the time. The field cores were sent to UNR for testing. Bulk specific gravity (Gmb) measurements were made prior to IDT testing and theoretical maximum specific gravity (Gmm) measurements were done after IDT testing to calculate the air voids (AV%) (AASHTO T209 2012, AASHTO T166 2016). The IDT testing of the cores was performed under three conditioning levels including:

1. Unaged Unconditioned (UU) –no moisture conditioning
2. Unaged Conditioned (UC) – per AASHTO T283, *Standard Method of Test for Resistance of Compacted Asphalt Mixtures to Moisture Induced Damage* (saturated and subjected to1 freeze-thaw cycle), and
3. Long-term aged Unconditioned (AU) – compacted specimens long-term aged 5 days at 185°F (85°C).

Afterwards the asphalt binder was extracted and recovered from unconditioned cores (unaged and long-term aged), and the Superpave PG system was utilized to grade the recovered binder and determine ΔTc. Figure 1 illustrates the test protocol conducted on field cores. The IDT strength results, and asphalt binder properties measured at the laboratory were used to assess the impact of recycled materials amount and aging level on the mixture cracking behavior.

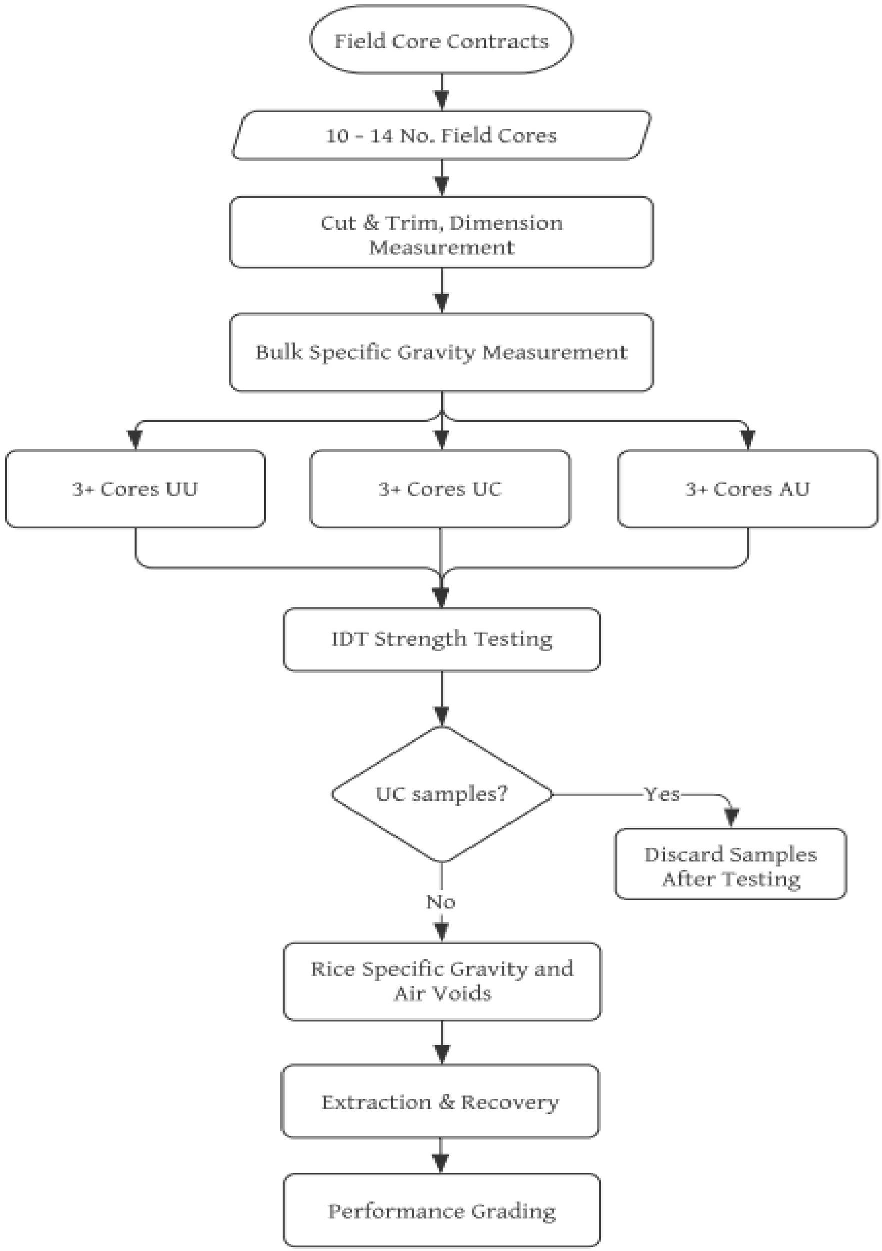


Figure 1: Flowchart A FMFC Test Plan

Field Mixed Laboratory Compacted (FMLC): Figure 2 presents the laboratory experiments conducted on FMLC samples. Loose field mixture samples were obtained from four different contracts (9145, 9262, 9229, and 9231) by WSDOT and shipped to UNR. The mixture was compacted to a height of 95 mm for IDT testing, using a Superpave Gyratory Compactor, then bulk measurements and AV% were verified to be within 7±0.5% prior to performance testing. The performance evaluation consists of conducting IDT tests under three conditions including:

1. Unaged Unconditioned (UU) –no moisture conditioning
2. Unaged Conditioned (UC) – per AASHTO T283 (saturated and subjected to one freeze-thaw cycle), and
3. Long-term aged Unconditioned (AU) – compacted specimens long-term aged 5 days at 185°F (85°C).

Additionally, HWTT specimens were prepared and shipped to WSDOT for testing. After IDT strength testing, the unconditioned (unaged and long-term aged) samples were subjected to centrifuge extraction, with second screening, and recovery in order to grade the binder as per the Superpave PG system.

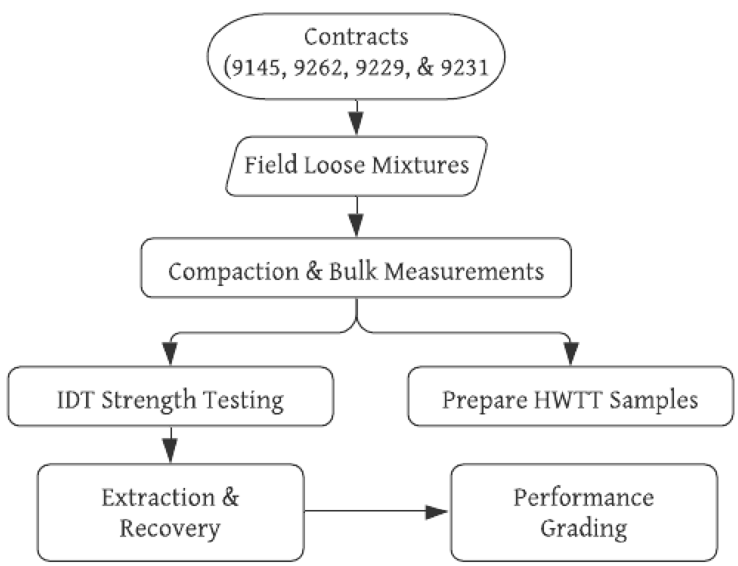


Figure 2: Flowchart B FMLC Test Plan

Laboratory Mixed Laboratory Compacted (LMLC): Figure 3 presents the various tests conducted on laboratory samples mixed from virgin and recycled materials obtained from Washington, including virgin aggregate stockpiles, RAP stockpiles, virgin asphalt binder, recycling agent (RA), and anti-stripping agent. Two binder suppliers who supply the Eastern portion of Washington state and three binder suppliers who supply the Western portion of the state provided virgin binders for this effort. WSDOT sampled and supplied the aggregates and binders to UNR. The materials were used to replicate the mix design provided by WSDOT that was the JMF used to produce the FMLC specimens. These mix designs were subjected to several variations so their impact on cracking performance could be evaluated. The variations included virgin binder source, virgin binder grade, RAP dose, and recycling agent. Furthermore, Superpave performance grading was done on each of the virgin binders after Rolling Thin Film Oven (RTFO)+20-hour PAV aging as well as RTFO+40-hour PAV aging, and ΔT_c was determined. Subsequently, the LMLC specimens were subjected to IDT strength (under UU, UC, and AU conditioning) and dynamic modulus (E*) testing for performance evaluation.

Asphalt binder was extracted from RAP samples and graded separately to develop the asphalt binder blending charts, using the true grade of multiple virgin binders. Virgin binder was also physically blended with the recovered RAP binder, and RA, if any, proportionally to the mix design and graded as shown in Figure 4, while also identifying ΔT_c.

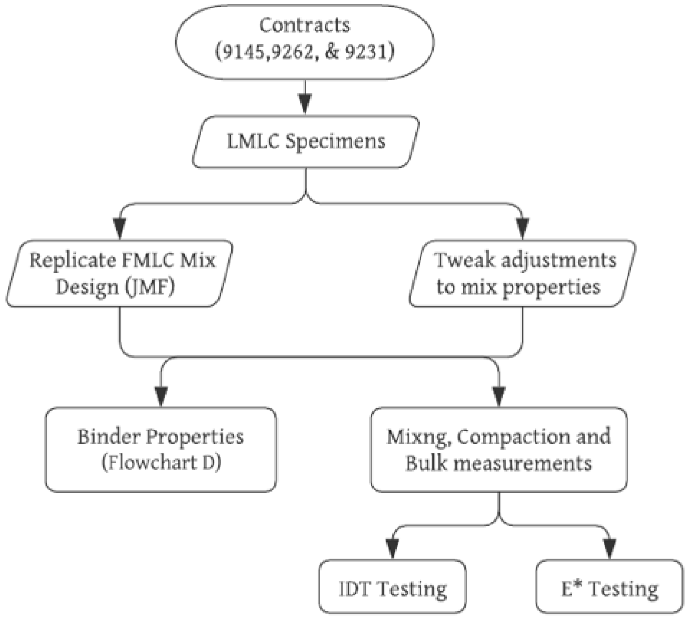


Figure 3: Flowchart C LMLC Test Plan

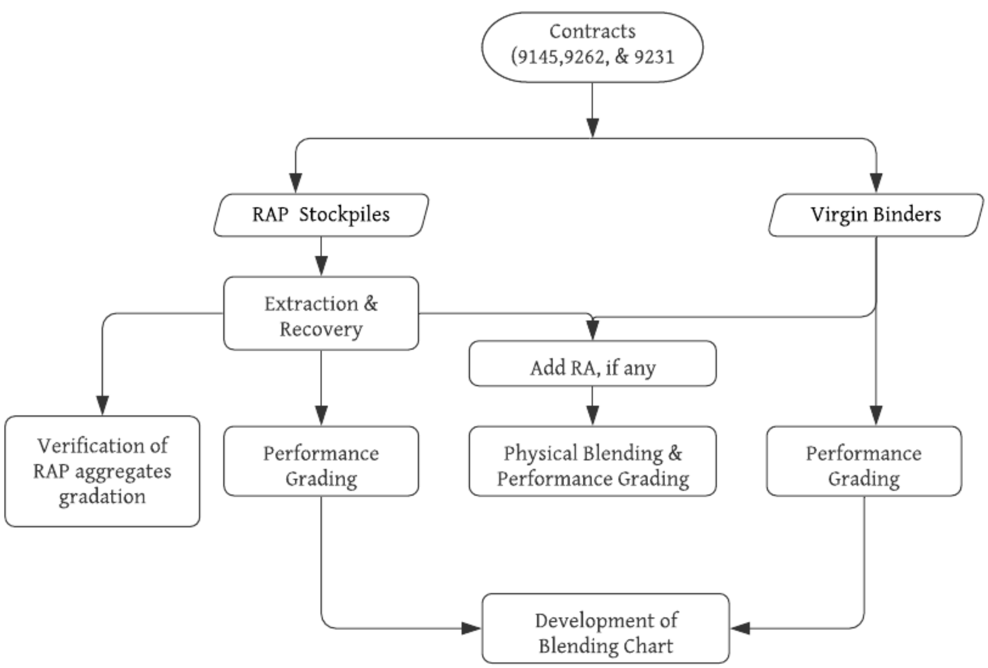


Figure 4: Flowchart D LMLC Binder Testing Plan

Table 4 presents the experimental analysis, and statistical correlation executed between various binder rheological properties, mixture components, laboratory performance test results, and field performance data.

Table 4: UNR Analysis Plan

Specimen Types		FMFC	FMLC	LMLC
ID	Contract No.	7706, 8438, 8128, 8624, 8433, 8441, and 8465	9145, 9262/9229, and 9231	9145, 9262/9229, and 9231
	RAP %	<20 - 45%	0 - 40%	0 - 40%
Analysis and Statistical Correlation	Statistical comparison between blending charts and physical blending			✓
	Impact of binder supplier			✓
	Variability between LMLC and FMLC Cracking Test Results		✓	✓
	Performance comparison with Varying RAP Amounts			✓
	Correlation between IDT strength testing parameters			✓
	ΔTc regression with mixture performance	✓		✓
	PMS data regression with ΔTc and mixture performance	✓		

4 USING COST, MIX DESIGN, CONSTRUCTION, AND PERFORMANCE DATA TO INFORM HOT MIX ASPHALT PAVEMENT POLICY AND STANDARDS

4.1 WAPA Survey and WAPA/WSDOT Interviews

Howell conducted a survey of WAPA members and interviewed WAPA and WSDOT staff related to use of RAM in asphalt mixture (Howell 2019). WAPA interviewees indicated that the average estimated asphalt content of RAP mixtures is about 5.0%. The top three barriers identified by WAPA survey respondents with the use of high-RAP mixtures were: meeting mix design volumetric requirements (25% of respondents); stockpile management requirements (19%) and other (19%). The other barriers identified include inclement weather (e.g., high moisture content of RAP stockpiles after heavy rain), inconsistencies between contractor and WSDOT testing procedures, and WSDOT stockpile management and testing frequency requirements. Only 13% of the survey respondents and one of seven WAPA interviewees indicated that high-RAP mixtures were used on WSDOT projects. Some WAPA interviewees indicated that they conduct high-RAP mixture designs for other agencies (e.g., cities, counties). The only WAPA interviewee with high-RAP mixture experience stated that they incur additional costs because of WSDOT's additional mixture testing and stockpile management requirements. According to the survey, these additional WSDOT requirements represent the top two barriers to using high-RAP mixtures. During the interviews, WSDOT staff expressed concerns about the potential for high RAP mixtures to cause an increase in the severity and extent of pavement distresses. Premature cracking is the predominant failure type for high-RAP mixtures due to rigidity and stiffer asphalt binder.

4.2 Assessment of High-RAP Influence on WSDOT Field and Performance Data

WSDOT field and performance data were used to characterize the influence of high-RAP mixtures on performance. This method used actual field data, and its usefulness relies on quality data. Also, there are many unmeasured variables (e.g., construction quality, underlying pavement/soil conditions, etc.) that could influence dependent performance variables beyond condition (e.g., asphalt content, density) data. Although industry perspectives can assist in results interpretation, this method is likely to only identify very broad, strong trends, and sometimes expected trends are not seen above the noise of unmeasured variables. This analysis uses and compares findings from the literature, field data, and industry perspectives. At times, the findings from these sources do not all agree. Additionally, almost all of the high-RAP mixtures came from one contractor and were constructed in Western Washington, where mixture prices have been historically higher and pavement life has been historically longer (Howell 2019).

Because this paper only includes five years of high-RAP field and performance data, trends that take longer to develop (notably, cracking) may not have had enough time to express themselves fully. The discussion points that follow may change as the high-RAP sections continue to age. The construction cost analysis weighted by quantity reveals that high-RAP mixtures cost about $89 per ton and the up-to-20%-RAP mixtures cost about $84 per ton, a difference of about $5 per ton. The statistical analysis of the construction bid price by contract of the high-RAP versus the up-to-20%-RAP contracts fails to reject that the difference between the means is zero. The statistical evidence suggests that there is insufficient evidence to conclude that high-RAP contracts produce a different construction bid price than up-to-20%-RAP contracts. This finding conflicts with the literature that high-

RAP mixtures are cheaper. A likely explanation for this finding is location. About 64% of all contracts (about 55% of tonnage) and about 88% of high-RAP contracts (about 80% of tonnage) are in Western Washington where the average weighted cost is about $11 per ton higher than Eastern Washington.

There is insufficient statistical evidence to conclude that high-RAP contracts produce a different overall average weighted structural and rutting condition than up-to-20%-RAP contracts. However, the cracking data show a couple of the high-RAP with good condition values but are slightly lower than the up-to-20%-RAP contracts at the same age. These contracts are worth monitoring for future analysis. Conversely, the rutting data show that high-RAP contract rutting performance may be trending higher than up-to-20%-RAP contracts for the oldest contracts, age four. During this time, the average weighted rutting for high-RAP mixtures (0.08 inches) is about half of up-to-20%-RAP mixtures (0.14 inches). Given the literature findings, Howell suggested a possible interpretation that high RAP mixtures perform similarly to or better than virgin RAP mixtures, particularly in terms of rutting resistance. However, because the high-RAP data is limited at this age (three contracts), it is difficult to conclude anything other than the two mixtures are no different (Howell 2019).

The high-RAP mixtures exhibit a slightly lower average weighted field density of 93.06% versus 93.25% for up-to-20%-RAP mixtures, a difference of about 0.2%. The statistical analysis suggests that the difference between these means is statistically significant. Of note, 20 of 37 (55%) high-RAP contracts exhibit a field density of less than 93% versus 78 of 210 (37%) up-to-20%-RAP contracts. This is consistent with the literature that the desired density is sometimes challenging to achieve with high-RAP mixtures, particularly with joint density.

The high-RAP mixtures exhibit a lower average weighted field measured asphalt content of 5.2% versus 5.4% for up-to-20%-RAP mixtures, a difference of about 0.2%. The statistical analysis suggests that the difference between these means is statistically significant. A possible interpretation is that the asphalt content is low for high-RAP mixtures due to a lower-than-expected contribution from the RAP. A lower asphalt content may lead to increased raveling and surface cracking. The high-RAP mixtures exhibit a lower average weighted VMA of about 13.9% versus 14.1% for up-to-20%-RAP mixtures, a difference of about 0.2%. There is insufficient statistical evidence to conclude that high-RAP contracts produce a different overall average weighted VMA than up-to-20%-RAP contracts. Because the literature is unclear on RAP's impact on VMA, it is difficult to determine the lower VMA for high-RAP mixtures. Given the slightly lower density (i.e., higher air voids) of high-RAP mixtures, a possible interpretation of the lower VMA may be due to a lower effective asphalt content potentially caused by a higher amount of fines in the mix. At age four, the data show that the high-RAP contract structural performance may be trending higher with a higher VMA ($R^2 = 0.801$). However, because the high-RAP data is limited at this age (three contracts), it is difficult to conclude anything other than the two mixtures are no different. The condition data analysis does not reveal a clear trend between density and cracking/rutting performance; however, the data show increased cracking and rutting with age for all densities. This finding does not align with literature and some survey/interview comments that elevated field density produces increased performance. This does not imply that the literature and survey/interviews are incorrect, but rather there is not available field evidence to support them (Howell 2019).

4.3 Conclusion

Based on investigating the impacts of high-RAP mixtures on performance in Washington State by synthesizing WSDOT mix design, Quality Assurance (QA), and performance data for mixtures completed between 2013 and 2017 as well as relevant industry perspectives Howell provided the following several conclusions (Howell 2019). Notably, that the high-RAP mixtures cost more likely because of location and have not shown significant performance benefits or issues on a statewide level over the last five years. The other conclusions were:

- The construction bid price of high-RAP mixtures slightly exceeds up-to-20%-RAP mixtures by about $5 per ton.
- There is no evidence that there is a difference between the cracking and rutting condition means for all mixtures.
- High-RAP mixtures have a slightly lower in-place density than up-to-20%-RAP mixtures by about 0.2% on average (93.06% for high-RAP mixtures, 93.25% for up-to20%-RAP mixtures).
- High-RAP mixtures have less asphalt than up-to-20%-RAP mixtures by about 0.2% on average (5.23% for high-RAP mixtures, 5.39% for up-to-20%-RAP mixtures).
- High-RAP mixtures have a slightly lower VMA than up-to-20%-RAP mixtures by about 0.2% on average (13.88% for high-RAP mixtures, 14.11% for up-to-20%-RAP mixtures).

The limited high-RAP data available (only data from the first five years of pavement life) reduces the ability to identify a compelling case that high-RAP mixtures are better or worse than mixtures with less RAP as per the box and whisker plots in Figure 5. It may be that performance differences will emerge in later years. The utility of the cost, mix design, field, and performance data method presented in Howell paper (1) analyzes data over a five-year period of time and (2) uses the data to compare with literature findings and industry perspectives. The numerous variables not analyzed (e.g., paving conditions) necessarily make the standard of proof quite high to show significant differences between high RAP mixtures and HMA performance. As a result, some analyses (e.g., cracking/rutting performance) showed no significant differences. This does not imply that there are no differences, but rather there is not enough evidence to identify them. Notably, performance reasons for high-RAP mixtures were not universally confirmed nor were they rejected. This could be because of the coarse nature of the comparison or because it is too early in the pavement life to identify significant performance differences (Howell 2019).

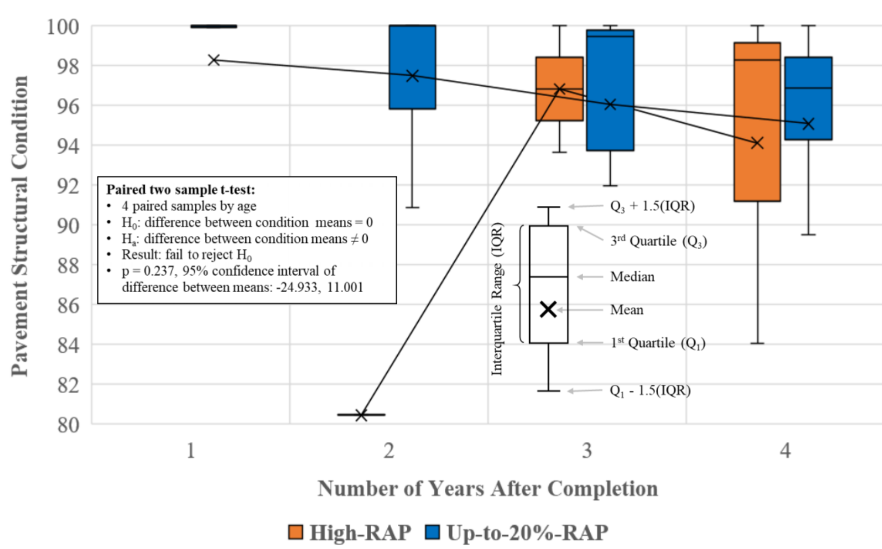


Figure 5: Average Weighted Pavement Structural Condition and Number of Contracts by Years After Completion for High-RAP and Up-to-20%-RAP (Howell 2019)

5 MATERIALS CHARACTERIZATION AND TEST DESCRIPTIONS

5.1 Materials Characterization

5.1.1 LMLC

The following sections on LMLC samples, describe the virgin and recycled materials employed to replicate FMLC samples per WSDOT JMF mix design presented in Appendices C, D, and E. WSDOT provided three different JMFs referring to contracts 9145, 9262/9229, and 9231 as shown in Table 5. Note that the mix designs for contracts 9262/9229, and 9231 include the addition of a recycling agent (RA) and anti-strip additive (Zycotherm), respectively. Moreover, these virgin and recycled materials were mixed to reproduce new asphalt mixtures with variations in RAP dose, binder source and binder grades shown in Table 6.

Table 5: WSDOT Mix IDs

Contract	Mix ID	WSDOT Mix Type
9145 (Eastern)	MD170081	AC Class ½”
9262/9229 (Western)	MD180027	AC Class 3/8”
9231(Western)	MD180054	AC Class 3/8”

Virgin Asphalt Binder

Typical asphalt binders, collected from five different binder sources that supply in Eastern and Western climatic zones of Washington State, were used with various aggregate sources and RAP to produce LMLC AC mixtures. For each contract mix design, the binder grade typically used in Washington State and an additional grade that was a one grade dropped binder (PG-1), were obtained from Husky Asphalt, Idaho Asphalt Supply, Targa, U.S. Oil, or Western States Asphalt. Suppliers were randomly assigned supplier IDs of S1-S5.

Table 6: Summary of Evaluated Tweaked LMLC Mixtures

Mix Nomenclature	RAP (%)	RAP Source	WSDOT AC Class	Binder Type	RA (%)	Zyco (%)
9145-0-6428-S2	0	RAP I	1/2 inch	PG6428-S2	-	-
9145-20-6428-S2	20	RAP I	1/2 inch	PG6428-S2	-	-
9145-25-6428-S2	25	RAP I	1/2 inch	PG6428-S2	-	-
9145-20-5834-S2	20	RAP I	1/2 inch	PG5834-S2	-	-
9145-25-5834-S2	25	RAP I	1/2 inch	PG5834-S2	-	-
9262-0-58H22-S4	0	RAP II	3/8 inch	PG58H22-S4	-	-
9262-20-58H22-S4	20	RAP II	3/8 inch	PG58H22-S4	-	-
9262-25-58H22-S4	25	RAP II	3/8 inch	PG58H22-S4	-	-
9262-36-58S28-S4+RA	36	RAP II	3/8 inch	PG58S28-S4	3.1	-
9231-0-58H22-S5+Z	0	RAP	3/8 inch	PG58H22-S5	-	0.10
9231-20-58H22-S5+Z	20	RAP	3/8 inch	PG58H22-S5	-	0.12
9231-20-52H28-S5+Z	20	RAP	3/8 inch	PG52H28-S5	-	0.12
9231-25-58H22-S5+Z	25	RAP	3/8 inch	PG58H22-S5	-	0.12
9231-40-52S28-S5+Z	40	RAP	3/8 inch	PG52S28-S5	-	0.15
9231-40-52S28-S3+Z	40	RAP	3/8 inch	PG52S28-S3	-	0.15

The Superpave PG system following AASHTO R29, *Practice for Grading or Verifying the Performance Grade of an Asphalt Binder* (AASHTO R29 2014), was used to verify the asphalt binder grades, and then calculate ΔTc equivalent to the critical stiffness low temperature minus critical relaxation low temperature. The nomenclatures of different blended binders at various RAP amounts are tabulated in APPENDIX F: LABORATORY SAMPLES NOMENCLATURE. Each virgin binder was associated with two subsequent PG, based on the following conditioning:

- PG1: After RTFO + 20-hour PAV aging
- PG2: After RTFO + 40-hour PAV aging

Table 7 summarizes the different virgin asphalt binder IDs evaluated in this study, along with relative grade and supplier.

Table 7: Virgin Asphalt Binder Description

Virgin Binder ID	Climate Zone	Supplier	Supplier Grade
PG6428-S1	Eastern	S1	PG64-28
PG6428-S2	Eastern	S2	PG64-28
PG5834-S2	Eastern	S2	PG58-34
PG58H22-S3	Western	S3	PG58H-22
PG58S28-S3	Western	S3	PG58S-28
PG52S28-S3	Western	S3	PG52S-28
PG58H22-S4	Western	S4	PG58H-22
PG58S28-S4	Western	S4	PG58S-28
PG58H22-S5	Western	S5	PG58H-22
PG52H28-S5	Western	S5	PG52H-28
PG52S28-S5	Western	S5	PG52S-28

Virgin Aggregates

WSDOT obtained virgin aggregates from contractors and provided them to UNR as summarized in Table 8. Shamrock Construction provided the virgin aggregates for contract 9145 and Granite Construction provided the virgin aggregates for contracts 9262/9229, and 9231. Ten virgin aggregate stockpile materials were received as per three WSDOT JMFs. Initially samples of three unwashed stockpiles for contract 9145 were received. The stockpile gradations were verified upon receipt using the wet sieve analysis as per WSDOT Errata to WAQTC FOP for AASHTO T27/AASHTO T11, *Sieve Analysis of Fine and Coarse Aggregates* (WSDOT Errata to FOP for AASHTO T27_T11 2020). These gradations were verified to meet WSDOT JMF, which can be found in Appendix C. Additional virgin aggregate stockpile samples for contract 9145 were received, that had been washed and separated on each individual sieve by WSDOT. Similarly, the virgin stockpile samples for other contracts 9262/9229 and 9231 were received washed and separated on each individual sieve by WSDOT. For all three contracts, the blend gradation was verified to meet the WSDOT JMF mix designs, in Appendices C, D, and E. Virgin aggregates were prepared for mixing per WSDOT Test Method T724, *Method of Preparation of Aggregate for HOT MIX ASPHALT (HMA) Mix Design* (WSDOT Test Method T724 2017).

Table 8: Virgin Aggregates Description

Contract	Aggregate Stockpile Types	Contractor	Source
9145	3/8" aggregate bags	Shamrock	C-336
	Chip aggregate bags	Shamrock	C-336
	Sand aggregate bags	Shamrock	GT336
9262/9229	#4 - #8	Granite	OR27
	#4 – 0	Granite	OR27
	1/2" - #8	Granite	Z157
	Fine aggregate	Granite	G102
9231	3/8" Chips	Granite	D217
	#4-0	Granite	D217
	Fine aggregate	Granite	F160

Recycled Asphalt Pavement (RAP)

WSDOT obtained and provided samples of RAP materials for mix design to UNR. Characterizing RAP samples, involves extracting the asphalt binder so the binder and aggregates can be characterized separately. Centrifuge extraction, per AASHTO T164 *Standard Method of Test for Quantitative Extraction of Asphalt Binder from Hot Mix Asphalt (HMA)*, and recovery per American Society for Testing and Materials (ASTM) D5404 *Standard Practice for Recovery of Asphalt from Solution Using the Rotary Evaporator*, were performed so the RAP asphalt content (AC%) was determined (AASHTO T164 2014, ASTM D5404 2012). Then extracted RAP aggregate gradations were verified, while the recovered asphalt binder was graded. The Superpave PG system following AASHTO R29, *Practice for Grading or Verifying the Performance Grade of an Asphalt Binder* (AASHTO R29 2014), was used to verify the asphalt binder grades, and subsequently the ΔTc parameter was determined. The extracted RAP aggregates were subjected to wet sieve analysis following WSDOT Errata to FOP for AASHTO T30, *Mechanical Analysis of extracted aggregates* (WSDOT Errata to FOP for AASHTO T30 2020). RAP binder properties are used to develop the binder blending charts and determine blended binder properties based on the virgin and recovered RAP binder gradings. Each recovered RAP binder was associated with two subsequent PG, based on the following conditioning:

- PG1: After RTFO aging
- PG2: After RTFO + 20-hour PAV aging

The three different RAP sources used in this study and incorporated in LMLC samples were identified as follows:

- RAP I for contract 9145 (low RAP mixtures): from Shamrock Construction (original JMF had 0% RAP. RAP I was used only for making 20% and 25% RAP LMLC specimens)
- RAP II for contract 9262/9229 (high RAP mixtures): from Granite Construction
- RAP III for contract 9231 (high RAP mixtures): from Granite Construction

Physical blending of virgin binders and recovered RAP binders, as well as RA if exists in the mix design, was conducted as well. The physically blended binder was graded and compared to the binder grades estimated from blending charts using the final grade and ΔTc parameter. Different combinations of physically blended binders are shown in Table 9.

Table 9: Summary of Physically Blended Binders

Blended Binder ID	Virgin Binder ID	RBR % / RAP% used in Blending	RA (%)
9145-20-5834-S2	PG5834-S2	16.0% / 20.0%	-
9145-25-5834-S2	PG5834-S2	20.0% / 25.0%	-
9145-20-6428-S2	PG6428-S2	16.0% / 20.0%	-
9262-20-58H22-S4	PG58H22-S4	16.7% / 20.0%	-
9262-25-58H22-S4	PG58H22-S4	20.8% / 25.0%	-
9262-36-58S28-S4	PG58S28-S4	29.6% / 36.0%	-
9262-36-58S28-S4+RA	PG58S28-S4	29.6% / 36.0%	3.1%
9231-20-58H22-S5	PG58H22-S5	15.7% / 20.0%	-
9231-25-58H22-S5	PG58H22-S5	19.6% / 25.0%	-
9231-40-58S28-S5	PG58S28-S5	31.4% / 40.0%	-
9231-40-58S28-S3	PG58S28-S3	31.4% / 40.0%	-

Recycling Agent (RA)

Mix design MD180027 used for contracts 9262/9229 included Revive 1114 recycling agent. Revive was blended with virgin binder per the contract 9262/9229 JMF dosage, when preparing performance test samples and for rheological property measurements of physically blending binders.

Anti-Stripping Additive Zycotherm

An anorganosilane additive called ZycoTherm-SP, was blended with virgin binder and mixed with aggregates to produce the laboratory mixtures for contract 9231, per the contract JMF. Mixing of Zycotherm-SP with asphalt binders is performed in accordance with Zydex industries protocol for mixing ZycoTherm-SP with virgin binder.

5.1.2 FMLC

Loose field mix samples were obtained from various high RAP and low RAP projects during the production phase by WSDOT and the contractors. Shamrock Construction supplied the AC field mixture samples for the projects from Eastern climatic zones (Contract 9145 MD170081), while Granite Construction supplied the AC field mixture samples from Western climatic zones (Contract 9262 MD180027, Contract 9229 MD180027 and Contract 9231 MD180054). The corresponding JMF of these AC mixtures, was adopted for LMLC samples preparation as mentioned previously, and can be found in Appendices C, D, and E. It should be noted that UNR received separate loose field mixtures for both contracts 9262 and 9229, while both contracts have the same mix design ID MD180027. Field mixtures were reheated to the compaction temperature for approximately 2 hours, split to the required quantity, and compacted using a Superpave Gyratory Compactor to generate FMLC specimens. Compacted samples were then subjected to bulk specific gravity measurements in order to verify AV% were within the specified tolerance for performance testing. IDT strength testing was conducted on samples under three different conditions (UU, UC, and AU). HWTT compacted samples were provided to WSDOT for testing. Following IDT strength tests, the asphalt binder was extracted and recovered from unconditioned short-term and long-term aged samples so the binder properties could be determined. The Superpave PG was determined on the recovered binders from both contracts (9145 and 9231), under the following conditions:

- PG1: short-term aged unconditioned samples after RTFO
- PG2: short-term aged unconditioned samples after RTFO + 20-hour PAV aging
- PG3: long-term aged unconditioned samples after RTFO
- PG4: long-term aged unconditioned samples after RTFO + 20-hour PAV aging

5.1.3 FMFC

AC cores were collected by WSDOT from 7 pavements that had been in service for several years. They represented 4 high RAP/RAS and 3 low RAP/no RAS (up to 20% RAP) projects. Table 10 summarizes the field cores, related to WSDOT contracts, and the regions in Washington State they came from. Furthermore, WSDOT provided the pavement condition data for these pavement sections as function of time, based on its Pavement Management System (PMS).

The core dimensions and bulk specific gravities were measured prior to IDT strength testing under UU, UC and AU conditions. After IDT testing some of the field cores were used for theoretical maximum specific gravity determination, so that in-place air voids could be calculated. As with the FMLC samples, asphalt binder was extracted and recovered from the cores for Superpave PG determination. The Superpave PG was determined on the recovered binders from the 7 projects under the following conditions:

- PG1: short-term aged unconditioned samples after RTFO
- PG2: short-term aged unconditioned samples after RTFO + 20-hour PAV aging
- PG3: long-term aged unconditioned samples after RTFO
- PG4: long-term aged unconditioned samples after RTFO + 20-hour PAV aging

Table 10: Summary of FMFC Received Samples

Washington Climatic Zones	Field Projects per Recycled Materials Level			
	Low RAP/no RAS (≤20% RAP)		High RAP/or RAS (20%<RAP)	
	No. of Contracts	Contract ID	No. of Contracts	Contract ID
Olympic Region	1	8465	2	8441, 8624
South Central Region	1	7706[1]	1	8433
Northwest Region	1	8128	1	8438

[1]: 7706 Contract was further subdivided into 7706-East (from the eastbound) and 7706-West (from the westbound)

5.2 Binder and Mixture Nomenclature

In order to simplify the physically blended binder and LMLC mixture descriptions presented in further plots, the different combinations generated from virgin and recycled materials, were assigned specific nomenclature presented in Table 11.

Table 11: Blended Binder and LMLC Samples Nomenclature

<table>
<tr><th rowspan="2">Contract</th><th colspan="5">Materials</th><th colspan="2">Nomenclature</th></tr>
<tr><th>Binder Grade</th><th>Binder Supplier</th><th>RAP (%)</th><th>RA (%)</th><th>Zyco (%)</th><th>Physically Blended Binder ID</th><th>LMLC ID</th></tr>
<tr><td rowspan="5">9145 (Eastern)</td><td>PG64-28</td><td>S2</td><td>0</td><td>-</td><td>-</td><td>-</td><td>9145-0-6428-S2</td></tr>
<tr><td>PG64-28</td><td>S2</td><td rowspan="2">20</td><td>-</td><td>-</td><td>9145-20-6428-S2</td><td>9145-20-6428-S2</td></tr>
<tr><td>PG58-34</td><td>S2</td><td>-</td><td>-</td><td>9145-20-5834-S2</td><td>9145-20-5834-S2</td></tr>
<tr><td>PG64-28</td><td>S2</td><td rowspan="2">25</td><td>-</td><td>-</td><td>-</td><td>9145-25-6428-S2</td></tr>
<tr><td>PG58-34</td><td>S2</td><td>-</td><td>-</td><td>9145-25-5834-S2</td><td>9145-25-5834-S2</td></tr>
<tr><td rowspan="6">9262/9229 (Western)</td><td>PG58H-22</td><td>S3</td><td rowspan="2">0</td><td>-</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>PG58H-22</td><td>S4</td><td>-</td><td>-</td><td>-</td><td>9262-0-58H22-S4</td></tr>
<tr><td>PG58H-22</td><td>S4</td><td>20</td><td>-</td><td>-</td><td>9262-20-58H22-S4</td><td>9262-20-58H22-S4</td></tr>
<tr><td>PG58H-22</td><td>S4</td><td>25</td><td>-</td><td>-</td><td>9262-25-58H22-S4</td><td>9262-25-58H22-S4</td></tr>
<tr><td>PG58S-28</td><td>S4</td><td rowspan="2">36</td><td>-</td><td>-</td><td>9262-36-58S28-S4</td><td>-</td></tr>
<tr><td>PG58S-28</td><td>S4</td><td>3.1</td><td>-</td><td>9262-36-58S28-S4+RA</td><td>9262-36-58S28-S4+RA</td></tr>
<tr><td rowspan="10">9231 (Western)</td><td>PG58H-22</td><td>S5</td><td>20</td><td>-</td><td>-</td><td>9231-20-58H22-S5</td><td>-</td></tr>
<tr><td>PG58H-22</td><td>S5</td><td>25</td><td>-</td><td>-</td><td>9231-25-58H22-S5</td><td>-</td></tr>
<tr><td>PG52S-28</td><td>S5</td><td rowspan="2">40</td><td>-</td><td>-</td><td>9231-40-52S28-S5</td><td>-</td></tr>
<tr><td>PG52S-28</td><td>S3</td><td>-</td><td>-</td><td>9231-40-52S28-S3</td><td>-</td></tr>
<tr><td>PG58H-22</td><td>S5</td><td>0</td><td>-</td><td>0.10</td><td>-</td><td>9231-0-58H22-S5+Z</td></tr>
<tr><td>PG58H-22</td><td>S5</td><td rowspan="2">20</td><td>-</td><td rowspan="3">0.12</td><td>-</td><td>9231-20-58H22-S5+Z</td></tr>
<tr><td>PG52H-28</td><td>S5</td><td>-</td><td>-</td><td>9231-20-52H28-S5+Z</td></tr>
<tr><td>PG58H-22</td><td>S5</td><td>25</td><td>-</td><td>-</td><td>9231-25-58H22-S5+Z</td></tr>
<tr><td>PG52S-28</td><td>S5</td><td rowspan="2">40</td><td>-</td><td rowspan="2">0.15</td><td>-</td><td>9231-40-52S28-S5+Z</td></tr>
<tr><td>PG52S-28</td><td>S3</td><td>-</td><td>-</td><td>9231-40-52S28-S3+Z</td></tr>
</table>

5.3 Description of Laboratory Test Methods

5.3.1 Characterization of Asphalt Binder (Performance Grading)

The Superpave PG system is based on rutting, fatigue, and thermal cracking performance of asphalt binder. Performance grading of the asphalt binder was done in accordance with AASHTO R29, which includes two aging provisions (AASHTO R29 2014). Short-term RTFO aging (85 minutes at 325°F) simulates aging that occurs during plant production and construction (AASHTO T240 2017). Long-term PAV aging (20-hour at 194°F to 230°F and 305 psi) simulates 7-10 years of in-service life aging (AASHTO R28 2016). It is suggested that 40-hour PAV aging can simulate 15 to 20 years of in-service aging life (Anderson et al. 2011). Both 20-hour and 40-hour PAV aging were applied in some cases to grade the binder under both conditions. On the original (unaged) and short term aged (RTFO) asphalt binder, DSR test was conducted at high temperature to characterize the elastic and viscous behaviors of asphalt binder, thus characterizing rutting behavior of the mixture (AASHTO T315 2019). Following DSR testing, the Complex Shear Modulus (G*) and the phase angle (δ) are obtained. Complex shear modulus denotes the asphalt binder resistance to deformation under repeated shear loading, where higher complex shear modulus denotes higher shear resistance of the binder. The phase angle (δ) is the time lag that occurs between applied shear stress and resulting shear strain, where higher δ indicates that the viscous component is increasing in the binder. For a superior rutting resistance, it is desired to have a stiff and elastic binder which can be achieved with higher G* and lower δ. Following the DSR test on original binder and RTFO residue at high temperatures, the high PG of the evaluated binder can be determined. The MSCR was conducted subsequently at the high PG as per AASHTO R92, *Evaluating the Elastic Behavior of*

Asphalt Binders Using the Multiple Stress Creep Recovery (MSCR) Test (AASHTO R92 2018). The MSCR is used to identify the vehicle class corresponding to the tested asphalt binder: S, H, V, or E.

Another major distress in AC pavements is fatigue cracking that occurs in a later stage of pavement life due to load repetitions at intermediate and low temperatures. To obtain the intermediate grade, the DSR test is performed on RTFO and PAV asphalt binder residue at intermediate temperature. For high cracking resistance, a soft and elastic binder is preferred which can be achieved by decreasing G* and/or decreasing δ. After determining the intermediate grade, the thermal cracking behavior of the binder is examined using the BBR. Thermal cracking is another major distress in AC pavements, which develops at low temperature when the induced tensile stress in AC layer exceeds its tensile strength. In BBR test, a constant load of 100 g is applied for 240 seconds on simply supported beam, and relative binder stiffness as well as relaxation m-value, are measured at 60 seconds of loading time. The stiffness measures thermal stress developed while m-value indicates the rate of change in stiffness with loading time. Higher stiffness suggests that higher thermal stress is induced in pavement, so stiffness is limited to maximum value of 300 MPa. The rate of stiffness variation with time (m-value) is desired to be more than or equal to 0.300 to avoid the progression of thermal cracking (AASHTO T313 2019). The detailed specifications imposed by Superpave as per AASHTO M332 *Standard Specification for Performance-Graded Asphalt Binder Using Multiple Stress Creep Recovery (MSCR) Test* (AASHTO M332 2019). The ΔTc parameter was determined in accordance with ASTM D7643-16, *Standard Practice for Determining the Continuous Grading Temperatures and Continuous Grades for PG Graded Asphalt Binder* (ASTM D7643 2016). Accordingly, the ΔTc parameter was calculated as the continuous grading temperature for stiffness value minus the continuous grading temperature for the m-value. A summary of the laboratory experiments performed for grading different binder types, can be found in Table 12.

Table 12: Summary of PG Tests

Test	Virgin/ Physically Blended Binder	RAP Recovered Binder	Loose Mix Recovered Binder[1]	FMFC Recovered Binder[1]
DSR original	✓	✓	✓	✓
RTFO aging	✓	✓	✓	✓
DSR on RTFO Residue	✓	✓	✓	✓
BBR on RTFO Residue		✓	✓	✓
20-hour PAV aging	✓	✓	✓	✓
DSR on 20-hour PAV Residue	✓	✓	✓	✓
BBR on 20-hour PAV Residue	✓	✓	✓	✓
40-hour PAV aging	✓			
DSR on 40-hour PAV Residue	✓			
BBR on 40-hour PAV Residue	✓			

[1]: Binder grading was done for both recovered binder from short-term and long-term aged, compacted specimens

5.3.2 Asphalt Binder Extraction

The extraction process is a fundamental step to calculate percentage asphalt content (AC%) of an AC mixture, conduct required testing on extracted aggregates, and recover and measure the rheological properties of the recovered binder to determine the performance grade. Several methods such as Strategic Highway Research Program (SHRP) Extractions, Abson Method, Ignition Oven Method, Reflux Method, and Centrifuge Method are used for these purposes. In this study, the Centrifuge Method as per AASHTO T164, including the high-speed centrifuge, was used for extraction and asphalt binder was recovered with the Rotary Evaporator Method following ASTM D5404 (AASHTO T164 2014, ASTM D5404 2012). These procedures were used on several material types: RAP stockpile, Field loose mixtures (short-term and long-term aged conditions), and FMFC (short-term and long-term aged conditions) samples. The asphalt binder was extracted using a chemical solution of 85% Toluene and 15% Ethanol. The mixture was first placed into the extraction bowl and the chemical solvent was added into the bowl. As well, to prevent the loss of some fines, a dried filter paper ring was placed around the top edge of the bowl. The centrifuge is then allowed to revolve slowly, and the speed was increased to a maximum of 3600 revolutions per minute until the solvent stop to flow from the drain. The washings and the extract were collected in an appropriate container and repeated until the extracted liquid was not darker than light straw color. Extracted solution was then subjected to a high-speed centrifuging process in order to separate the fines from the solution. Extraction bowl, filter paper, centrifuge tube, and centrifuge screen, were dried to a constant mass at a temperature (230 ± 9) °F prior to mass measurement. After the high-speed centrifuging process, binders were recovered from the collected extract using the rotatory evaporator in accordance with ASTM D5404 (ASTM D5404 2012).

5.3.3 Indirect Tensile (IDT) Strength Test

The Indirect Tensile Strength test was selected to evaluate the cracking and stripping susceptibility of the AC mixtures. During this test, 150 mm diameter cylindrical compacted specimens are loaded across the vertical diametral plane at a rate of 2 inch/min at room temperature. The load displacement curves are obtained to derive two different parameters:

- The indirect tensile strength defined by the peak load from load-displacement curve.
- The cracking tolerance index (CT-Index), which determines the cracking potential of AC mixtures by means of fracture energy.

The IDT Strength Test (ASTM D6931) is adopted by WSDOT with a maximum dry tensile strength set to 175 psi, based on WSDOT Standard Specifications Book 2020 (WSDOT 2020). However, it is worth the effort to implement a more accurate cracking specification based on the CT-Index, and tightly correlate it with the current WSDOT specification for dry strength. This test was chosen because it is the only cracking test that could potentially, practically, be implemented for mix design, QC, and acceptance during production. It was originally designed to examine the cracking resistance of the AC mixtures with recycled materials and has been correlated with current cracking tests. The testing conditions were selected based on the ASTM standard D6931-17, *Standard Test Method for Indirect Tensile (IDT) Strength of Asphalt Mixtures* (ASTM D6931 2017) and listed below:

- Test Temperature: 77°F
- Specimen Size: 150mm diameter x 95mm height
- Displacement rate: 2 inch/min
- Freeze/thaw cycles No: 0 and 1 cycle
- Short term aging on loose mixture: 16-hour at 140°F, as per AASHTO T283 (AASHTO T283 2014).
- Long term aging on compacted mixture: 5 days at 185°F
- Target air void content 7 ± 0.5%

The data obtained after the IDT strength test can be analyzed to find the CT-Index and fracture energy, following Eq. (1)

$$\text{CT-Index} = \frac{t}{62} * \frac{Gf}{|m_{75}|} * \frac{l_{75}}{D} * 10^6 \qquad \text{Eq. (1)}$$

Where,

CT-Index: Cracking Tolerance Index,

G_f: Fracture Energy (The area under load versus displacement Curve) (J/m^2),

l_{75}: displacement at 75 % the peak load after the peak (mm),

$|m_{75}|$: absolute value of the post-peak slope m_{75} (N/m),,

D: diameter of the test specimen (mm), and

t: specimen thickness (mm).

Below are some potential advantages when using the CT-Index versus other cracking tests:

- Simplicity: no cutting, gluing, drilling, or notching is required. This significantly reduces sample preparation time and cost, and operator error involved with testing.
- Practicality: the test requires minimum training for routine operations. Additionally, if the operator knows how to run the IDT strength test, there is little to no training needed at all.
- Efficiency: the test can be completed within 1 min. After compaction and aging take place results can quickly be evaluated.
- Test equipment: the testing equipment involved in breaking the samples costs less than $10,000. This is significantly lower compared to many other cracking tests.
- Repeatability: coefficient of variation (COV) is less than 20 %.
- Sensitivity: sensitive to changes in asphalt binder as well as recycled materials, which is applicable for this study with 15% RAP.

- Correlation to field: limited, but positive correlation with field cracking.

Table 13 summarizes the conditions of the IDT strength test performed for different sample types. However, it should be noted that FMFC (cores) were tested as received and in some cases the air voids were outside the limits set in ASTM D8225-19 (ASTM D8225 2019).

Table 13: IDT Strength Test Performed

Conditions	LMLC			FMLC			FMFC		
	UU	UC	AU	UU	UC	AU	UU	UC	AU
16-hour @ 140°F on loose mix	✓	✓	✓						
1 Freeze/thaw cycle		✓			✓			✓	
5 days,185°F on compacted sample			✓			✓			✓

UU = Unaged Unconditioned
UC = Unaged Conditioned
AU = Aged Unconditioned

5.3.4 HWTT Sample Preparation

The HWTT is used to assess rutting and stripping resistance of AC mixtures in accordance with WSDOT Errata to FOP for AASHTO T324, *Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA)* (WSDOT Errata to AASHTO T324 2018). During this test, a loaded steel wheel tracks over the specimens inside a heated water bath. As load cycles accumulate, a series of Linear Variable Differential Transformer (LVDT) continuously measure the specimens rut depth. WSDOT performed all the HWTT of FMLC specimens compacted to 60 mm height and air voids of 7 ± 1% using a Superpave Gyratory Compactor.

5.3.5 *Dynamic Modulus Test*

The dynamic modulus test is a cyclic axial load test used to determine engineering properties of compacted AC mixtures. During this test, the sample is subjected to compressive sinusoidal stress of multiple frequencies at several temperatures. To compute the dynamic modulus and phase angle of tested specimen, the applied haversine stresses and resulting strains are measured with respect to time. The dynamic modulus can be used for mechanistic empirical pavement design and in mechanistic analyses to evaluate AC mixture resistance to rutting and fatigue cracking. The dynamic modulus test was conducted on the fifteen LMLC mixture combinations following AASHTO Designation: TP132-19, *Standard Method of Test for Determining the Dynamic Modulus for Asphalt Mixtures* ***Using Small Specimens*** *in the Asphalt Mixture Performance Tester (AMPT)* (AASHTO TP132 2019).

Cylindrical specimens that were 38 mm diameter by 110 mm height were fabricated at the target air void level of 7 ± 0.5%, in accordance with AASHTO PP99-19, *Standard Practice for Preparation of Small Cylindrical Performance Test Specimens Using Superpave Gyratory Compactor (SGC) or Field Cores* (AASHTO PP99 2019). To simulate the short-term aging of the mixtures, loose mixes were subjected to 4-hour of conditioning at 275°F, defined by AASHTO R30-02, *Standard Practice for Mixture Conditioning of Hot Mix Asphalt (HMA)* (AASHTO R30 2019). After short term aging, the mixtures were compacted to 150 mm diameter by 180 mm height cylindrical specimens in an SGC. Compacted samples were cored, removing 38 mm diameter cores that were then saw cut to a final height of 110 mm. Gauge points were attached to the specimens maintaining a gauge length of (70 ± 1) mm, and test specimens were conditioned in the test chamber at the selected test temperatures. The mixtures tested at various temperatures and multiple loading frequencies are shown in Table 14. The dynamic modulus and phase angle of all specimens were measured starting at the lowest temperature and highest frequency, with the aim of developing LMLC master curves.

Table 14: Testing Temperature and Loading Frequencies for Different Binders (AASHTO TP132 2019)

PG 58-XX and Softer		PG64-XX and Stiffer	
Temperature, °C	Loading Frequencies, Hz	Temperature, °C	Loading Frequencies, Hz
4	10, 1, 0.1	4	10, 1, 0.1
20	10, 1, 0.1	20	10, 1, 0.1
35	10, 1, 0.1	40	10, 1, 0.1

6 LABORATORY TEST RESULTS AND ANALYSIS

6.1 LMLC Evaluation

6.1.1 Virgin Asphalt Binder

Virgin asphalt binder was graded following AASHTO R29, and assigned subsequently with two different PG, based on these aging conditions (AASHTO R29 2014):

- PG1: After RTFO + 20-hour PAV
- PG2: After RTFO + 40-hour PAV

The performance grading results with MSCR vehicle class from AASHTO M332, of the virgin asphalt binders along with the ΔTc parameter are presented in Table 15, Table 16, and Table 17 below (AASHTO M332 2019). The tables show the true PG; final PG grade; MSCR grade; critical high, intermediate, and low temperatures (at 20-hour and 40-hour PAV aging); and ΔTc values, while noting that the maximum between both stiffness and m-value temperatures was considered as the low critical temperature (Tc). It should be noted that MSCR grading was performed on the supplier high PG, and not based on UNR laboratory high temperature grading. With the aim of getting the low PG, Tc should be subtracted by an additional 10°C, since the BBR test was conducted at 10°C warmer based on time-temperature superposition. The test results denote that the low PG for most virgin binders was controlled by the relaxation parameter m-value after 20-hour PAV. However, after 40-hour PAV aging, all virgin binders became m-value controlled, due to their loss of relaxation properties with additional PAV aging. These observations emphasize the effectiveness of 40-hour PAV aging on identifying m-controlled binders, compared to 20-hour PAV aging. The impact of the additional 20-hour PAV aging (between PG1 and PG2) on ΔTc can be seen from the data shown in Table 15 and Table 16. The results demonstrate significant decrease in ΔTc (more negative) with 40-hour PAV aging. The drop in ΔTc between PG1 and PG2, varied between 1.3°C for PG58H22-S5 and 6.3°C for PG52S28-S3 with an average of 3.4°C for the eleven virgin binders.

Table 15: Virgin Binder PG1 (20-hour PAV)

Binder ID	True Grade	PG Grade	MSCR Grade	Tc (high)	Tc (int)	Tc (low)	ΔTc
PG6428-S1	71.2-29.0	70-28	64V-28	71.2	18.4	-19.0	-0.1
PG6428-S2	70.1-30.0	70-28	64V-28	70.1	17.2	-20.0	-0.8
PG5834-S2	62.1-34.7	58-34	58H-34	62.1	9.7	-24.7	-3.6
PG58H22-S3	66.5-24.5	64-22	58H-22	66.5	22.2	-14.5	-3.0
PG58S28-S3	60.3-28.9	58-28	58S-28	60.3	17.3	-18.9	-3.5
PG52S28-S3	56.3-32.6	52-28	52S-28	56.3	13.1	-22.6	-1.8
PG58H22-S4	66.1-24.2	64-22	58H-22	66.1	23.4	-14.2	-1.9
PG58S28-S4	57.4-31.6	52-28	58S-28	57.4	15.7	-21.7	-0.2
PG58H22-S5	58.7-32.5	58-28	58S-28	58.7	14.5	-22.5	0.3
PG52H28-S5	64.0-32.7	64-28	52E-28	64.0	14.4	-22.7	0.8
PG52S28-S5	57.9-33.7	52-28	52H-28	57.9	14.5	-23.7	-0.3

Table 16: Virgin Binder PG2 (40-hour PAV)

Binder ID	True Grade	PG Grade	Tc (int)	Tc (low)	ΔTc
PG6428-S1	71.2-25.5	70-22	21.8	-15.5	-2.5
PG6428-S2	70.1-27.2	70-22	19.6	-17.2	-2.7
PG5834-S2	62.1-30.0	58-28	12.5	-20.0	-7.7
PG58H22-S3	66.5-17.6	64-16	24.9	-7.6	-8.9
PG58S28-S3	60.3-22.1	58-22	20.3	-12.1	-8.7
PG52S28-S3	56.3-25.9	52-22	15.9	-15.9	-8.1
PG58H22-S4	66.1-20.3	64-16	26.1	-10.3	-4.6
PG58S28-S4	57.4-27.3	52-22	19.2	-17.3	-4.0
PG58H22-S5	58.7-30.6	58-28	17.0	-20.6	-1.0
PG52H28-S5	64.0-30.3	64-28	16.2	-20.3	-1.2
PG52S28-S5	57.9-29.9	52-28	17.6	-19.9	-2.5

Figure 6 shows ΔTc values for all eleven virgin binders after 20-hour and 40-hour PAV aging, along with the cracking warning and cracking limits of -2.5℃ and -5.0℃, respectively, as previously reported state practice for ΔTc (Asphalt Institute, 2019). Interestingly, PG6428-S1, PG58H22-S5, PG52H28-S5, and PG52S28-S5 met the ΔTc cracking warning of -2.5℃ after 20-hour and 40-hour PAV as well. Hence, the observations indicate that these four virgin binders are less susceptible to age-related cracking than the others.

Table 17: Virgin Binder MSCR Results

Virgin Binder ID	%Recovery @ 3.2	Jnr @ 3.2	Jnr_{diff}	MSCR temp	MSCR Grade
PG6428-S1	40.12	0.80	34.7	64	PG64V-28
PG6428-S2	51.96	0.74	25.2	64	PG64V-28
PG5834-S2	31.21	1.25	47.7	58	PG58H-34
PG58H22-S3	4.62	1.02	8.8	58	PG58H-22
PG58S28-S3	0.63	3.08	12.0	58	PG58S-28
PG52S28-S3	1.40	2.05	11.3	52	PG52S-28
PG58H22-S4	2.18	1.38	7.1	58	PG58H-22
PG58S28-S4	-0.77	4.48	7.1	58	PG58S-28
PG58H22-S5	0.28	2.97	8.9	58	PG58S-28
PG52H28-S5	42.75	0.34	10.8	52	PG52E-28
PG52S28-S5	1.59	1.87	7.5	52	PG52H-28

All the evaluated virgin binders met ΔTc cracking limit of -5.0°C after 20-hour PAV, whereas after 40-hour PAV, the binders PG5834-S2, PG58H22-S3, PG58S28-S3, and PG52S28-S3 exceeded this cracking limit. Supplier source had a significant effect within same binder grade, such as PG58H-22, which had a ΔTc of -9.0°C and -3.0°C from supplier 3, compared to -1°C and +0.3°C from supplier 5. Lastly, Figure 7 presents the relation between PG1-ΔTc and PG2-ΔTc as a linear relation.

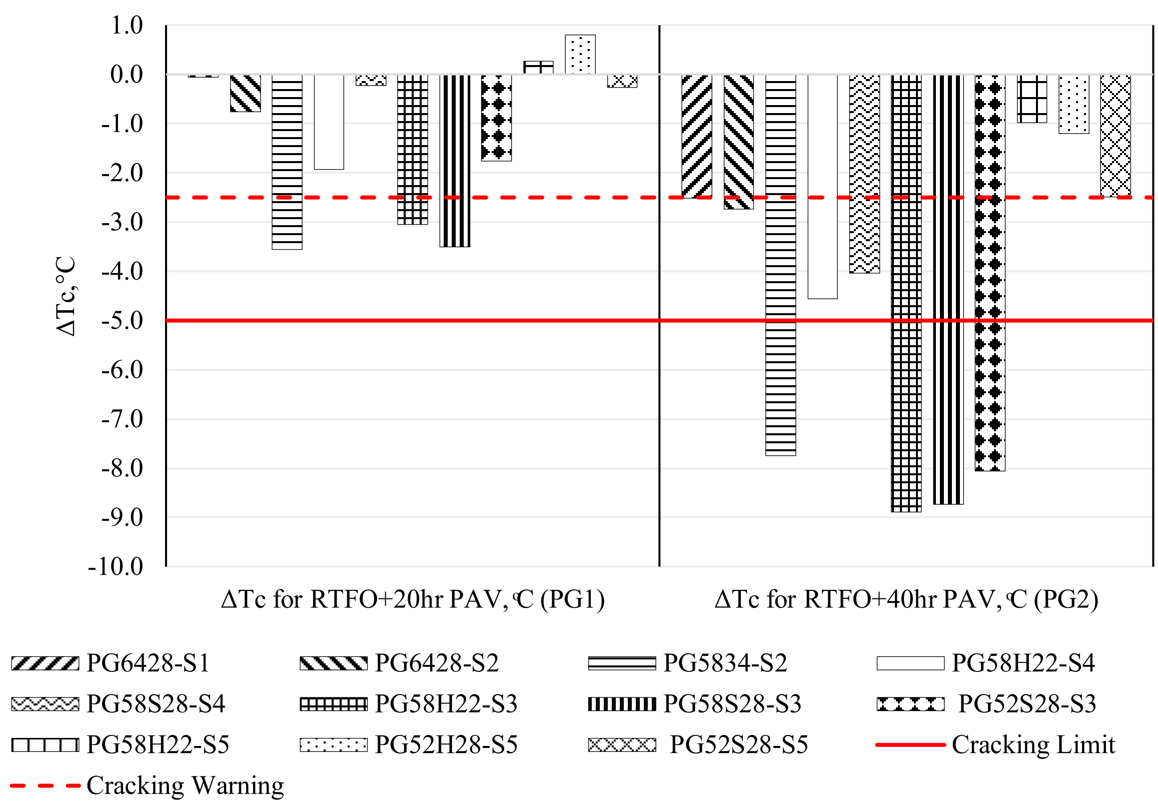


Figure 6: ΔTc Values for Eleven Virgin Binders

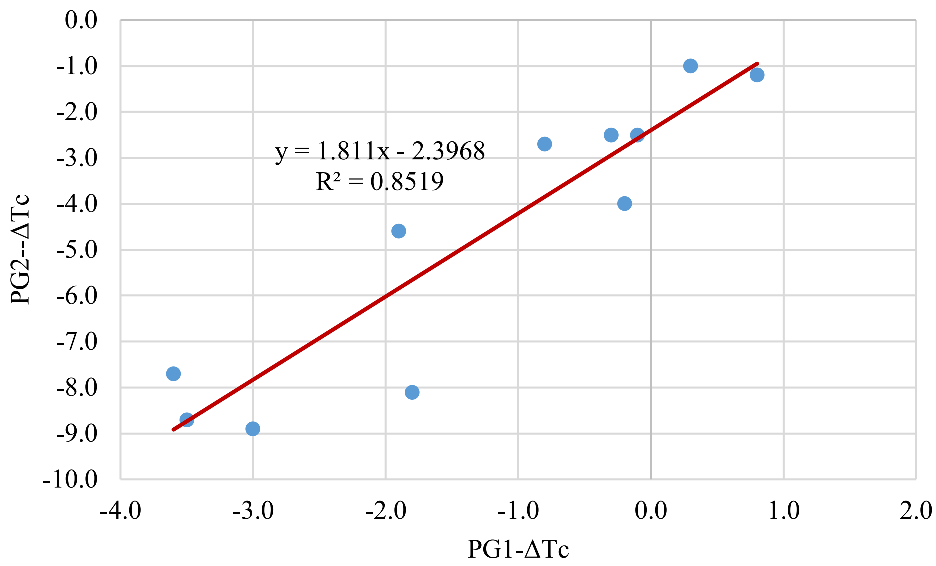


Figure 7: Relation between PG1-ΔTc and PG2-ΔTc

6.1.2 Recovered RAP Binder

The rheological properties of recovered binder from RAP samples were obtained. The extraction and recovery tests were performed in accordance with AASHTO T164 and ASTM D5404, respectively (AASHTO T164 2014, ASTM D5404 2012). Each recovered RAP binder was then subjected to two different PG after the following aging conditions, including PG1 based on AASHTO M323 (AASHTO M323 2017):

- PG1: After RTFO
- PG2: After RTFO + 20-hour PAV

The recovered RAP binder was tested using DSR at high temperature as if it was original, unaged binder. The remaining RAP binder was then aged in the RTFO and tested with the DSR and BBR to develop PG1. The remaining RTFO-aged RAP binder was subjected to 20-hour PAV aging, then tested through DSR and BBR again to develop PG2. These aging protocols were adopted considering that RTFO of recovered RAP binder may be equivalent to 20-hour PAV aging of virgin binder, whereas 20-hour PAV aging for RAP recovered binder may be equivalent to 40-hour of PAV aging of the virgin binder.

Table 18 and Table 19 show the binder grading results under both conditions. The results showed that RAP II was the stiffest among the RAP sources, while RAP III got the softest grade. The values obtained for ΔTc parameter are plotted in Figure 8, and indicate that all RAP samples exceeded the cracking warning of -2.5°C for PG1, and PG2. However, the cracking limit of -5.0°C was met solely for PG1 (after RTFO) of RAP I, II, and III, and exceeded for PG2.

Table 18: RAP Binder PG1 (RTFO)

ID	True Grade	PG Grade	Tc (high)	Tc (int)	Tc (low)	ΔTc
RAP I (9145)	93.6-13.2	88-10	93.6	34.8	-3.2	-3.7
RAP II (9262/9229)	105.8-6.4	100-6	105.8	42.6	3.6	-4.5
RAP III (9231)	87.9-18.7	82 -16	87.9	28.9	-8.7	-3.2

Table 19: RAP Binder PG2 (RTFO+20-hour PAV)

ID	True Grade	PG Grade	Tc (int)	Tc (low)	ΔTc
RAP I (9145)	93.6-11.3	88-10	37.3	-1.3	-5.2
RAP II (9262/9229)	105.8-1.8	100-0	46.2	8.2	-7.7
RAP III (9231)	87.9-14.5	82-10	32.2	-4.5	-6.4

The ΔTc variation from PG1 to PG2 aging conditions was equivalent to 41%, 71%, and 100% for RAP I, RAP II, and RAP III, respectively. In other words, the additional 20-hour PAV aging (PG1 versus PG2) lowered the RAP ΔTc between 1.5°C and 3.2°C, with an average of 2.6°C. Pavement aging follows a power function starting at a high rate and reaches an approximate plateau at the end of pavement life. This is why the ΔTc reduction between PG1 and PG2 for the virgin binder was slightly more significant than for the extracted RAP binder.

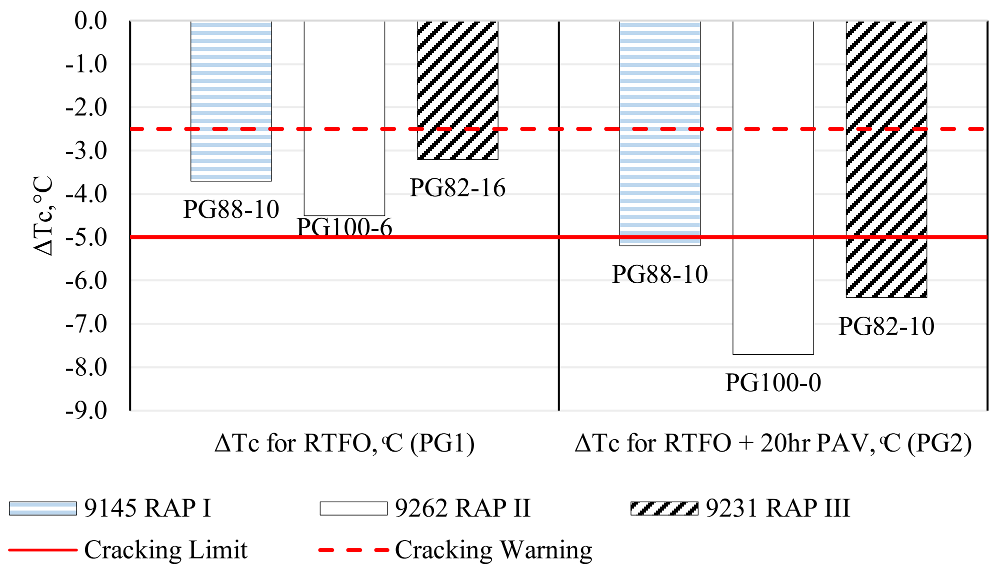


Figure 8: ΔTc Values for RAP Binders

6.1.3 Blending Charts

The rheological properties including critical temperatures (at high, intermediate, and low) with ΔTc parameter of the virgin binder graded in section 6.1.1, along with recovered RAP binder graded in section 6.1.2, were used to develop multiple blending charts. The true critical temperatures of the virgin and RAP binder were used in the blending charts, rather than the final binder grade. The critical temperatures and ΔTc parameter of the blended binder, were determined per AASHTO M323 using the following relationships (AASHTO M323 2017):

$$T_{Blend} = T_{Virgin} * (1 - \%RBR) + T_{RAP} * \%RBR \qquad \text{Eq. (2)}$$

$$\Delta\text{Tc}_{Blend} = \Delta\text{Tc}_{Virgin} * (1 - \%RBR) + \Delta\text{Tc}_{RAP} * \%RBR \qquad \text{Eq. (3)}$$

Where,

T_{Blend}: Critical temperature of blended asphalt binder,

ΔTc_{Blend} : ΔTc of blended asphalt binder,

ΔTc_{Virgin} : ΔTc of virgin binder,

ΔTc_{RAP} : ΔTc of RAP binder,

$\%RBR$: binder replacement ratio,

T_{Virgin}: Critical temperature of virgin binder, and

T_{RAP}: Critical temperature of RAP binder.

The different combinations of blending charts were presented in APPENDIX F: LABORATORY SAMPLES NOMENCLATURE . They were developed three times for each combination of binders, under the following aging conditions:

- BPG1: blended PG based on PG1 of virgin binder (RTFO+20-hour PAV) with PG1 of RAP binder (RTFO)
- BPG2: blended PG based on PG1 of virgin binder (RTFO+20-hour PAV) with PG2 of RAP binder (RTFO+20-hour PAV)
- BPG3: blended PG based on PG2 of virgin binder (RTFO+40-hour PAV) with PG2 of RAP binder (RTFO+20-hour PAV)

The true and final PG, along with ΔTc parameter are presented in Table 20 and Table 21, for different combinations of virgin and recycled asphalt binders, relative to BPG1, BPG2, and BPG3. The comparisons of ΔTc for some binders between BPG1 versus BPG2, and BPG1 versus BPG3 are shown in Figure 9 and Figure 10, respectively. The results indicate that ΔTc was reduced from BPG1 to BPG2 conditions on average by 0.5°C with 20% RAP, 0.6°C with 25% RAP, and 1.0°C with a high RAP amount (36% to 40%). Similarly, ΔTc was reduced from BPG1 to BPG3 conditioning on average by 3.6°C with 20% RAP, 3.5°C with 25% RAP, and 3.7°C with a high RAP amount (36% to 40%). The decrease in ΔTc was expected to be more pronounced with more extensive aging conditions and higher RAP amount. However, when comparing ΔTc between BPG1 and BPG3 after extended aging, this drop reached approximately a constant plateau of 3.5°C at different RAP amounts.

Table 20: Blending Charts Results (BPG1 and BPG2)

Nomenclature	True Grade		PG Grade		ΔTc, °C	
	BPG1	BPG2	BPG1	BPG2	BPG1	BPG2
9145-20-6428-S1	74.7-26.5	74.7-26.2	70-22	70-22	-0.6	-0.9
9145-25-6428-S1	75.6-25.9	75.6-25.5	70-22	70-22	-0.8	-1.1
9145-20-6428-S2	73.9-27.3	73.9-27.0	70-22	70-22	-1.2	-1.5
9145-25-6428-S2	74.8-26.6	74.8-26.2	70-22	70-22	-1.4	-1.7
9145-20-5834-S2	67.1-31.3	67.1-31.0	64-28	64-28	-3.6	-3.8
9145-25-5834-S2	68.4-30.4	68.4-30.0	64-28	64-28	-3.6	-3.9
9262-20-58H22-S3	73.0-21.5	73.0-20.8	70-16	70-16	-3.3	-3.8
9262-25-58H22-S3	74.6-20.8	74.6-19.8	70-16	70-16	-3.4	-4.0
9262-36-58H22-S3	78.1-19.2	78.1-17.8	76-16	76-16	-3.5	-4.5
9262-20-58S28-S3	67.7-25.2	67.7-24.4	64-22	64-22	-3.7	-4.2
9262-25-58S28-S3	69.6-24.3	69.6-23.3	64-22	64-22	-3.7	-4.4
9262-36-58S28-S3	73.7-22.2	73.7-20.8	70-22	70-22	-3.8	-4.8
9262-20-58H22-S4	72.6-21.3	72.6-20.5	70-16	70-16	-2.4	-2.9
9262-25-58H22-S4	74.2-20.5	74.2-19.6	70-16	70-16	-2.5	-3.1
9262-36-58H22-S4	77.8-18.9	77.8-17.5	76-16	76-16	-2.7	-3.7
9262-20-58S28-S4	65.3-27.6	65.3-26.7	64-22	64-22	-0.9	-1.5
9262-25-58S28-S4	67.3-26.5	67.3-25.5	64-22	64-22	-1.1	-1.8
9262-36-58S28-S4	71.7-24.2	71.7-22.7	70-22	70-22	-1.5	-2.5
9231-20-58H22-S3	69.9-23.6	69.9-22.9	64-22	64-22	-3.1	-3.6
9231-25-58H22-S3	70.7-23.4	70.7-22.5	70-22	70-22	-3.1	-3.7
9231-40-58H22-S3	73.2-22.7	73.2-21.4	70-22	70-16	-3.1	-4.1
9231-20-52S28-S3	61.2-30.4	61.2-29.7	58-28	58-28	-2.0	-2.5
9231-25-52S28-S3	62.5-29.9	62.5-29.0	58-28	58-28	-2.0	-2.7
9231-40-52S28-S3	66.2-28.2	66.2-26.9	64-28	64-22	-2.2	-3.2
9231-20-58H22-S5	63.3-30.4	63.3-29.7	58-28	58-28	-0.3	-0.8
9231-25-58H22-S5	64.4-29.8	64.4-29.0	64-28	64-28	-0.4	-1.0
9231-40-58H22-S5	67.9-28.2	67.9-26.9	64-28	64-22	-0.8	-1.8
9231-20-52H28-S5	67.7-30.5	67.7-29.8	64-28	64-28	0.2	-0.3
9231-25-52H28-S5	68.7-29.9	68.7-29.1	64-28	64-28	0.0	-0.6
9231-40-52H28-S5	71.5-28.3	71.5-26.9	70-28	70-22	-0.5	-1.5
9231-20-52S28-S5	62.6-31.4	62.6-30.7	58-28	58-28	-0.7	-1.2
9231-25-52S28-S5	63.7-30.8	63.7-30.0	58-28	58-28	-0.8	-1.5
9231-40-52S28-S5	67.3-29.0	67.3-27.7	64-28	64-22	-1.2	-2.2

BPG1: PG1 of virgin binder (after RTFO+20-hour PAV) + PG1 of RAP (after RTFO)
BPG2: PG1 of virgin binder (after RTFO+20-hour PAV) + PG2 of RAP (after RTFO+ 20-hour PAV)

Table 21: Blending Charts Results (BPG1 and BPG3)

Nomenclature	True Grade		PG Grade		ΔTc, °C	
	BPG1	BPG3	BPG1	BPG3	BPG1	BPG3
9145-20-6428-S1	74.7-26.5	74.7-23.2	70-22	70-22	-0.6	-2.9
9145-25-6428-S1	75.6-25.9	75.6-22.7	70-22	70-22	-0.8	-3.1
9145-20-6428-S2	73.9-27.3	73.9-24.6	70-22	70-22	-1.2	-3.1
9145-25-6428-S2	74.8-26.6	74.8-24.0	70-22	70-22	-1.4	-3.2
9145-20-5834-S2	67.1-31.3	67.1-27.0	64-28	64-22	-3.6	-7.3
9145-25-5834-S2	68.4-30.4	68.4-26.2	64-28	64-22	-3.6	-7.2
9262-20-58H22-S3	73.0-21.5	73.0-15.0	70-16	70-10	-3.3	-8.7
9262-25-58H22-S3	74.6-20.8	74.6-14.3	70-16	70-10	-3.4	-8.7
9262-36-58H22-S3	78.1-19.2	78.1-12.9	76-16	76-10	-3.5	-8.6
9262-20-58S28-S3	67.7-25.2	67.7-18.8	64-22	64-16	-3.7	-8.6
9262-25-58S28-S3	69.6-24.3	69.6-17.9	64-22	64-16	-3.7	-8.5
9262-36-58S28-S3	73.7-22.2	73.7-16.1	70-22	70-16	-3.8	-8.4
9262-20-58H22-S4	72.6-21.3	72.6-17.2	70-16	70-16	-2.4	-5.1
9262-25-58H22-S4	74.2-20.5	74.2-16.5	70-16	70-16	-2.5	-5.2
9262-36-58H22-S4	77.8-18.9	77.8-14.8	76-16	76-10	-2.7	-5.5
9262-20-58S28-S4	65.3-27.6	65.3-23.1	64-22	64-22	-0.9	-4.7
9262-25-58S28-S4	67.3-26.5	67.3-22.1	64-22	64-22	-1.1	-4.8
9262-36-58S28-S4	71.7-24.2	71.7-19.7	70-22	70-16	-1.5	-5.1
9231-20-58H22-S3	69.9-23.6	69.9-17.1	64-22	64-16	-3.1	-8.5
9231-25-58H22-S3	70.7-23.4	70.7-17.0	70-22	70-16	-3.1	-8.4
9231-40-58H22-S3	73.2-22.7	73.2-16.6	70-22	70-16	-3.1	-8.1
9231-20-52S28-S3	61.2-30.4	61.2-24.1	58-28	58-22	-2.0	-7.8
9231-25-52S28-S3	62.5-29.9	62.5-23.6	58-28	58-22	-2.0	-7.7
9231-40-52S28-S3	66.2-28.2	66.2-22.3	64-28	64-22	-2.2	-7.5
9231-20-58H22-S5	63.3-30.4	63.3-28.1	58-28	58-28	-0.3	-1.8
9231-25-58H22-S5	64.4-29.8	64.4-27.5	64-28	64-22	-0.4	-2.0
9231-40-58H22-S5	67.9-28.2	67.9-25.6	64-28	64-22	-0.8	-2.7
9231-20-52H28-S5	67.7-30.5	67.7-27.8	64-28	64-22	0.2	-2.0
9231-25-52H28-S5	68.7-29.9	68.7-27.2	64-28	64-22	0.0	-2.2
9231-40-52H28-S5	71.5-28.3	71.5-25.3	70-28	70-22	-0.5	-2.8
9231-20-52S28-S5	62.6-31.4	62.6-27.5	58-28	58-22	-0.7	-3.1
9231-25-52S28-S5	63.7-30.8	63.7-26.9	58-28	58-22	-0.8	-3.3
9231-40-52S28-S5	67.3-29.0	67.3-25.0	64-28	64-22	-1.2	-3.7

BPG1: PG1 of virgin binder (after RTFO+20-hour PAV) + PG1 of RAP (after RTFO)
BPG3: PG2 of virgin binder (after RTFO+40-hour PAV) + PG2 of RAP (after RTFO+ 20-hour PAV)

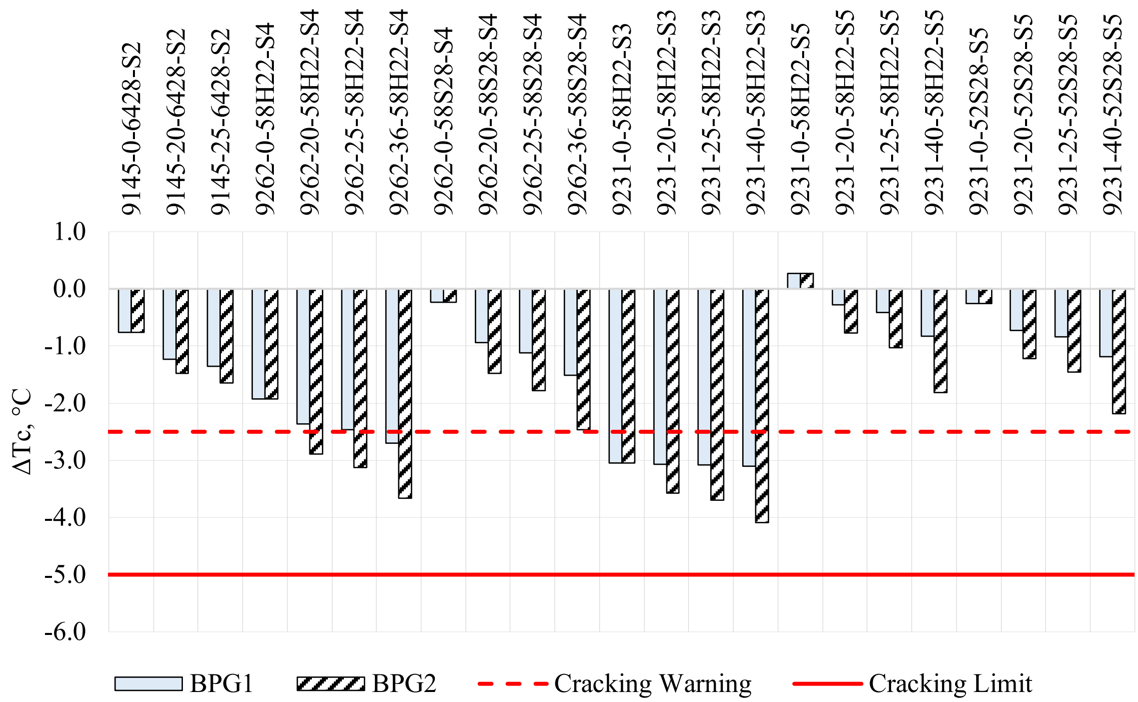


Figure 9: Comparative Chart of BPG1 and BPG2

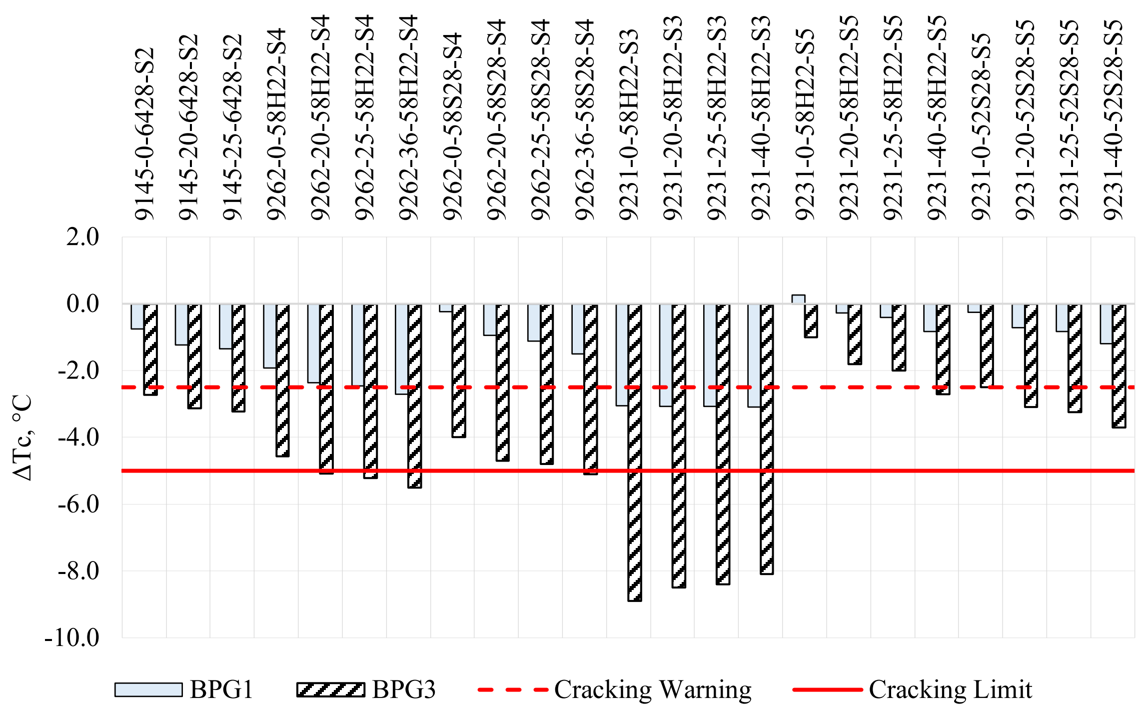


Figure 10: Comparative Chart of BPG1 and BPG3

The effect of RAP dosage on ΔTc within the same binder supplier grade, is plotted in Figure 11, Figure 12, and Figure 13 for BPG1, BPG2, and BPG3, respectively. Figure 11 focuses on the impact of adding different RAP amounts on ΔTc, and suggest the following results for the binders analyzed as per BPG1:

- 20% RAP reduced ΔTc on average by 0.5℃ compared to 0% RAP
- 25% RAP reduced ΔTc on average by 0.6℃ compared to 0% RAP, and 0.1℃ compared to 20% RAP
- High RAP amount (36% and 40%) reduced ΔTc on average by 0.9℃ compared to 0% RAP

Figure 12 focuses on the impact of adding different RAP amounts on ΔTc, and suggests the following for the binders analyzed as per BPG2:

- 20% RAP reduced ΔTc on average by 1℃ compared to 0% RAP
- 25% RAP reduced ΔTc on average by 1.2℃ compared to 0% RAP, and 0.2℃ compared to 20% RAP
- High RAP amount (36% and 40%) reduced ΔTc on average by 1.9℃ compared to 0% RAP

Figure 13 focuses on the impact of different RAP amounts on ΔTc, and suggests the following for the binders analyzed as per BPG3:

- 20% RAP reduced ΔTc on average by 0.2℃ compared to 0% RAP
- 25% RAP reduced ΔTc on average by 0.3℃ compared to 0% RAP, and 0.1℃ compared to 20% RAP

- High RAP amount (36% and 40%) reduced ΔTc on average by 0.5℃ compared to 0% RAP

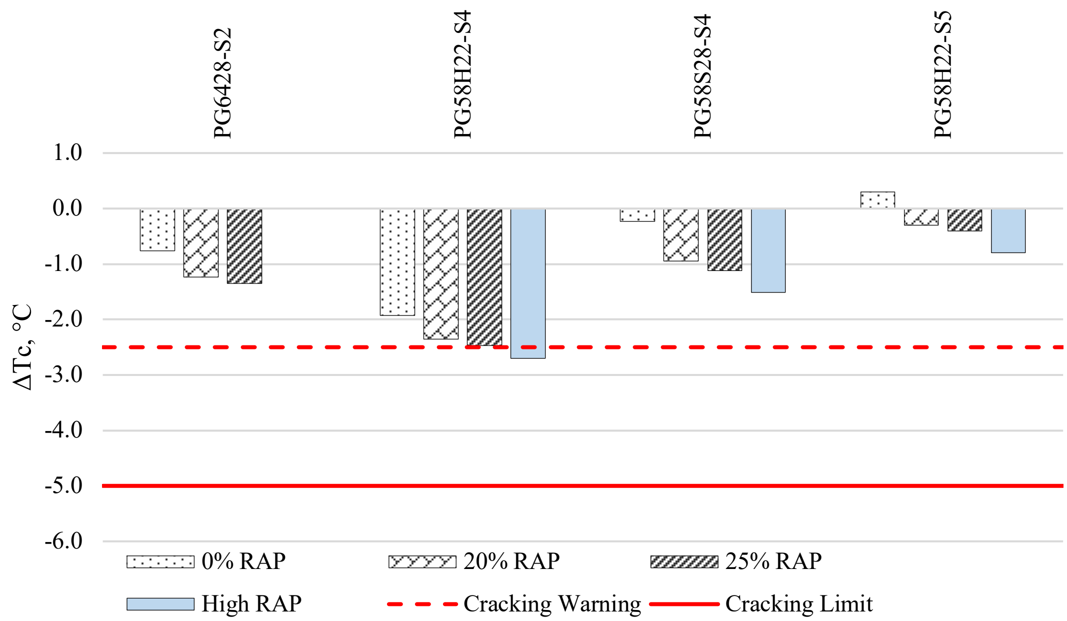


Figure 11: Effect of RAP on ΔTc (BPG1)

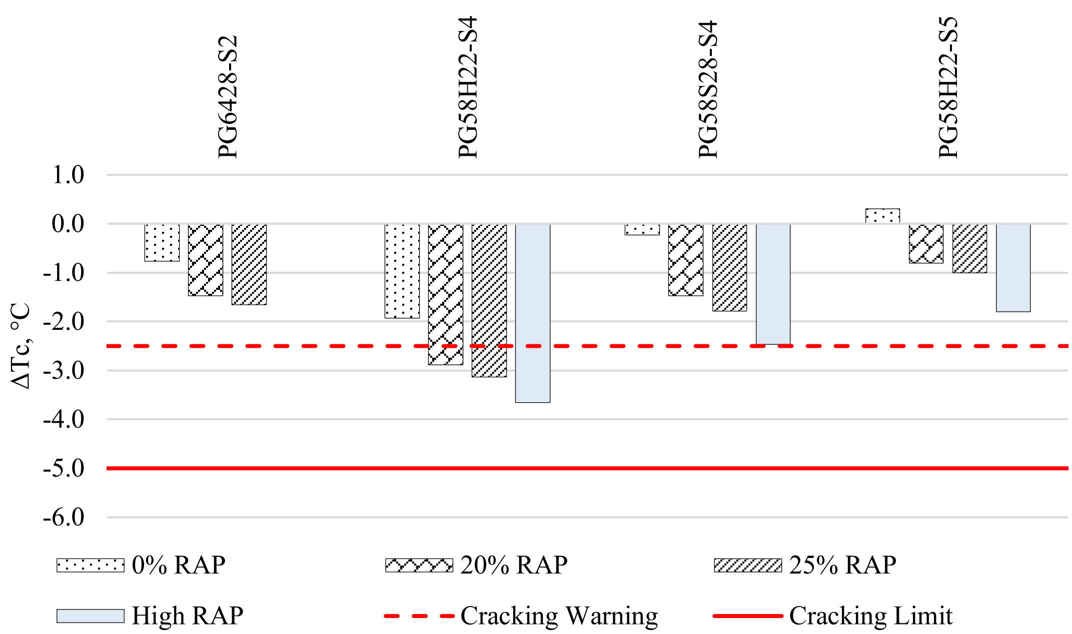


Figure 12: Effect of RAP on ΔTc (BPG2)

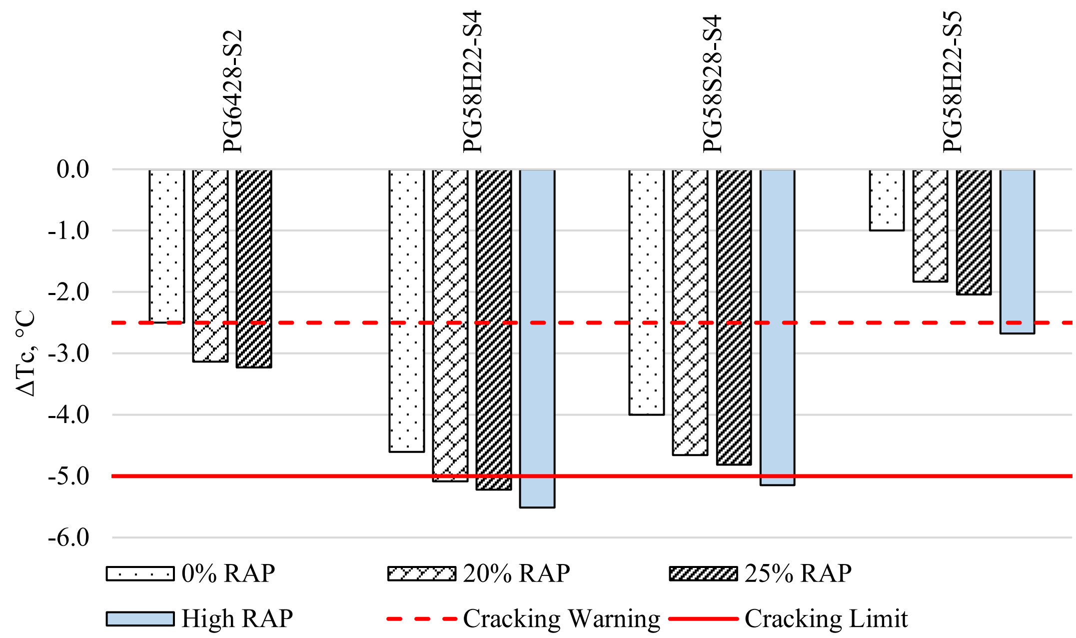


Figure 13: Effect of RAP on ΔTc (BPG3)

6.1.4 Physically Blended Binders and Comparison

The virgin asphalt binders were blended with the specified weight of recovered RAP binder, and RA if any, in order to prepare 11 different combinations of physically blended asphalt binders. These combinations from different contracts were previously presented in Table 9, while noting that RAP I, II, and III were used for contracts 9145, 9262/9229, and 9231, respectively. The PG and MSCR test were conducted on the physically blended binders and compared to the blending chart results in terms of true PG, final PG, and ΔTc. Two different final PG, denoted Physical PG1 and Physical PG2 were obtained for the physically blended binders, following the two aging protocols described in Figure 14.

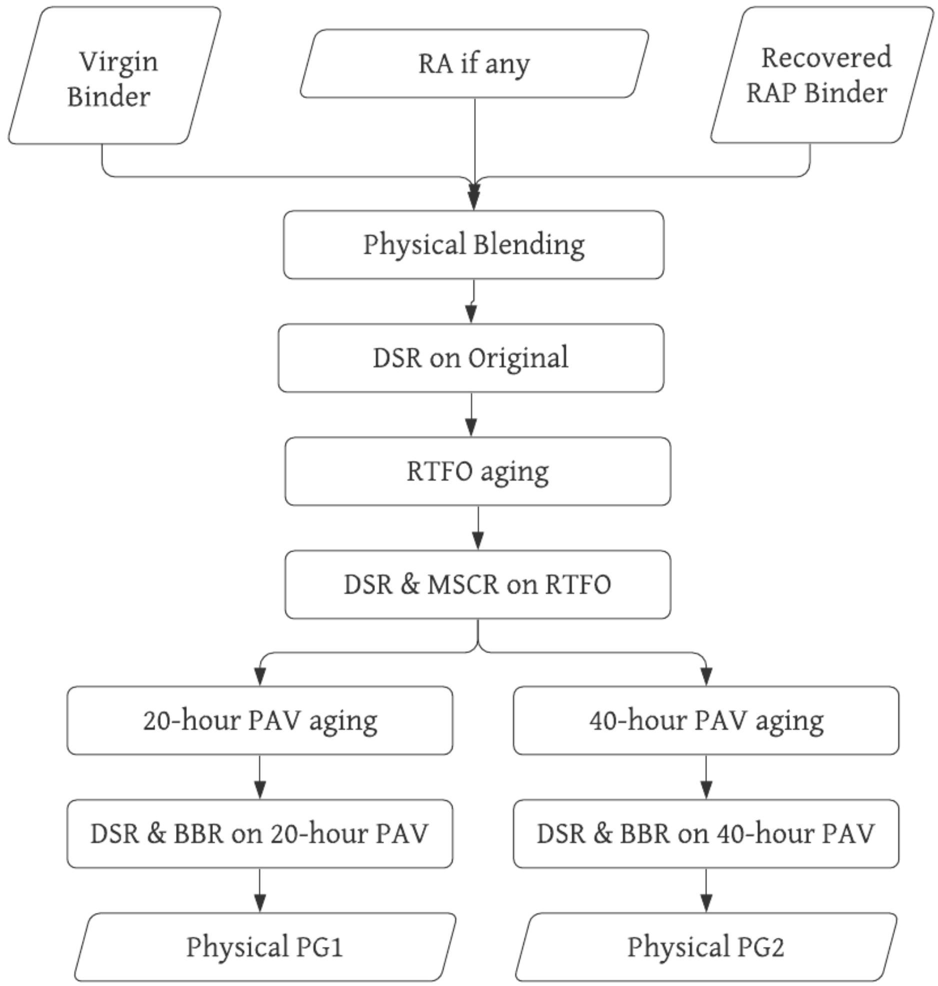


Figure 14: Aging Protocols for Physical PG

The results in Table 22, Table 23, and Figure 15 underline on the impact of extended 20-hour PAV aging (between Physical PG1 and Physical PG2) on the true PG, final PG, and ΔTc. This additional 20-hour PAV induced a decrease of ΔTc between 1.2℃ and 5.9℃ with an average of 3.2℃ for the 11 physically blended binders. When analyzing the influence of RA on the blended binder properties, the physical PG1 results of 9262-36-58S28-S4 and 9262-36-58S28-S4+RA imply a minor improvement after 20-hour PAV. However, it was found that the RA significantly influenced the critical low temperature and ΔTc parameter after 40-hour PAV aging for physical blending PG2. It can be concluded that the RA improved the relaxation property of the long-term aged RAP binder blended with virgin binder and reduced the propensity for cracking.

Table 22: Physical Blending Results

Blended Binder ID	True PG		Final PG	
	Physical PG1	Physical PG2	Physical PG1	Physical PG2
9145-20-5834-S2	66.4-29.0	66.4-24.1	64-28	64-22
9145-25-5834-S2	66.6-27.7	66.6-22.9	64-22	64-22
9145-20-6428-S2	72.4-26.9	70.1-22.6	70-22	70-22
9262-20-58H22-S4	73.0-20.7	73.0-17.0	70-16	70-16
9262-25-58H22-S4	74.6-19.4	74.6-15.5	70-16	70-10
9262-36-58S28-S4	72.3-23.4	72.3-18.5	70-22	70-16
9262-36-58S28-S4+RA	71.7-25.5	71.7-23.1	70-22	70-22
9231-20-58H22-S5	63.4-30.3	63.4-28.2	58-28	58-28
9231-25-58H22-S5	64.6-29.7	64.6-27.3	64-28	64-22
9231-40-52S28-S5	67.6-28.0	67.6-25.1	64-28	64-22
9231-40-52S28-S3	66.4-27.5	66.4-20.8	64-22	64-16

Physical PG1: after RTFO+20-hour PAV
Physical PG2: after RTFO+40-hour PAV

Table 23: Physical Blending ΔTc

Blended Binder ID	ΔTc, °C	
	Physical PG1	Physical PG2
9145-20-5834-S2	-6.0	-10.1
9145-25-5834-S2	-6.6	-11.0
9145-20-6428-S2	-2.7	-5.3
9262-20-58H22-S4	-3.1	-6.8
9262-25-58H22-S4	-3.7	-7.4
9262-36-58S28-S4	-2.7	-6.8
9262-36-58S28-S4+RA	-2.2	-3.8
9231-20-58H22-S5	-1.1	-2.3
9231-25-58H22-S5	-1.0	-2.6
9231-40-52S28-S5	-1.9	-4.0
9231-40-52S28-S3	-4.1	-10.0

Physical PG1: after RTFO+20-hour PAV
Physical PG2: after RTFO+40-hour PAV

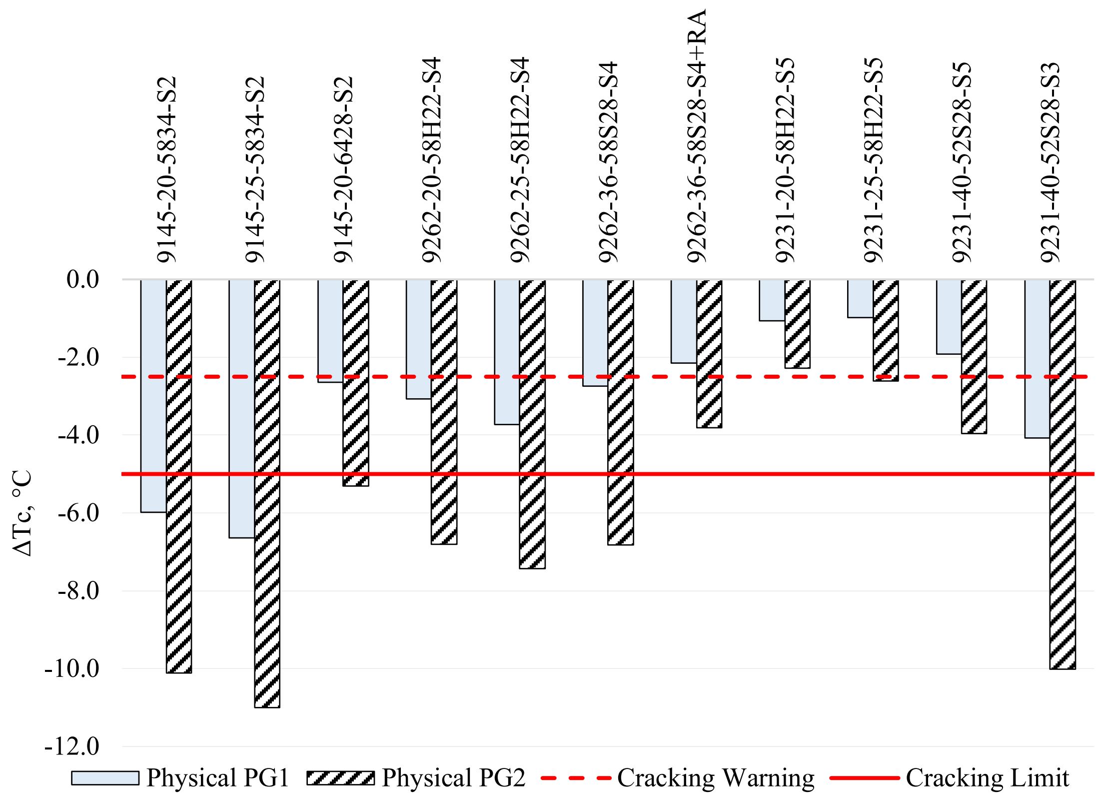


Figure 15: Effect of Extended 20-hour PAV on ΔTc of Physically Blended Binders

According to the same aging process for virgin and RAP binder, Physical PG1 was compared to BPG2, where the virgin and RAP binder were subjected to RTFO and 20-hour PAV in both cases. The results are shown in Table 24 and Figure 16, with the aim of identifying any differences that existed between blending charts and physically blended binder results. A comparison between the ΔTc of the blending charts versus physically blended binders, revealed that the blending charts underestimated the ΔTc (in absolute value) for 9 out of the 10 evaluated binders, the difference ranged between -2.7 °C and +0.3°C, with an average of -0.9°C. With respect to final PG in Table 24, the difference in grade between physical blending and blending charts was not as pronounced as ΔTc values.

Table 24: Comparison between Physical PG1 and BPG2

Blended Binder ID	True PG		Final PG		ΔTc, °C	
	Physical PG1	BPG2	Physical PG1	BPG2	Physical PG1	BPG2
9145-20-5834-S2	66.4-29.0	67.1-31.0	64-28	64-28	-6.0	-3.8
9145-25-5834-S2	66.6-27.7	68.4-30.0	64-22	64-28	-6.6	-3.9
9145-20-6428-S2	72.4-26.9	73.9-27.0	70-22	70-22	-2.7	-1.5
9262-20-58H22-S4	73.0-20.7	72.6-20.5	70-16	70-16	-3.1	-2.9
9262-25-58H22-S4	74.6-19.4	74.2-19.6	70-16	70-16	-3.7	-3.1
9262-36-58S28-S4	72.3-23.4	71.7-22.7	70-22	70-22	-2.7	-2.5
9231-20-58H22-S5	63.4-30.3	63.3-29.7	58-28	58-28	-1.1	-0.8
9231-25-58H22-S5	64.6-29.7	64.4-29.0	64-28	64-28	-1.5	-1.0
9231-40-52S28-S5	67.6-28.0	67.3-27.7	64-28	64-22	-1.9	-2.2
9231-40-52S28-S3	66.4-27.5	66.2-26.9	64-22	64-22	-4.1	-3.2

Physical PG1: after RTFO+20-hour PAV
BPG2: PG1 of virgin binder (after RTFO+20-hour PAV) + PG2 of RAP (after RTFO+ 20-hour PAV)

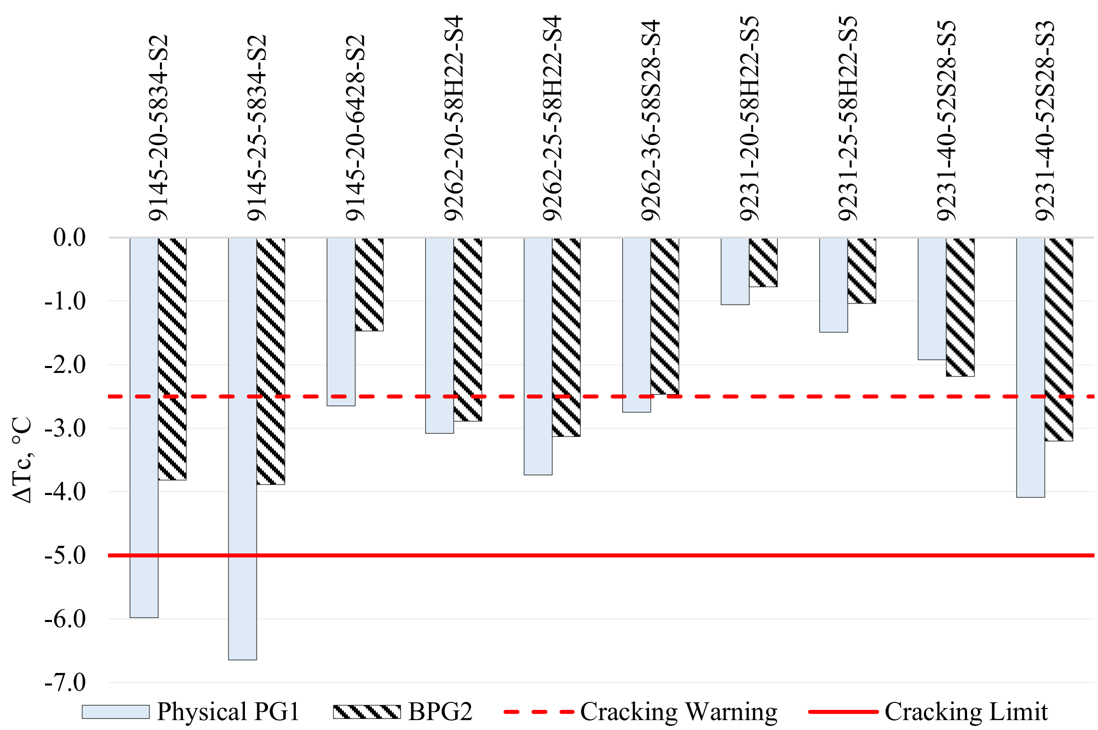


Figure 16: ΔTc Comparison between Physical PG1 and BPG2

6.1.5 Verification of Aggregate Gradations

When preparing LMLC specimens with different RAP percentages, summarized in Table 6, the aggregate blends were always matched to the mix design blend gradations in the WSDOT JMFs in APPENDIX C: CONTRACT 9145 MIX DESIGN, APPENDIX D: CONTRACT 9262 / 9229 MIX DESIGN, and APPENDIX E: CONTRACT 9231 MIX DESIGN. The 0.45 power charts presented in Figure 17, Figure 18, Figure 19 for contracts 9145, 9262/9229, and 9231, respectively, show the verification mix design and LMLC gradations at different RAP doses. The control points refer to WSDOT standard specifications for class ½" relative to contract 9145, and class 3/8" for contracts 9262/9229 and 9231 (WSDOT 2020). Recall that the original WSDOT mix designs include 0%, 36%, and 40% RAP, for contracts 9145, 9262/9229, and 9231, respectively.

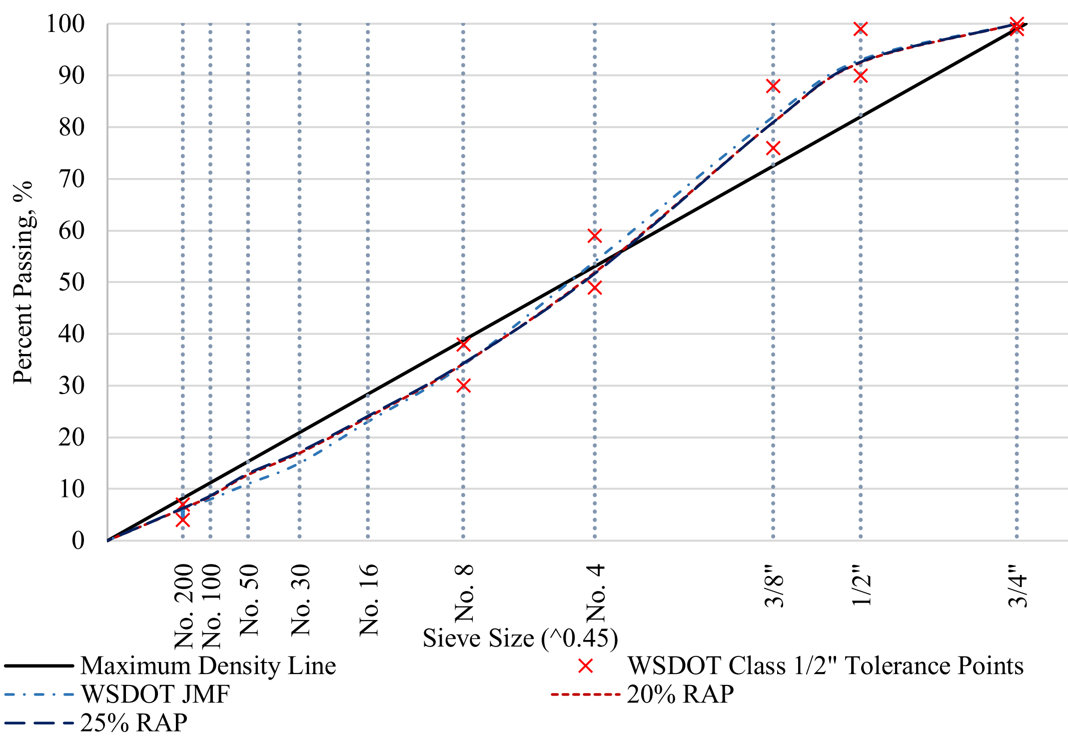


Figure 17: 0.45 Power Chart of 9145 Contract

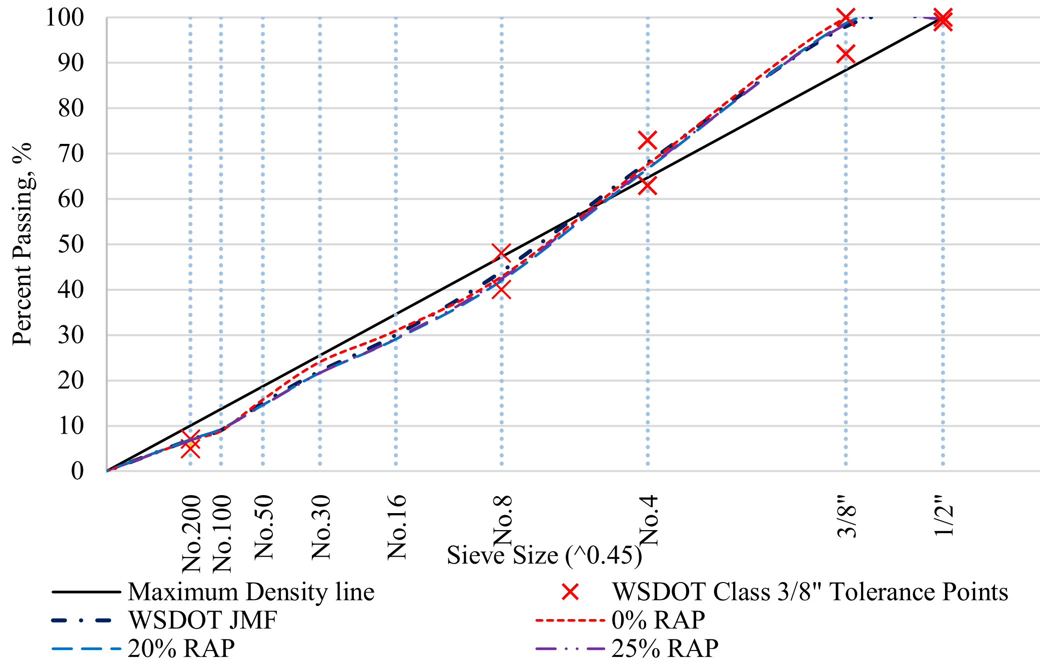


Figure 18: 0.45 Power Chart of 9262/9229 Contract

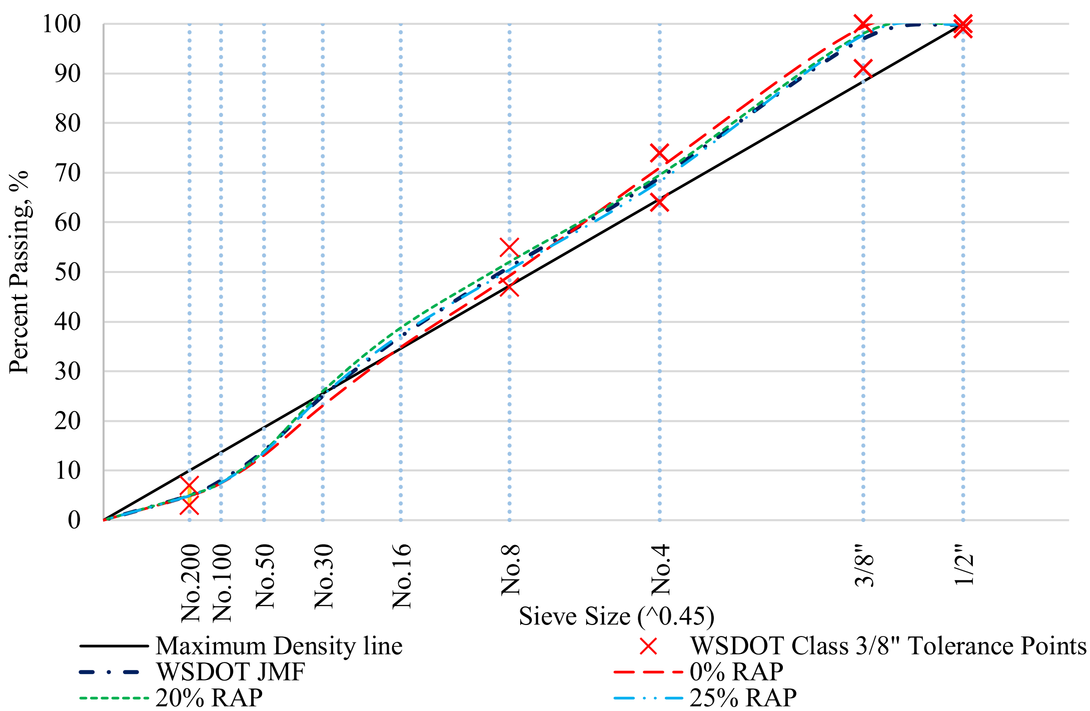


Figure 19: 0.45 Power Chart of 9231 Contract

Following the centrifuge extraction of RAP samples incorporated in LMLC specimens, AC% and sieve analysis was verified with WSDOT JMF properties shown in Table 25.

Table 25: RAP Stockpiles Properties

RAP Properties	RAP I (Contract 9145)		RAP II (Contract 9262/9229)		RAP III (Contract 9231)	
	UNR	WSDOT JMF	UNR	WSDOT JMF	UNR	WSDOT JMF
AC%	4.1	-	4.7	-	4.4	-
3/4"	100.0	-	100.0	100.0	100.0	100.0
1/2"	95.4	-	99.1	99.3	100.0	99.8
3/8"	87.2	-	95.3	94.2	94.3	93.2
#4	64.2	-	68.4	70.1	66.9	68.9
#8	47.8	-	47.7	50.6	49.8	51.2
#16	35.8	-	34.3	36.0	38.1	38.7
#30	26.2	-	25.7	26.4	28.6	28.8
#50	19.1	-	18.9	18.6	19.2	18.7
#100	14.1	-	13.8	13.5	12.2	12.7
#200	10.7	-	11.1	10.1	9.1	9.2

6.1.6 IDT Strength Testing

IDT strength performance test was performed on LMLC specimens to obtain the following cracking test parameters:

- Indirect tensile strength
- Tensile Strength Ratio (TSR) as the ratio of wet and dry tensile strengths
- CT-Index
- Fracture energy

The test methods followed were ASTM D6931 for IDT strength, AASHTO T283 for TSR, and ASTM D8225 for CT-Index. All specimens were prepared to 7 ± 0.5% air voids and tested at 77°F (ASTM D6931 2017, AASHTO T283 2014, ASTM D8225 2019). The aforementioned parameters were measured or calculated for each of the 15 mixtures, on 3 replicates of LMLC samples, under each of the following conditions:

- UU (Unaged Unconditioned) :16-hour conditioning at 140°F on loose mix without any freeze-thaw cycle
- UC (Unaged Conditioned): 16-hour conditioning at 140°F on loose mix with one freeze-thaw cycle
- AU (long-term Aged Unconditioned): 16-hour conditioning at 140°F on loose mix followed by 5 days at 185°F on compacted samples without any freeze thaw cycle

The fifteen mixtures comprised many variations of RAP dose and asphalt binder grades for each of the project mixture design. Table 26 summarizes IDT strength and TSR data, while Table 27 summarizes CT-Index, and fracture energy results.

Table 26: LMLC IDT Strength Results and TSR

Mix Nomenclature	AU		UU		UC		TSR, %
	IDT, psi	Met Spec	IDT, psi	Met Spec	IDT, psi	Met Spec	
9145-0-6428-S2*	160	Yes	118	Yes	93	Yes	78
9145-20-6428-S2	170	Yes	145	Yes	103	Yes	71
9145-25-6428-S2	175	Yes	158	Yes	113	Yes	72
9145-20-5834-S2	118	Yes	89	Yes	70	Yes	78
9145-25-5834-S2	120	Yes	97	Yes	77	Yes	80
9262-0-58H22-S4	171	Yes	132	Yes	88	Yes	67
9262-20-58H22-S4	200	No	169	Yes	137	Yes	81
9262-25-58H22-S4	205	No	176	No	141	Yes	80
9262-36-58S28-S4+RA*	158	Yes	127	Yes	102	Yes	80
9231-0-58H22-S5+Z	101	Yes	69	Yes	56	Yes	81
9231-20-58H22-S5+Z	117	Yes	83	Yes	63	Yes	75
9231-25-58H22-S5+Z	119	Yes	86	Yes	70	Yes	82
9231-20-52H28-S5+Z	152	Yes	102	Yes	90	Yes	88
9231-40-52S28-S5+Z*	139	Yes	101	Yes	89	Yes	88
9231-40-52S28-S3+Z	130	Yes	99	Yes	86	Yes	87

*: Duplicated same as WSDOT JMF

Table 27: LMLC CT-Index and Fracture Energy Results

Mix Nomenclature	AU		UU		UC	
	CT-Index	Fracture Energy, J/m^2	CT-Index	Fracture Energy, J/m^2	CT-Index	Fracture Energy, J/m^2
9145-0-6428-S2*	52	7918	103	7609	335	8949
9145-20-6428-S2	45	8193	82	8803	305	8901
9145-25-6428-S2	42	8608	80	9162	228	9180
9145-20-5834-S2	36	5308	81	5101	202	5294
9145-25-5834-S2	31	5686	66	5207	172	5624
9262-0-58H22-S4	77	9550	121	8652	233	7007
9262-20-58H22-S4	55	10234	92	10447	201	10801
9262-25-58H22-S4	47	10053	86	10589	173	10689
9262-36-58S28-S4+RA*	71	8943	115	8450	262	8669
9231-0-58H22-S5+Z	98	5999	146	4767	312	5024
9231-20-58H22-S5+Z	94	7067	133	5587	278	5383
9231-25-58H22-S5+Z	87	6909	129	6039	253	6607
9231-20-52H28-S5+Z	82	8704	177	7456	413	8955
9231-40-52S28-S5+Z*	74	7820	126	6721	294	7863
9231-40-52S28-S3+Z	60	6726	103	6165	268	7358

*: Duplicated same as WSDOT JMF

The IDT strength values, CT-Index, and fracture energy results are plotted in Figure 20, Figure 21, and Figure 22, respectively, for the three different conditions per mixture. The average IDT strength of 15 LMLC mixtures under UU, UC, and AU conditioning are equivalent to 117, 92, and 149 psi, respectively. As expected, the freeze-thaw damage dropped the average tensile strength on average by 21% relative to UU samples, whereas long-term aging increased the tensile strength on average by 28% relative to UU samples. In terms of CT-Index, the average values of 15 LMLC mixtures subjected to UU, UC, and AU conditioning are 109, 259, and 63, respectively. Despite the moisture damage of UC samples, the freeze-thaw cycle made these samples less brittle, reducing the post-peak slope of the load-displacement curve, hence increasing the CT-Index on average by 137%

comparing to UU samples. Conversely, the long-term aging of AU specimens reduced the average CT-Index by 43%, when compared to UU specimens.

The higher RAP amounts increased the strength values and lowered the CT-Index between different mixture types. However, the use of a recycling agent with high RAP mixture (9262-36-58S28-S4+RA) effectively reduced the mixture stiffness, thus lowered the relative IDT strength, and increased the CT-Index. The WSDOT maximum IDT strength limit of 175 psi was met for all 15 mixtures except for the UU sample of mix 9262-25-58H22-S4. Conversely, this mixture did not show the maximum CT-index, which highlights on the weak correlation between IDT strength values and CT-Index.

When assuming preliminary criteria for further evaluation, a minimum CT-Index of 65 was identified for UU samples, and a minimum CT-Index of 37 for AU samples. The difference is a 43% reduction in CT-Index due to long-term aging. A couple of mixtures exceeded the preliminary AU CT-Index set criterion, while meeting the UU criterion of 65, which indicates the importance of considering longer aging for cracking tests. In other words, the AU CT-Index identifies more propensity for cracking deficiencies compared to UU samples, which is logical with the difference in aging condition. In terms of fracture energy, the 9262-25-58H22-S4 mixture that exceeded WSDOT IDT strength value of 175 psi, had the highest fracture energy of 10589 J/m^2, which emphasizes the robust correlation between IDT strength and fracture energy parameters. Accordingly, a maximum fracture energy preliminary criterion was identified at 10589 J/m^2 for UU samples, and 11436 J/m^2 for AU samples. Note the relatively small percent difference of an 8% increase from UU to AU conditions compared to the difference of over 40% for the CT-Index under the same aging conditions. In general, the recorded fracture energy increased with higher RAP doses, however this trend was not consistent between all mixtures as shown in Figure 22. Based on the IDT strength test results, the following can be concluded:

- 20% RAP increased the IDT strength by 24% and 13% for UU and AU specimens, respectively, within the same binder supplier grade and source, compared to virgin mixtures.
- 25% RAP increased the IDT strength by 31% and 16% for UU and AU specimens, respectively, within the same binder supplier grade and source, compared to virgin mixtures.
- 25% RAP increased the IDT strength by 6% and 2% for UU and AU specimens, respectively, within the same binder supplier grade and source, compared to 20% RAP mixtures.
- Using 36% RAP with softer virgin binder and 3.1% RA (9262-36-58S28-S4+RA), lowered the IDT strength by 4%, and 8% for UU, and AU specimens, respectively, compared to virgin mixtures.
- The 40% RAP with softer virgin binder in contract 9231, increased the IDT strength on average by 33% and 45% for UU and AU specimens, respectively, compared to virgin mixtures.

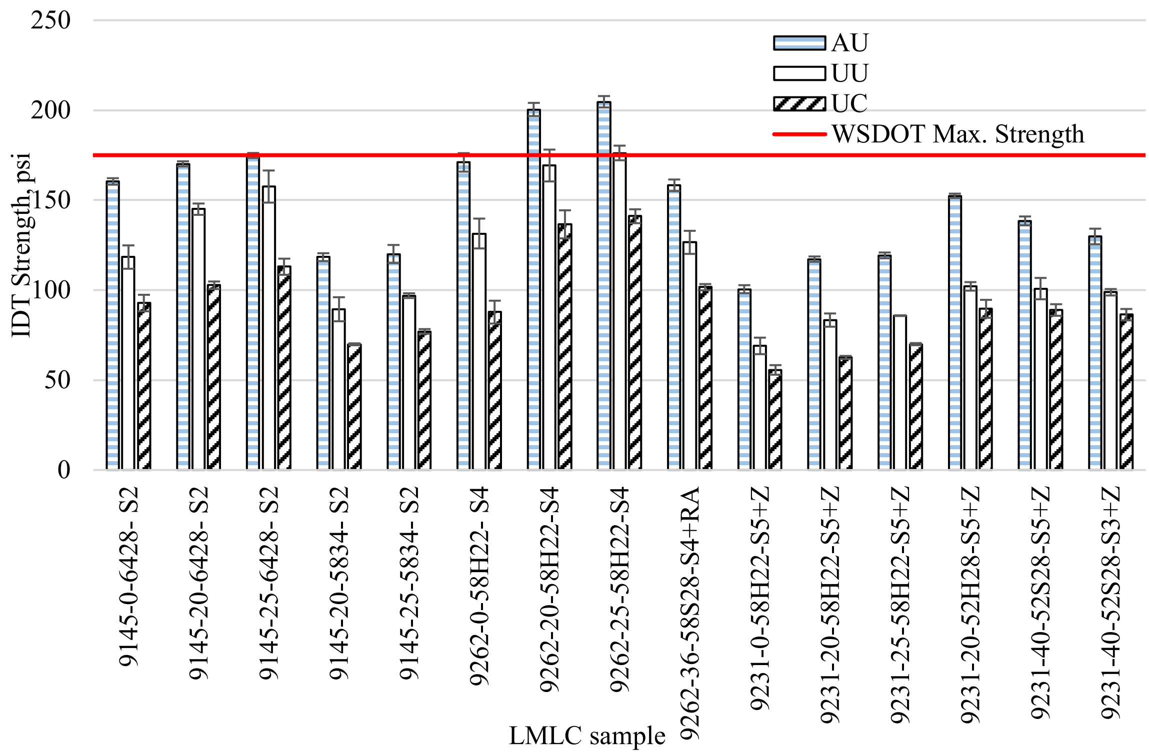


Figure 20: LMLC IDT Strength

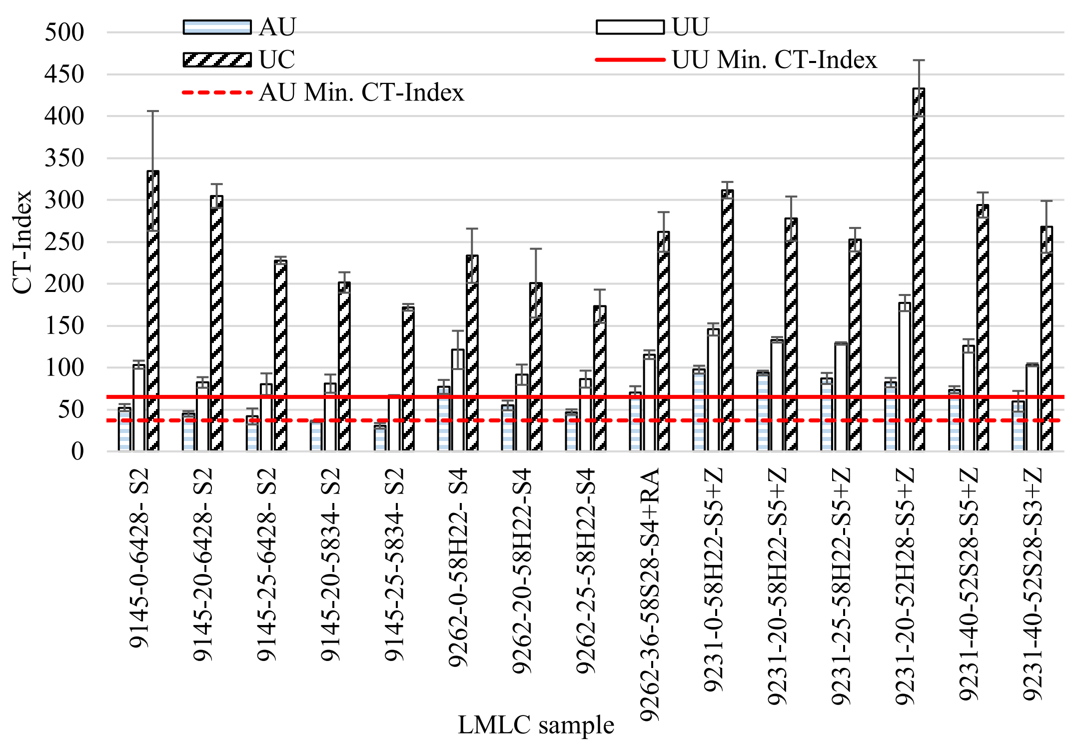


Figure 21: LMLC CT-Index

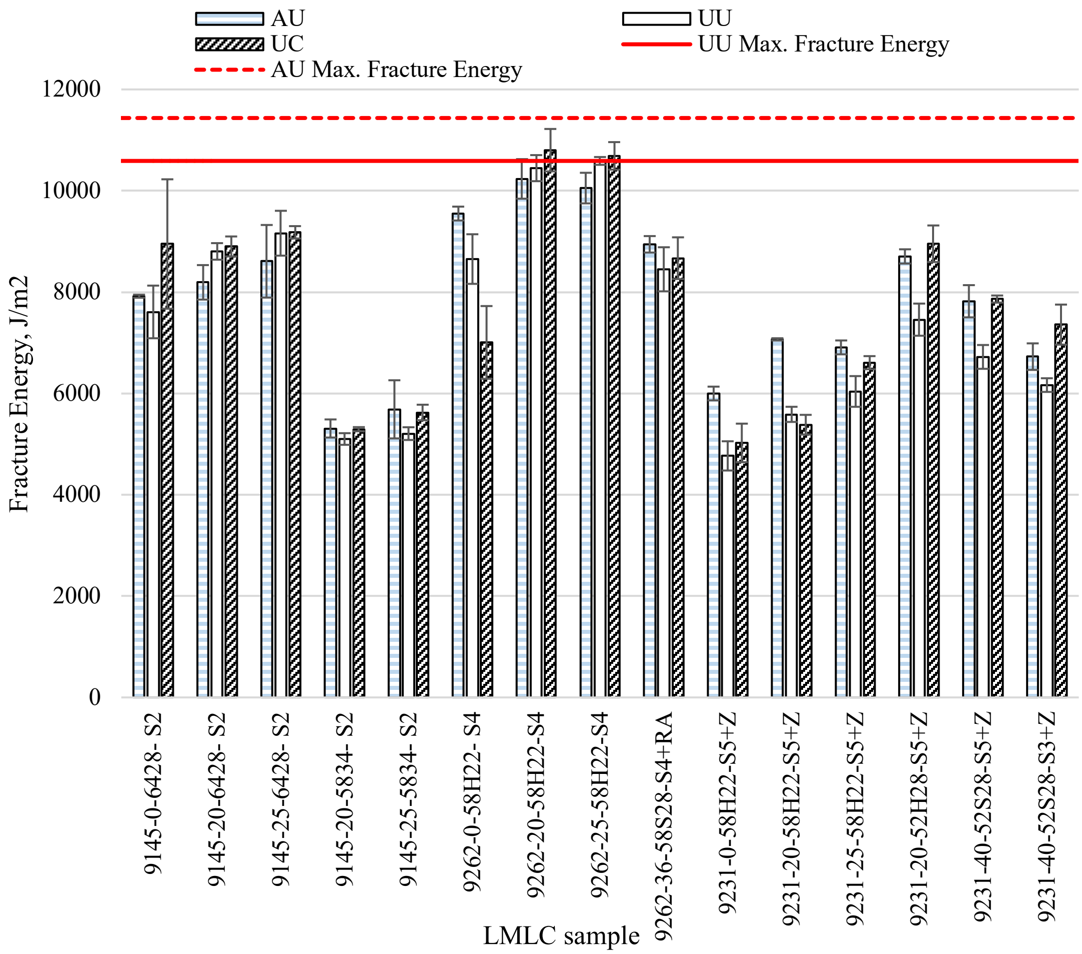


Figure 22: LMLC Fracture Energy

6.1.7 Dynamic Modulus Testing

The dynamic modulus test was used to develop master curves for each of the 15 LMLC mixtures, after short-term aging the loose mixtures as per AASHTO R30-02 for 4 hours at 275°F (135°C) (AASHTO R30 2019). This test was performed on two or three replicates per mixture, under three different frequencies at three different temperatures, previously tabulated in Table 14, prior to shifting each master curve to a reference temperature of 68°F. The master curves associated with 9145, 9262/9229, and 9231 contracts are shown in Figure 23, Figure 24, Figure 25, respectively.

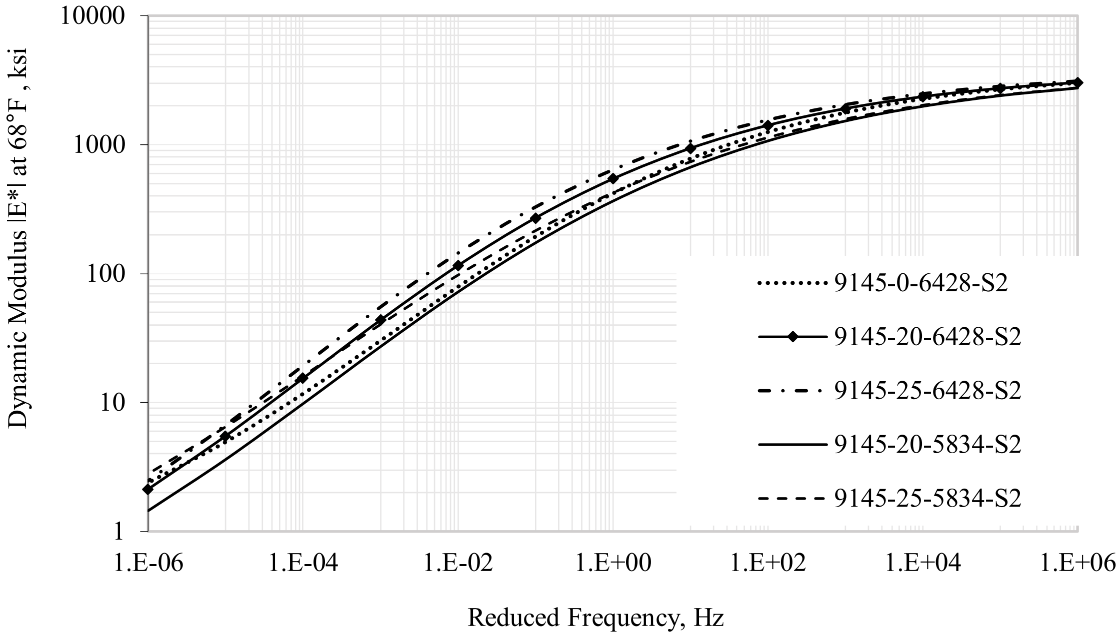


Figure 23: LMLC 9145 Master Curves

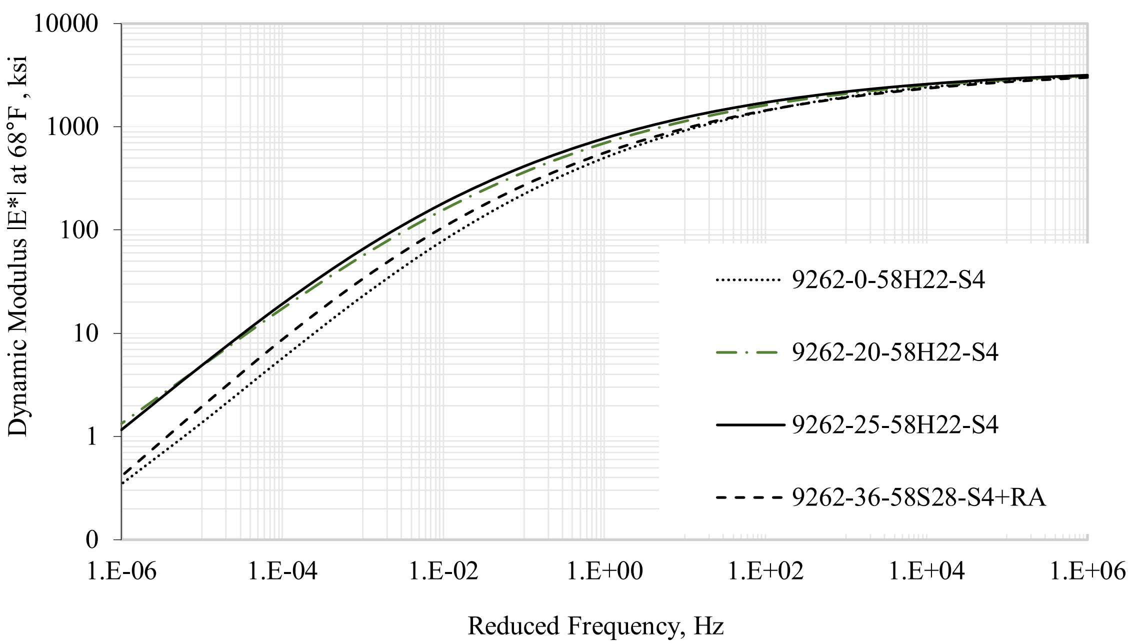


Figure 24: LMLC 9262/9229 Master Curves

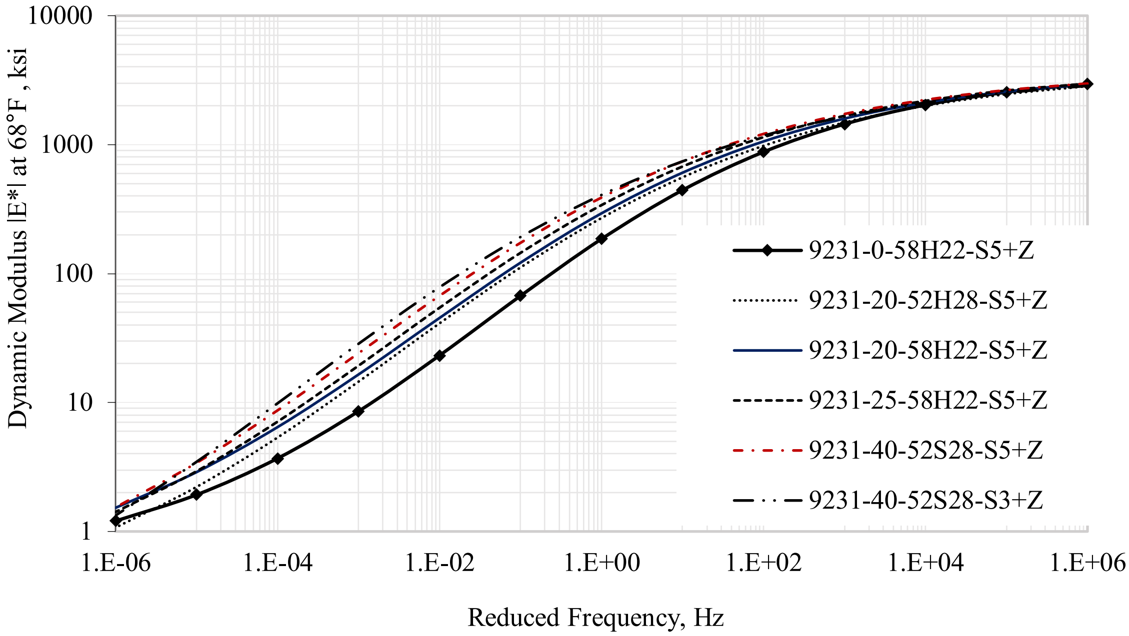


Figure 25: LMLC 9231 Master Curves

A comparison between the average dynamic modulus of the 15 laboratory mixtures, at 68°F and the most common frequency encountered by the pavement of 10Hz, is presented in Figure 26. Similar to the general trend previously observed in IDT results (section 6.1.6), the mixture stiffness increases as RAP dose increases. However, dropping the virgin binder grade while adding more RAP, or incorporating RA may soften the mixture and improve the performance, as expected. The data in Figure 26 at intermediate temperature of 68°F and 10 Hz, suggests that the mix 9262-25-58H22-S4 had the highest dynamic modulus of 1220 ksi, while noting that same mix had the highest UU, UC, and AU IDT strengths as shown in Figure 20. On the other hand, mix 9231-0-58H22-S5+Z had the lowest dynamic modulus based on Figure 26 (at intermediate temperature of 68°F and 10 Hz), while showing the lowest UU, UC, and AU IDT strengths in Figure 20 and the highest AU CT-Index between all 15 mixes, as shown in Figure 21. This analysis showed that dynamic modulus (E*) correlated best with the IDT strength parameter, comparing to CT-Index and fracture energy. This broad comparison between dynamic modulus at 68°F and IDT strength test performed at 77°F, is still rational despite the 9°F difference in comparison temperatures.

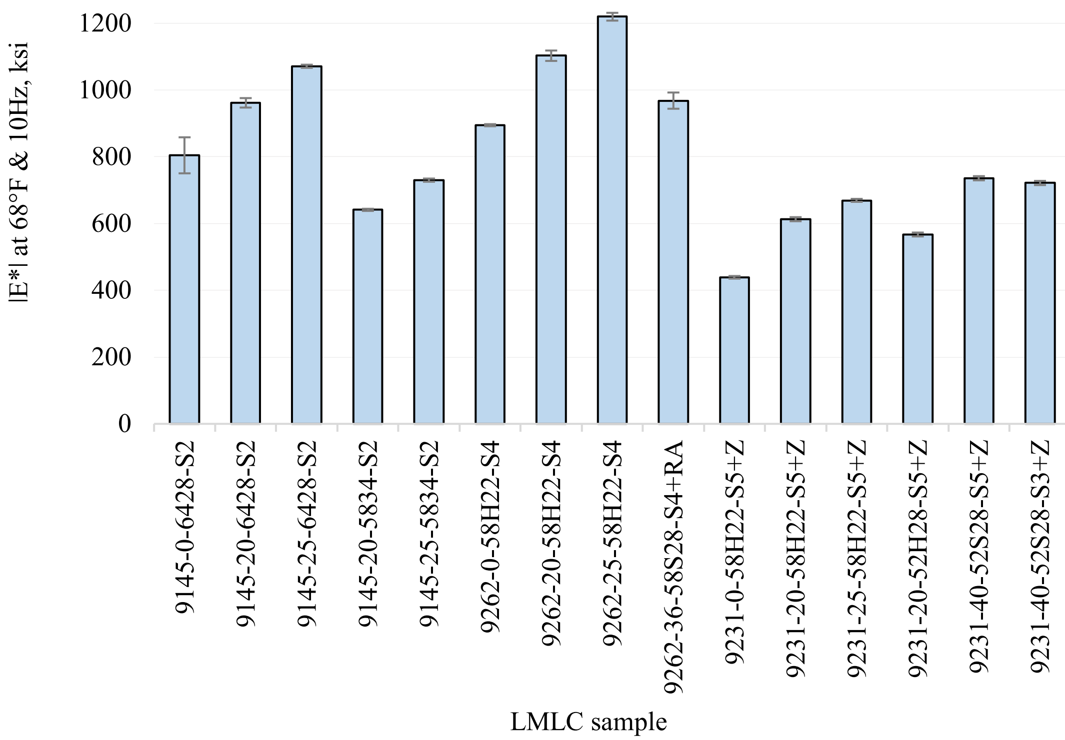


Figure 26: LMLC Average Dynamic Modulus at 68°F (20°C) and 10 Hz

6.1.8 Hamburg Wheel Track Test

The rut depth data generated from the HWTT performed by WSDOT for LMLC combinations are summarized in Table 28 and Figure 27. The presented rutting data demonstrate that even with softer virgin binder and/or recycling agent, none of the LMLC samples exceeded WSDOT criterion of 10 mm rut depth after 15,000 passes. Additionally, none of the mixtures, except the mixture 9231-40-52S28-S3+Z, experienced stripping failure. It can be inferred that all the evaluated mixtures still meet WSDOT rutting specification regardless of the recycled materials amount, or binder grade and supplier.

Table 28: LMLC HWTT results by WSDOT

Mix Nomenclature	Rut Depth at 15000 passes, mm
9145-0-6428-S2*	3.9
9145-20-5834-S2	3.1
9145-20-6428-S2	3.1
9145-25-5834-S2	4.0
9145-25-6428-S2	2.9
9262-0-58H22-S4	3.6
9262-20-58H22-S4	3.7
9262-25-58H22-S4	3.9
9262-36-58S28-S4+RA*	3.8
9231-0-58H22-S5+Z	6.9
9231-20-58H22-S5+Z	4.4
9231-20-52H28-S5+Z	6.7
9231-25-58H22-S5+Z	4.9
9231-40-52S28-S3+Z	8.7
9231-40-52S28-S5+Z*	5.6

*: Duplicated same as WSDOT JMF

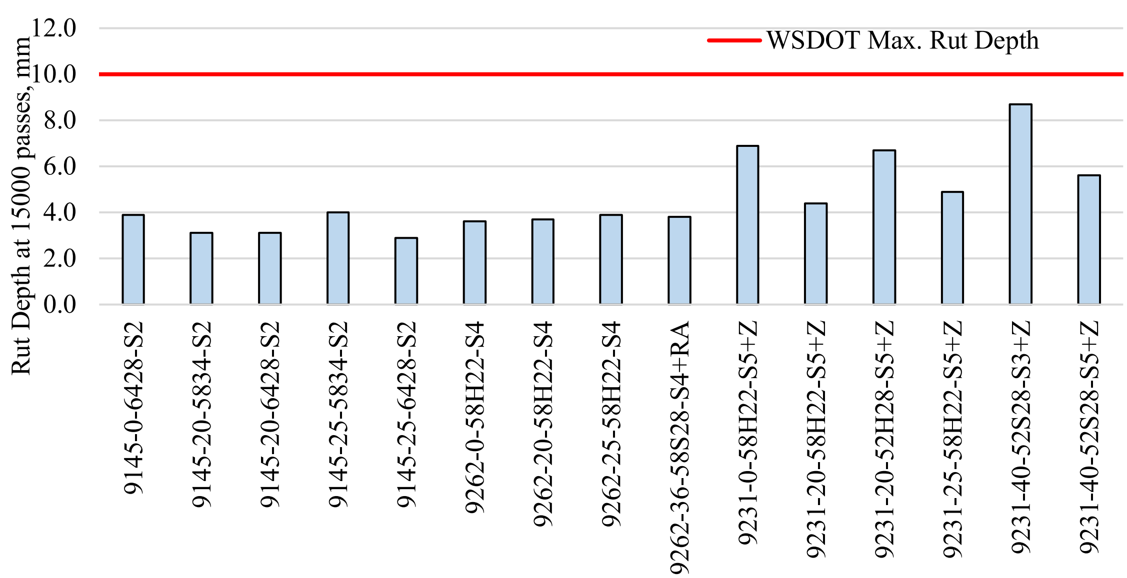


Figure 27: LMLC HWTT rut depth

6.2 FMLC Evaluation

6.2.1 IDT Strength Testing

Loose field mixtures were received for the following contracts: 9145, 9262, 9229, and 9231, with the aim of preparing FMLC samples for further IDT strength performance testing. Similar to LMLC samples, the IDT strength test was performed at below three conditions, while mentioning that the loose field mixtures were considered field short-term aged and were not subjected to any conditioning prior to compaction:

- UU (Unaged Unconditioned): No freeze-thaw cycle
- UC (Unaged Conditioned): One freeze-thaw cycle
- AU (long-term Aged Unconditioned): 5 days at 185°F on compacted samples without any freeze thaw cycle

Table 29 and Table 30 summarize the mean IDT test results in terms of IDT strength, TSR, CT-Index, and fracture energy. Figure 28, Figure 29, and Figure 30 illustrate the results of IDT strength, CT-Index, and fracture energy, respectively. All the dry tensile strength values of UU samples, met the WSDOT maximum criterion of 175 psi. Despite the fact that contracts 9262 and 9229 have same mix design, their IDT test results differed by 12% and 6%, for UU strength and CT-Index, respectively, which may be due to variation in plant production. The 95% confidence level error bars on the plots illustrate in most cases, lower variability associated with IDT strength results compared to CT-Index and fracture energy results. As observed with LMLC samples, the freeze-thaw cycle made the samples less brittle, reducing the post-peak slope of the load-displacement curve, hence increasing the CT-Index on average by 188% comparing to UU samples. Conversely, the long-term aging of AU specimens reduced the average CT-Index by 37%, when compared to UU specimens. The results indicate that all evaluated mixtures met the preliminary minimum CT-Index criteria under UU and AU conditions, while exceeding the preliminary maximum fracture energy criteria in many cases. When comparing the field mixtures to the replicated LMLC samples, the IDT strength parameter exhibited the most consistency between LMLC and FMLC performance data, when compared to the CT-Index and fracture energy.

Table 29: FMLC IDT Strength Results and TSR

Contract	AU		UU		UC		TSR, %
	IDT, psi	Met Spec	IDT, psi	Met Spec	IDT, psi	Met Spec	
9145	181	No	147	Yes	85	Yes	58
9229	N/A	N/A	162	Yes	127	Yes	79
9262	150	Yes	145	Yes	143	Yes	99
9231	144	Yes	139	Yes	107	Yes	77

N/A: Not Applicable due to lack of materials received

Table 30: FMLC CT-Index and Fracture Energy

Contract	AU		UU		UC	
	CT-Index	Fracture Energy, J/m^2	CT-Index	Fracture Energy, J/m^2	CT-Index	Fracture Energy, J/m^2
9145	50	11828	91	12859	327	10310
9229	N/A	N/A	91	14248	263	16222
9262	50	7370	86	11832	182	14239
9231	68	8154	91	8294	264	9149

N/A: Not Applicable due to lack of materials received

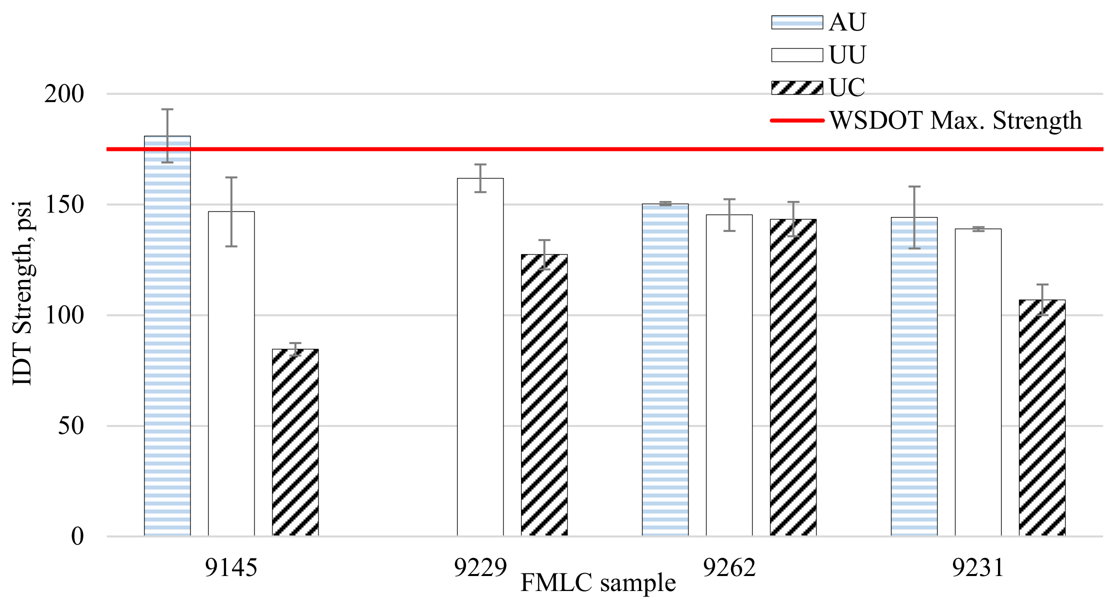


Figure 28: FMLC IDT Strength

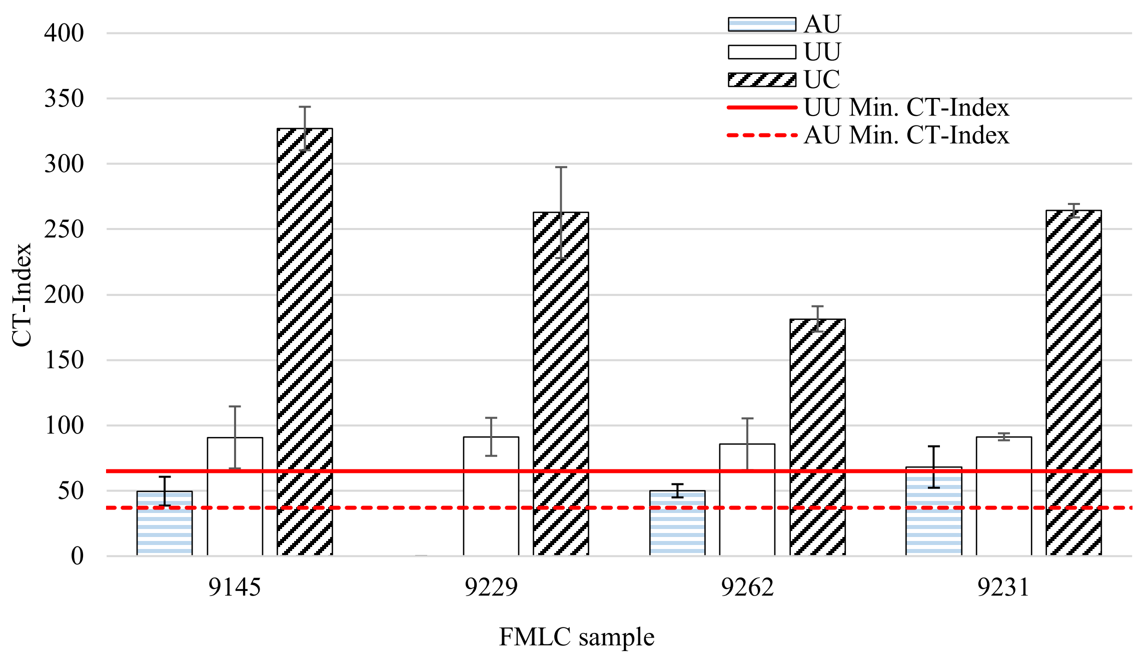


Figure 29: FMLC CT-Index

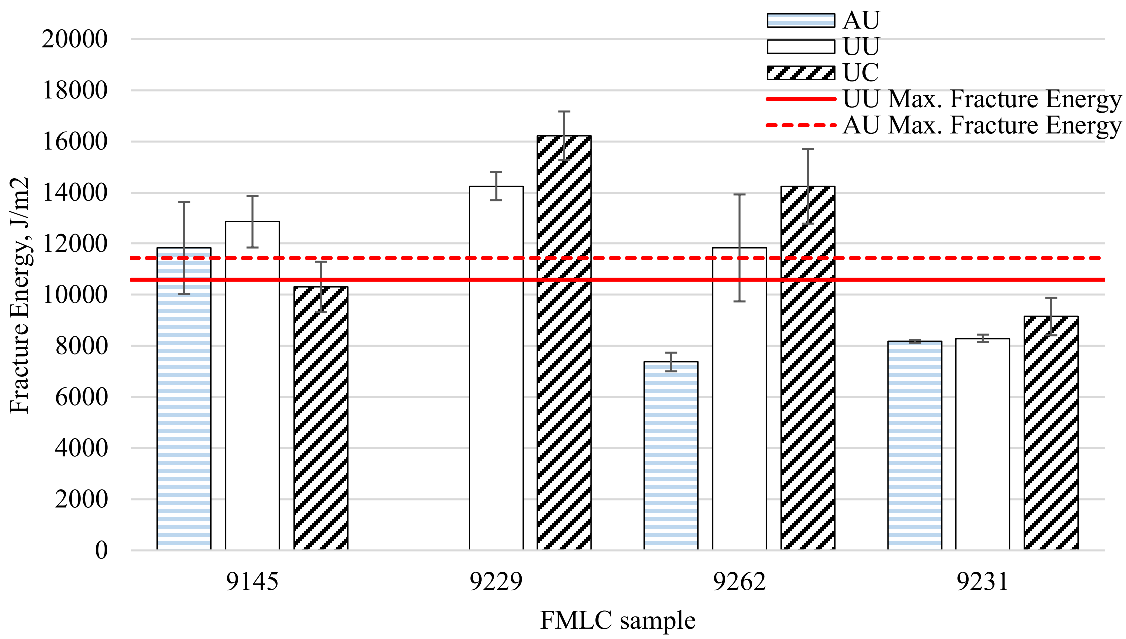


Figure 30: FMLC Fracture Energy

6.2.2 Extraction and Binder Grading

After performing the IDT strength performance testing, some UU and AU samples were subjected to centrifuge extraction and recovery of the asphalt binder, which was then graded per the same AASHTO test methods/specifications used for virgin and blended binder samples. Four different PG were developed as shown per the flowcharts in Figure 31, from UU and AU IDT test specimens. The binder grading results are summarized in Table 31 and Table 32 for 9145, and 9231 contracts, in terms of four different PG. Both contracts, 9145 and 9231 exceeded the ΔTc cracking limit of -5.0°C under PG4 aging conditions.

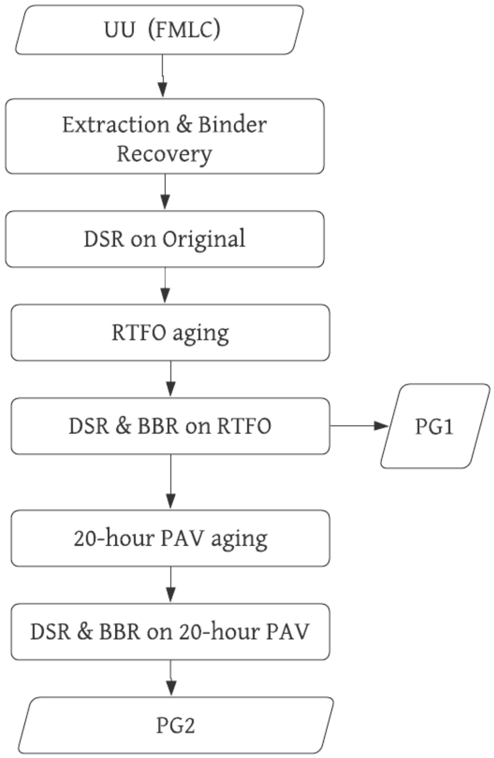


Figure 31: PG Procedure for Field Loose Mix

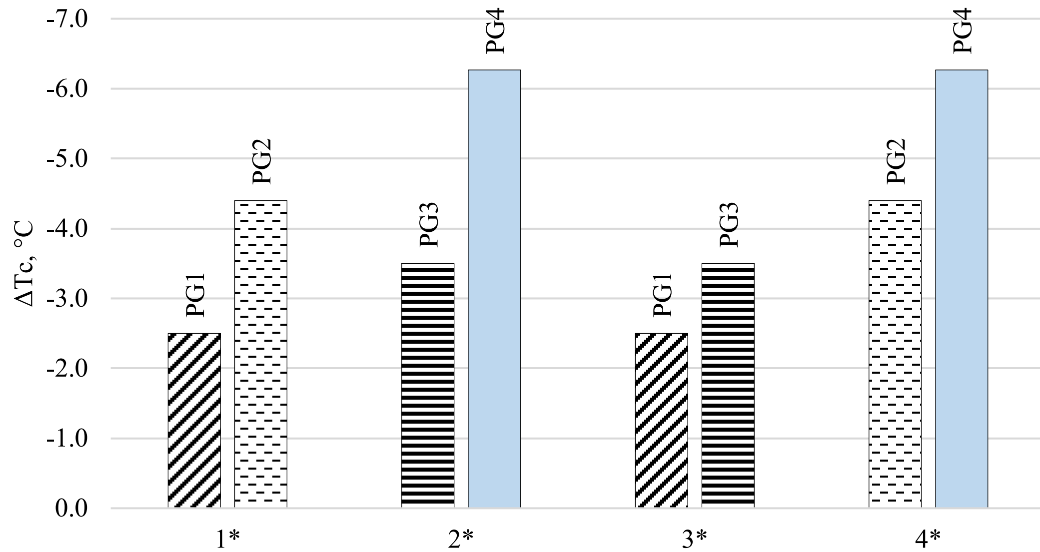
Table 31: FMLC PG Results (Contract 9145)

Material Description		True Grade	PG Grade	Tc (high)	Tc (int)	Tc (low)	ΔTc
PG1	9145-UU	75.7-27.6	70-22	75.7	19.0	-17.6	-2.5
PG2	9145-UU	75.7-24.9	70-22	75.7	22.0	-14.9	-4.4
PG3	9145-AU	87.0-24.4	82-22	87.0	22.4	-14.4	-3.5
PG4	9145-AU	87.0-20.7	82-16	87.0	25.4	-10.7	-6.3

Table 32: FMLC PG Results (Contract 9231)

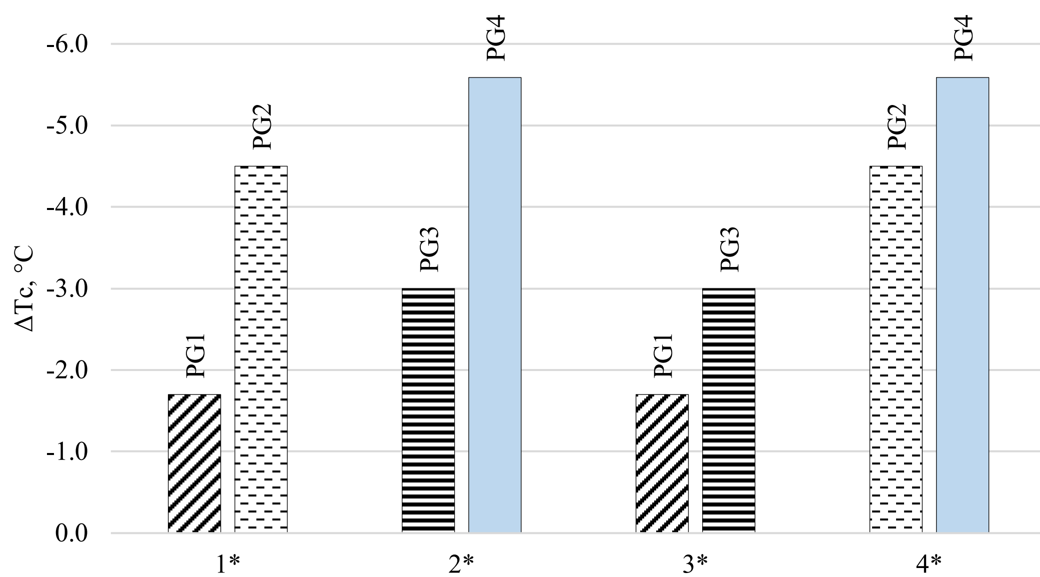
Material Description		True Grade	PG Grade	Tc (high)	Tc (int)	Tc (low)	ΔTc
PG1	9231-UU	71.3-28.2	70-28	71.3	19.5	-18.2	-1.7
PG2	9231-UU	71.3-24.0	70-22	71.3	21.9	-14.0	-4.5
PG3	9231-AU	83.7-24.4	82-22	83.7	23.7	-14.4	-3.0
PG4	9231-AU	83.7-21.2	82-16	83.7	25.9	-11.2	-5.6

Figure 32 and Figure 33 examine the impact of 20-hour PAV aging by comparing the ΔTc parameter between PG1 versus PG2 for UU IDT samples, and PG3 versus PG4 for AU IDT samples. For UU samples, it can be seen that 20-hour PAV aging reduced ΔTc by 1.9°C, and 2.8°C for contracts 9145, and 9231 respectively. Whereas for AU samples, ΔTc dropped by 2.8°C, and 2.6°C for contracts 9145, and 9231, respectively. The same plots show the impact of long-term aging the compacted specimens for 5 days at 185°F, by comparing ΔTc parameter between PG1 versus PG3 (subjected only to RTFO aging), and PG2 versus PG4 (subjected to RTFO and 20-hour PAV). The graphs show that aging compacted specimens for 5 days at 185°F reduced ΔTc by 1.0°C, and 1.3°C for contracts 9145, and 9231, respectively after RTFO aging, compared to ΔTc dropping by 1.9°C, and 1.1°C for contracts 9145, and 9231, respectively, after RTFO and 20-hour of PAV aging. Despite the limited data points available, it can be concluded that 20-hour PAV was able to age the mixtures and drop ΔTc more than the 5 days at 185°F long-term mixture aging protocol on compacted specimens.



Note:-
1* = Effect of 20-hour PAV aging on ΔTc UU sample
2* = Effect of 20-hour PAV aging of ΔTc AU sample
3* = Effect of 5days 185°F (85°C) aging on binder ΔTc
4* = Effect of 5days 185°F (85°C) aging on binder ΔTc

Figure 32: FMLC 9145 ΔTc Comparison



Note:-
1* = Effect of 20-hour PAV aging on ΔTc UU sample
2* = Effect of 20-hour PAV aging of ΔTc AU sample
3* = Effect of 5days 185°F (85°C) aging on binder ΔTc
4* = Effect of 5days 185°F (85°C) aging on binder ΔTc

Figure 33: 9231 ΔTc Comparison

6.3 FMFC Evaluation

6.3.1 IDT Strength Testing

Field cores (FMFC) were obtained from WSDOT contracts 7706, 8438, 8128, 8624, 8433, 8441, and 8465 for further evaluation. The core dimensions were recorded and the bulk specific gravity (Gmb) of each core was measured. The cores were then divided into three different subsets prior to IDT strength testing:

- UU (Unaged Unconditioned): No freeze-thaw cycle
- UC (Unaged Conditioned): One freeze-thaw cycle
- AU (long-term Aged Unconditioned): 5 days at 185°F on compacted samples without any freeze thaw cycle

Note that after performing IDT strength test, some cores were used for Gmm determination and binder properties. Table 33 and Table 34 below summarize the IDT test results for IDT strength, TSR, CT-Index, and fracture energy. The AV% was computed for UC samples for saturation, while assuming AU and UU samples collected from same field area, had the same average AV%. Similarly, Figure 34, Figure 35 and Figure 36 illustrate the IDT strength, CT-Index, and fracture energy, respectively. There was not a clear trend observed between these test results, due to the high field variability including thickness and AV% among FMFC samples within the same contract, which amplified significantly the error bars plotted in below graphs. The air voids level varied from 3.6% to 8.6% among all of the contracts and on average by 1.3% within a contract. Hence this inconsistency made the IDT strength of UC samples higher than UU samples in some cases, leading to a TSR > 100%. All the UU tensile strength values reported were less than the WSDOT criterion of

175 psi, except for contract 8441, which barely exceeded the limit with 176 psi, and resulted in the highest AU strength of 177 psi. Furthermore, the fracture energy preliminary criteria were met for all mixtures under AU and UU conditions, while the preliminary CT-Index criteria were exceeded in some cases. The 95% confidence level error bars on the plots illustrate in most cases, lower variability associated with IDT strength results compared to CT-Index and fracture energy results.

Table 33: FMFC IDT Strength Results and TSR

FMFC Contract ID	AU		UU		UC		TSR, %
	IDT, psi	Met Spec	IDT, psi	Met Spec	IDT, psi	Met Spec	
7706 WEST	139	Yes	147	Yes	116	Yes	79
7706 EAST	135	Yes	119	Yes	123	Yes	103
8128	152	Yes	143	Yes	189	No	132
8438	167	Yes	163	Yes	170	Yes	104
8433	103	Yes	139	Yes	134	Yes	96
8441	177	No	176	No	181	No	103
8465	126	Yes	150	Yes	164	Yes	109
8624	155	Yes	138	Yes	175	Yes	126

Table 34: FMFC CT-Index and Fracture Energy

FMFC Contract ID	AU		UU		UC	
	CT-Index	Fracture Energy, J/m^2	CT-Index	Fracture Energy, J/m^2	CT-Index	Fracture Energy, J/m^2
7706 WEST	32	8973	39	6623	76	5664
7706 EAST	36	6751	89	8920	123	9309
8128	88	7702	190	9401	174	11460
8438	35	6871	97	9066	40	11749
8433	97	5417	47	5834	88	6782
8441	44	6937	99	8995	119	9837
8465	144	6437	231	7404	434	9700
8624	29	5486	77	7032	70	7718

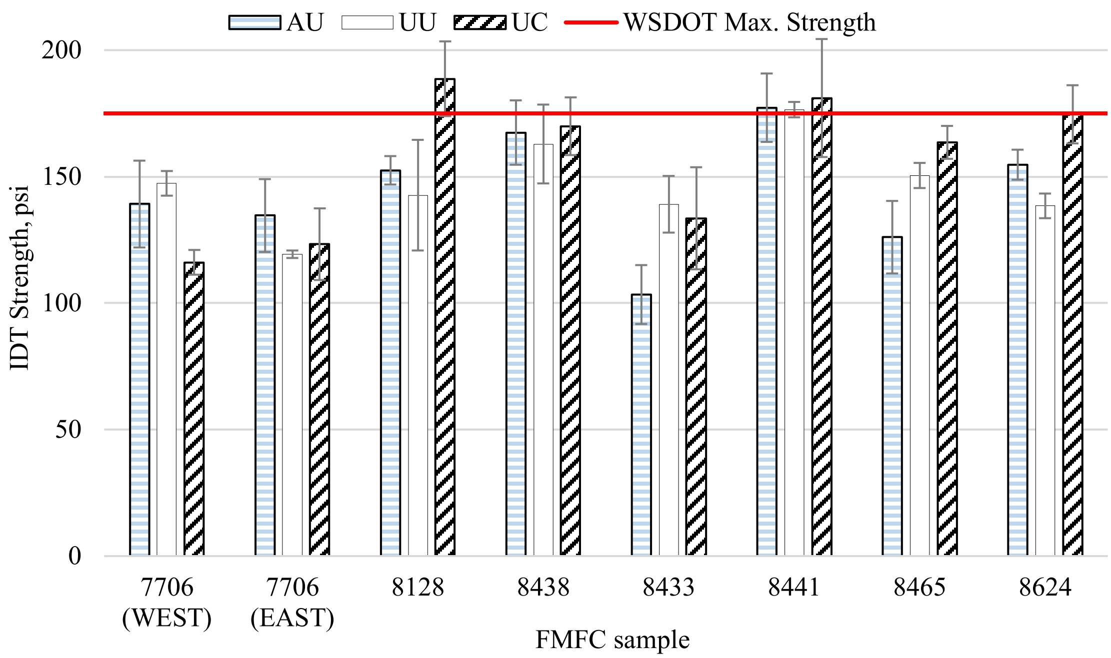


Figure 34: FMFC IDT Strength

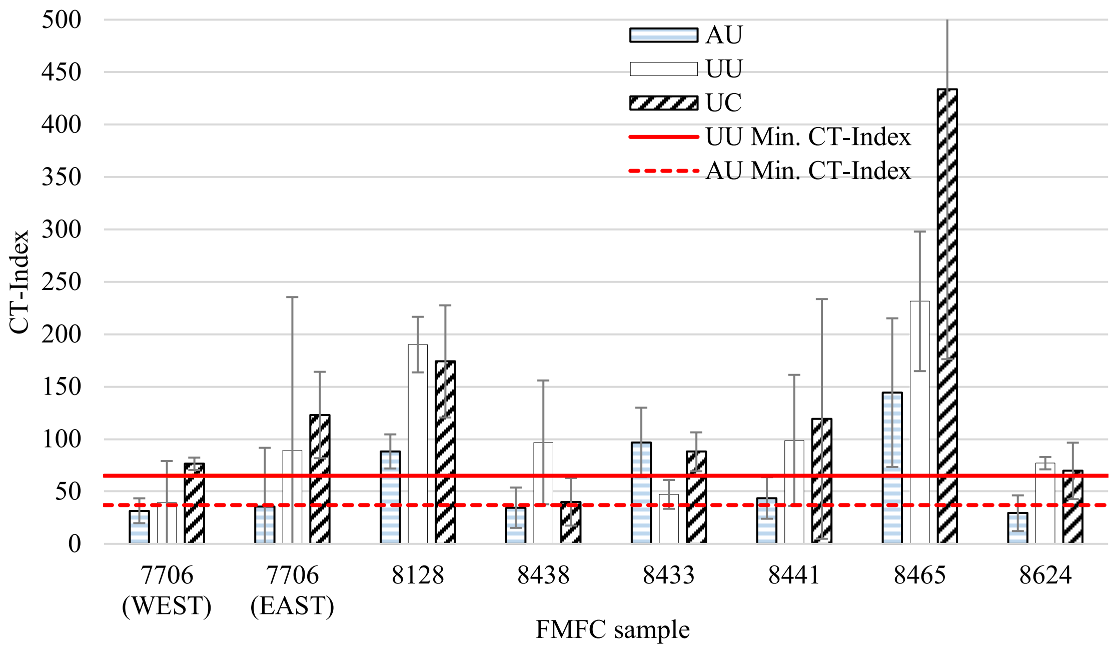


Figure 35: FMFC CT-Index

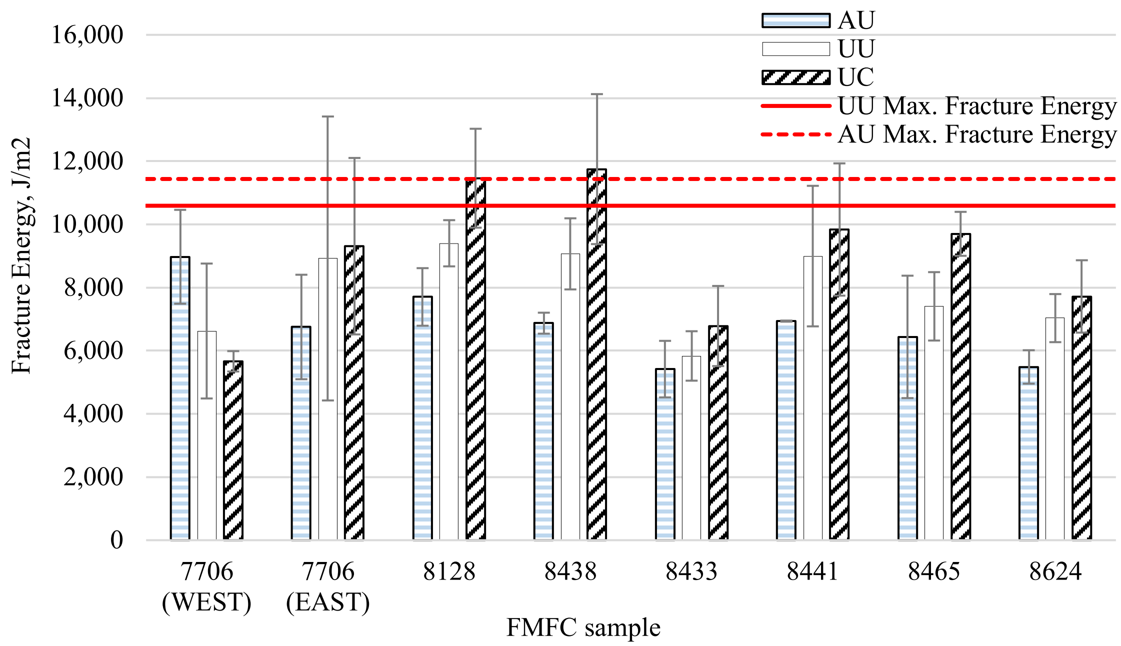


Figure 36: FMFC Fracture Energy

6.3.2 Extraction and Binder Grading

After the IDT strength testing, AU and UU field cores were subjected to extraction and recovery using the same methods as for FMLC samples in order to grade the asphalt binder. Four different PG were generated as per the flowchart in Figure 37, from UU and AU IDT test specimens. The binder grading results are shown in Table 35 for all seven contracts, following each of the four different PG. Similar to the grading observations of the virgin binders and recovered binders from field mixture, the rate of increase of m-value was greater than the stiffness in most of the cases, which makes the binder strongly m-controlled, especially after extended aging. The recovered binder ΔTc data for the four different PG are compared in Figure 38, Figure 39, Figure 40, and Figure 41. Contract 8433 was the only contract that was significantly lower ΔTc than all other sections and exceeded the ΔTc cracking limit of -5.0°C for the four different PG.

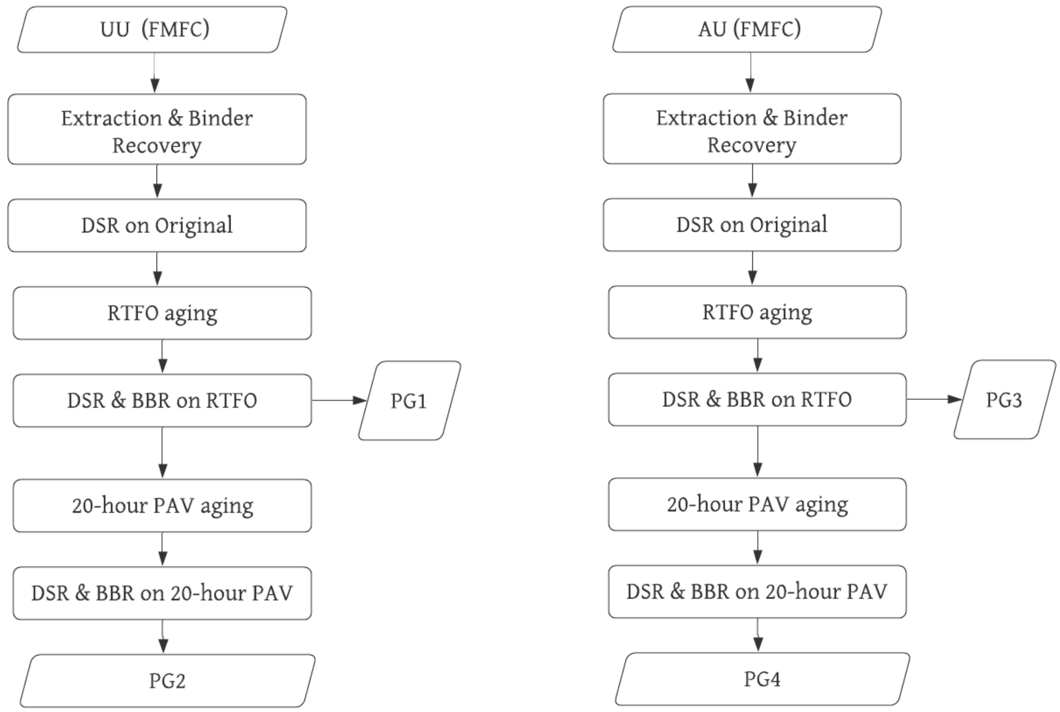
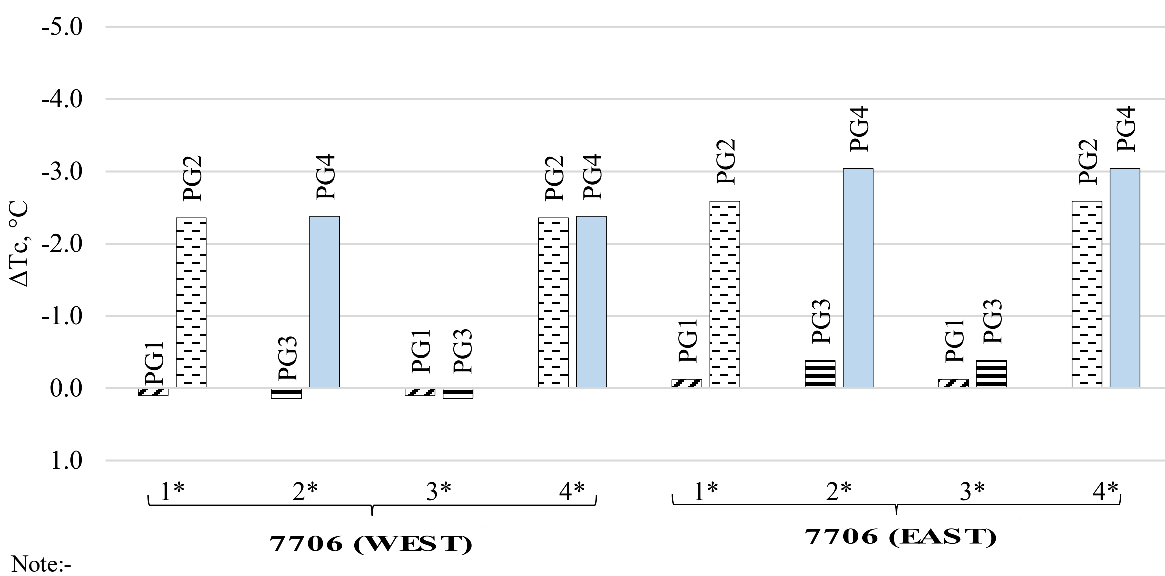


Figure 37: PG Procedure for FMFC

The impact of 20-hour PAV aging can be analyzed by comparing ΔTc parameter between PG1 versus PG2 for UU IDT samples, and PG3 versus PG4 for AU IDT samples. The data shows that 20-hour PAV aging reduced ΔTc on average by 2.5°C, and 2.8°C for UU and AU specimens, respectively. The same plots evaluate the impact of 5 days long-term aging on compacted specimens at 185°F, by comparing ΔTc between PG1 versus PG3 (subjected only to RTFO aging), and PG2 versus PG4 (subjected to RTFO and 20-hour PAV aging). The four graphs demonstrate that 5 days aging at 185°F on compacted specimens, dropped ΔTc on average by 1.0°C after RTFO aging, while lowering it by 1.3°C after RTFO and 20-hour PAV. Consistent with the FMLC findings in section 6.2.2, the results suggest that 20-hour PAV was able to age the mixtures and drop ΔTc more than the 5 days at 185°F long-term aging protocol on compacted specimens.

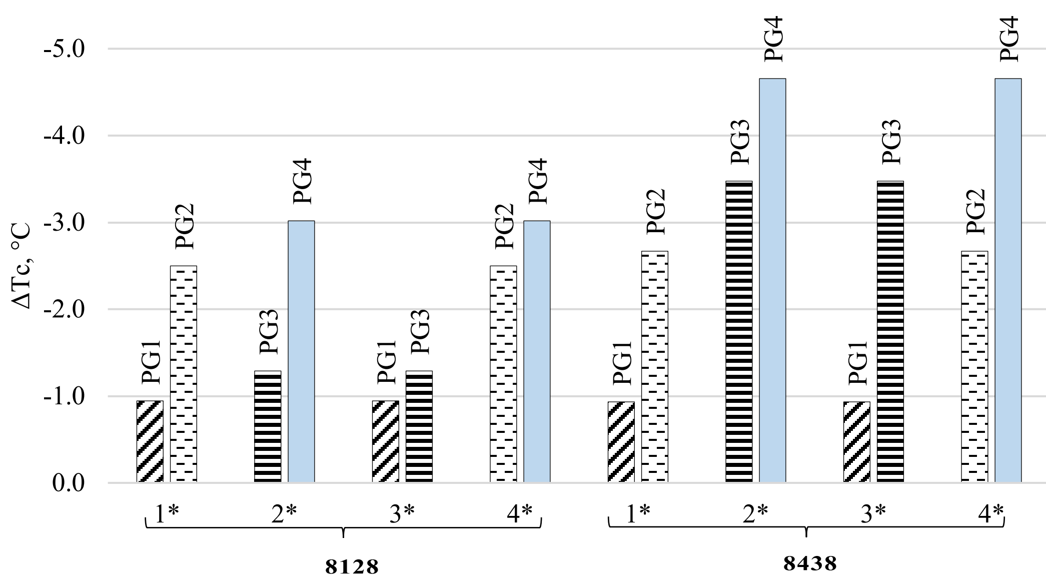
Table 35: FMFC PG Results

FMFC Contract	PG	True grade	PG grade	Tc, High	Tc, int	Tc, low	ΔTc
7706 (WEST)	PG1	86.3-19.5	82-16	86.3	30.2	-9.5	0.1
	PG2	86.3-15.0	82-10	86.3	33.7	-5.0	-2.4
	PG3	90.1-18.1	88-16	90.1	31.7	-8.1	0.1
	PG4	90.1-14.1	82-10	90.1	35.2	-4.1	-2.4
7706 (EAST)	PG1	79.7-25.8	76-22	79.7	21.9	-15.8	-0.1
	PG2	79.7-21.5	76-16	79.7	26.7	-11.5	-2.6
	PG3	83.8-23.4	82-22	83.8	25.5	-13.4	-0.4
	PG4	83.8-18.9	82-16	83.8	29.6	-8.9	-3.0
8128	PG1	80.9-23.4	76-22	80.9	24.7	-13.4	-0.9
	PG2	80.9-20.3	76-16	80.9	28.0	-10.3	-2.5
	PG3	81.4-22.7	76-22	81.4	24.9	-12.7	-1.3
	PG4	81.4-19.3	76-16	81.4	28.3	-9.3	-3.0
8438	PG1	83.3-22.5	82-22	83.3	26.4	-12.5	-0.9
	PG2	83.3-21.0	82-16	83.3	30.1	-11.0	-2.7
	PG3	83.9-20.1	82-16	83.9	27.0	-10.1	-3.5
	PG4	83.9-18.5	82-16	83.9	30.6	-8.5	-4.7
8433	PG1	79.8-28.1	76-28	79.8	16.2	-18.1	-5.5
	PG2	79.8-21.4	76-16	79.8	19.6	-11.4	-11.3
	PG3	83.2-26.0	82-22	83.2	18.2	-16.0	-6.8
	PG4	83.2-21.6	82-16	83.2	21.0	-11.6	-13.9
8441	PG1	80.9-23.8	76-22	80.9	25.4	-13.8	-0.1
	PG2	80.9-20.1	76-16	80.9	28.4	-10.1	-2.4
	PG3	84.9-20.4	82-16	84.9	27.6	-10.4	-1.6
	PG4	84.9-18.0	82-16	84.9	29.7	-8.0	-3.4
8465	PG1	80.7-23.8	76-22	80.7	24.6	-13.8	-0.8
	PG2	80.7-20.3	76-16	80.7	26.7	-10.3	-3.4
	PG3	81.3-23.3	76-22	81.3	27.3	-13.3	-1.1
	PG4	81.3-18.6	76-16	81.3	28.6	-8.6	-4.6
8624	PG1	81.5-22.8	76-22	81.5	25.9	-12.8	-0.1
	PG2	81.5-20.7	76-16	81.5	28.4	-10.7	-1.0
	PG3	90.4-19.9	88-16	90.4	29.2	-9.9	-1.9
	PG4	90.4-16.8	88-16	90.4	35.3	-6.8	-3.7



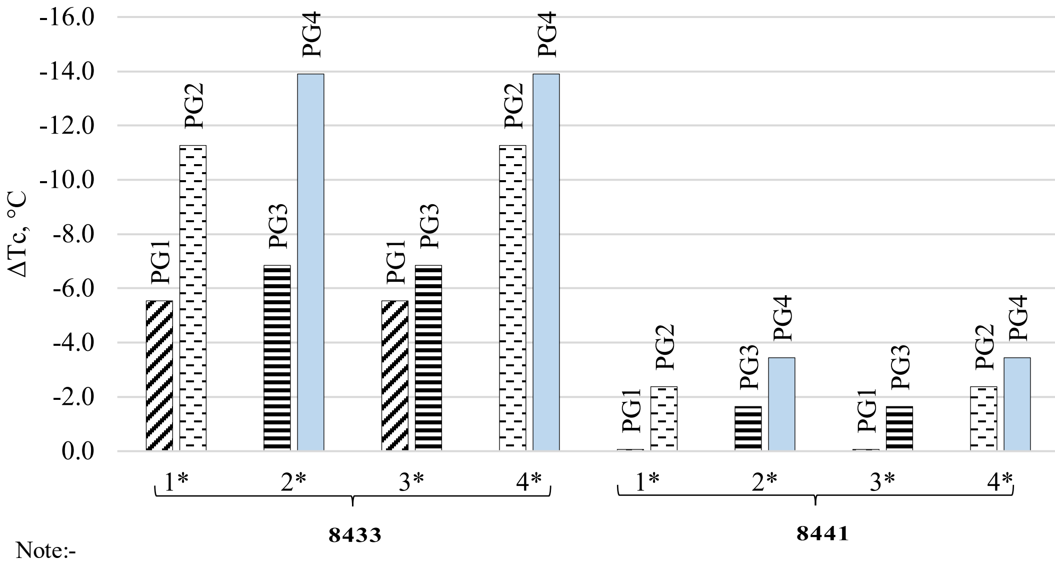
Note:-
1* = Effect of 20-hour PAV aging on ΔTc UU sample
2* = Effect of 20-hour PAV aging of ΔTc AU sample
3* = Effect of 5days 185°F (85°C) aging on binder ΔTc
4* = Effect of 5days 185°F (85°C) aging on binder ΔTc

Figure 38: FMFC 7706 ΔTc Comparison



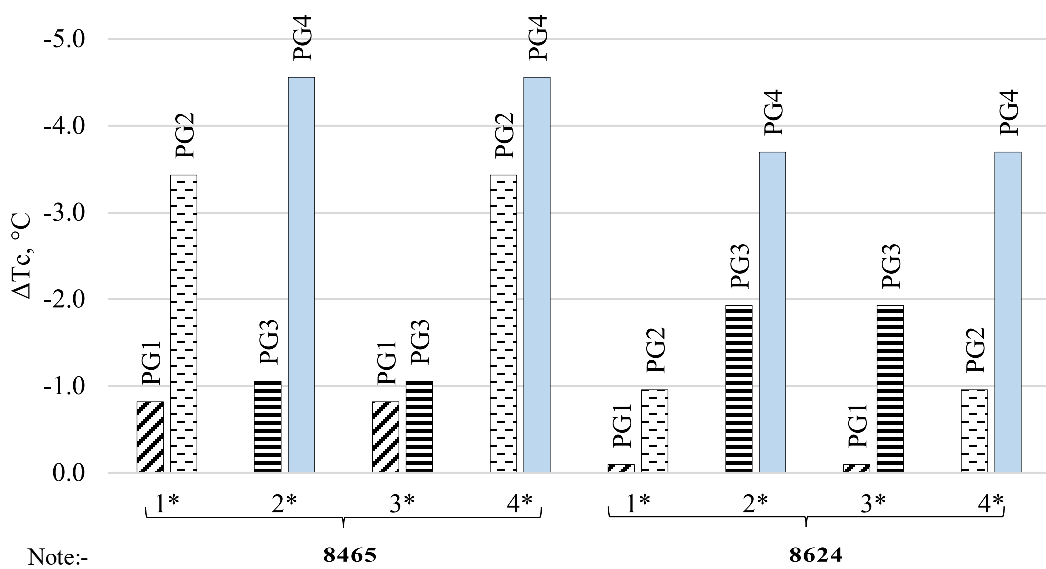
Note:-
1* = Effect of 20-hour PAV aging on ΔTc UU sample
2* = Effect of 20-hour PAV aging of ΔTc AU sample
3* = Effect of 5days 185°F (85°C) aging on binder ΔTc
4* = Effect of 5days 185°F (85°C) aging on binder ΔTc

Figure 39: FMFC 8128 and 8438 ΔTc Comparison



Note:-
1* = Effect of 20-hour PAV aging on ΔTc UU sample
2* = Effect of 20-hour PAV aging of ΔTc AU sample
3* = Effect of 5days 185°F (85°C) aging on binder ΔTc
4* = Effect of 5days 185°F (85°C) aging on binder ΔTc

Figure 40: FMFC 8433 and 8441 ΔTc Comparison



Note:-
1* = Effect of 20-hour PAV aging on ΔTc UU sample
2* = Effect of 20-hour PAV aging of ΔTc AU sample
3* = Effect of 5days 185°F (85°C) aging on binder ΔTc
4* = Effect of 5days 185°F (85°C) aging on binder ΔTc

Figure 41: FMFC 8465 and 8624 ΔTc Comparison

7 FMLC PAVEMENT MANAGEMENT SYSTEM DATA ANALYSIS

An objective of this effort was to assess the relative performance of asphalt pavements constructed with mixtures containing low and high levels of recycled materials. A rational approach to this is to analyze WSDOT pavement management system (PMS) data. Howell et al. incorporated WSDOT PMS in-service AC pavement condition data with multiple disconnected data sources to evaluate pavement performance and the influence of several factors on it (Howell et al. 2019). This included mix design, cost, construction, and field performance data. The integrated data architecture that was used, accommodated handling the large dataset and allowed for rapid analysis. WSDOT data was analyzed over the 10-year period from 2007 to 2017 to assess the performance of the following:

1. 9.5 mm versus 12.5 mm NMAS mixtures
2. Low versus high in-place density mixtures
3. Low (<20%) versus high (>20%) RAP mixtures

WSDOT initiated the use of high RAP (>20% RAP) mixtures in 2013, so there was more long-term performance data for low RAP, than high RAP mixtures. Wisely, WSDOT had constructed several test sections, coupled with additional testing and requirements. The hypothesis that high RAP mixtures result in lower structural capacity and rutting than low RAP mixtures was tested. Statistically significant differences were not observed among several analyzed properties. Figure 42 and Figure 43 show the properties analyzed between low and high RAP mixtures including average weighted density, pavement structural condition (PSC), and pavement rutting condition (PRC). Data analysis revealed a statistically significant difference in mean of asphalt contents, with high RAP mixes showing 0.2% lower asphalt content than low RAP mixtures. Overall average weighted in place density of high RAP mixtures was 0.14% lower than low RAP mixtures. However, this was not statistically sufficient to suggest a significant difference between the mixture types. A comprehensive analysis and statistical comparisons led to the following conclusions:

- Structural and rutting conditions were not statistically different for high and low RAP mixtures.
- Reduced PSC values showed higher variability in high RAP mixtures, possibly due to the smaller sample size for this mixture type.
- Four years after construction, the average weighted rutting by tonnage of high RAP mixtures was equivalent to 0.20 cm comparing to 0.34 cm for low RAP mixtures.

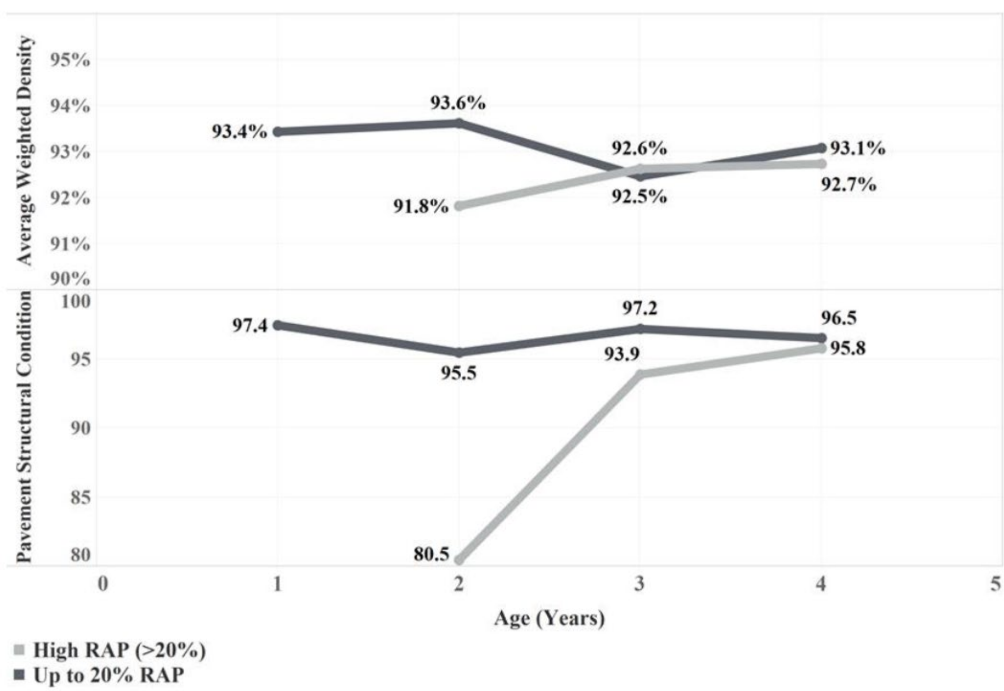


Figure 42: Average Weighted Pavement Structural Condition and Average Weighted Density for High and Low RAP Mixtures

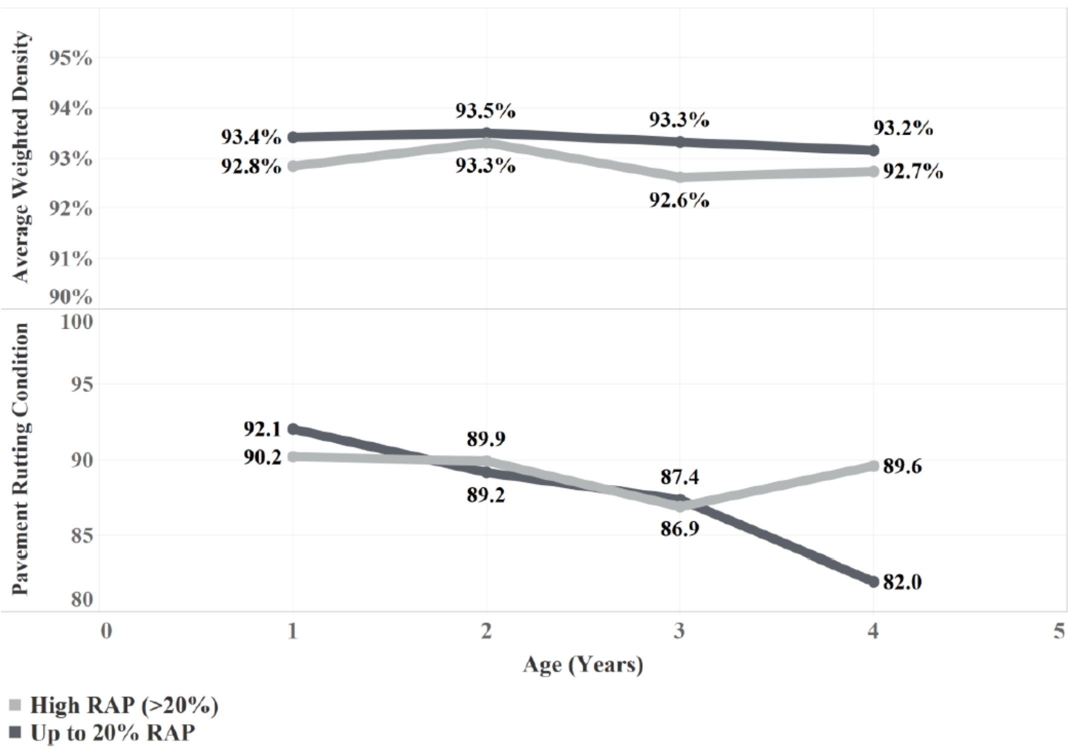


Figure 43: Average Weighted Pavement Rutting Condition and Average Weighted Density for High and Low RAP Mixtures

WSDOT provided field cores and detailed distress survey data, including rut depth (inch), IRI (inch/mile), alligator cracking (ft), longitudinal cracking (ft), patching (ft), and transverse cracking (count), for seven projects. The projects included three low RAP (≤20%) contracts (7706, 8465, and 8128) and four high RAP (>20%) contracts (8441, 8438, 8433, 8624). According to the WSDOT Pavement Management System, the equivalent cracking (EC), the alligator cracking component of equivalent cracking (ACEC), longitudinal cracking component of equivalent cracking (LCEC), transverse cracking component of equivalent cracking (TCEC), and patching cracking component of equivalent cracking (PTEC) were calculated as follows (Kay et al. 1993):

$$EC = ACEC + LCEC + TCEC + PTEC \qquad \text{Eq. (4)}$$

Where,

EC: Total equivalent cracking,

ACEC: alligator cracking component of equivalent cracking,

LCEC: longitudinal cracking component of equivalent cracking,

TCEC: transverse cracking component of equivalent cracking, and

PTEC: patching cracking component of equivalent cracking.

$$ACEC = AC3 + 0.445(AC2)^{1.15} + 0.13(AC1)^{1.35} \quad \text{Eq. (5)}$$

Where,

ACEC: alligator cracking component of equivalent cracking,

AC1: percent of wheel path length with hairline alligator cracking,

AC2: percent of wheel path length with spalled alligator cracking, and

AC3: percent of wheel path length with spalled and pumping alligator cracking.

$$LCEC = (0.1LC3) + 0.445(0.1LC2)^{1.15} + 0.13(LC1)^{1.35} \quad \text{Eq. (6)}$$

Where,

LCEC: longitudinal cracking component of equivalent cracking,

LC1: percent of section length with a less than 1/4-inch width severity level,

LC2: percent of section length with a greater than 1/4-inch width severity level, and

LC3: percent of section length with a spalling severity level.

$$TCEC = (0.8TC3) + 0.445(0.8TC2)^{1.15} + 0.13(0.8TC1)^{1.35} \quad \text{Eq. (7)}$$

Where,

TCEC: transverse cracking component of equivalent cracking,

TC1: number of transverse cracks per 100 ft of section length with a less than 1/4-inch width severity level, TC2: number of transverse cracks per 100 ft of section length with a greater than 1/4-inch width severity level, and

TC3: number of transverse cracks per 100 ft of section length with a spalling severity level.

$$PTEC = PT3 + 0.445[0.75(PT2)]^{1.15} + 0.13[0.75(PT1)]^{1.35} \qquad \text{Eq. (8)}$$

Where,

PTEC: patching component of equivalent cracking,

PT1: percent of wheel track length with BST patching,

PT2: percent of wheel track length with blade patching, and

PT3: percent of wheel track length with full depth patching.

Table 36 is a summary of the pavement condition results for the low RAP field projects.

Table 37 show the pavement condition results for the high RAP field projects, respectively. The pavement condition data available for high RAP projects was up to 5 years post-construction. However, the projects were not all constructed in the same season, so there was up to 3 years of post-construction data available for all projects. The surveyed units were averaged, with the cracking percentages calculated relative to the cracking rated length, consistently with the WSDOT PMS 1993 update (Kay et al. 1993). In cases where the cracking rated length was either zero or non-zero, but no distress was recorded with a crack length, the sample unit was assumed surveyed without any cracking observed, hence reported as 0% EC%. The greatest rut depth observed was 0.30 inch, for high RAP contract 8441, 4 years after construction, compared to 0.20 inch of rut depth within low RAP contracts over the same period. The lowest IRI observed was 46 inch/mile for the high RAP project 8433, 1-year post-construction, whereas the high RAP project 8438 experienced the maximum IRI of 135 inch/mile, 1-year post-construction.

Table 36: Low RAP Projects PMS Data

Years after const.		1	2	3	4	5	6	7	8	9
7706	Rutting, inch	0.11	0.12	0.17	0.19	0.20	0.17	0.23	0.25	0.29*
	IRI, inch/mile	61	54	88	86	58	71	65	72	56*
	ACEC, %	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01*
	TCEC, %	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.03	0.02*
	LCEC, %	0.06	0.04	0.00	0.00	0.01	0.05	0.00	0.00	0.10*
	PTEC, %	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.23	0.17*
	EC, %	0.06	0.02	0.00	0.00	0.01	0.06	0.05	0.26	0.27*
8465	Rutting, inch	0.13	0.13	0.16	0.20	0.21*	-	-	-	-
	IRI	61	63	62	70	77*	-	-	-	-
	ACEC	0.00	0.12	0.12	0.00	0.30*	-	-	-	-
	TCEC	0.00	0.18	0.17	0.02	0.36*	-	-	-	-
	LCEC	0.00	0.32	0.27	0.00	0.31*	-	-	-	-
	PTEC	0.00	0.02	0.73	0.00	0.06*	-	-	-	-
	EC, %	0.00	0.21	0.44	0.00	1.04*	-	-	-	-
8128	Rutting, inch	0.11	0.13	0.12*	-	-	-	-	-	-
	IRI	72	74	71*	-	-	-	-	-	-
	ACEC	0.07	0.88	0.01*	-	-	-	-	-	-
	TCEC	0.03	0.01	0.05*	-	-	-	-	-	-
	LCEC	0.19	0.78	1.79*	-	-	-	-	-	-
	PTEC	0.35	0.03	0.00*	-	-	-	-	-	-
	EC, %	0.12	0.35	0.21*	-	-	-	-	-	-

* Indicates 2018, the year FMFC cores were obtained and used in further regression analysis.

Table 37: High RAP Projects PMS Data

Years after const.		1	2	3	4	5
8441	Rutting, inch	0.21	0.19	0.23	0.30	0.30*
	IRI, inch/mile	73	75	76	82	79*
	ACEC, %	0.00	0.14	0.11	0.11	0.00*
	TCEC, %	0.01	0.00	0.00	0.00	0.00*
	LCEC, %	0.06	0.27	0.50	0.40	0.01*
	PTEC, %	22.53	10.66	9.83	21.42	0.00*
	EC, %	3.94	4.14	4.09	17.34	0.00*
8438	Rutting, inch	0.13	0.10	0.12	0.15	0.12*
	IRI, inch/mile	135	99	100	99	119*
	ACEC, %	0.00	0.00	0.00	0.00	0.00*
	TCEC, %	0.01	0.00	0.00	0.00	0.02*
	LCEC, %	0.00	0.00	0.00	0.00	0.07*
	PTEC, %	0.00	0.00	0.00	0.00	0.00*
	EC, %	0.01	0.00	0.00	0.00	0.08*
8433	Rutting, inch	0.06	0.10	0.19	0.22	0.17*
	IRI, inch/mile	46	48	54	56	52*
	ACEC, %	0.00	0.00	0.00	0.00	0.00*
	TCEC, %	0.00	0.00	0.03	0.17	0.32*
	LCEC, %	0.00	0.00	0.00	0.19	0.11*
	PTEC, %	0.00	0.00	0.08	0.00	0.01*
	EC, %	0.00	0.00	0.11	0.35	0.44*
8624	Rutting, inch	0.09	0.12	0.17	0.15*	-
	IRI, inch/mile	57	56	63	60*	-
	ACEC, %	0.07	0.00	0.00	0.00*	-
	TCEC, %	0.00	0.00	0.00	0.00*	-
	LCEC, %	0.07	0.07	0.00	0.00*	-
	PTEC, %	0.00	0.00	0.00	0.00*	-
	EC, %	0.01	0.01	0.00	0.00*	-

*: Indicates 2018, the year FMFC cores were obtained and used in further regression analysis.

Figure 44 and Figure 45 are based on the available data for the first three years after construction for all the projects. They show some overlapping of rut depth and IRI among high RAP (dotted lines) and low RAP (solid lines) projects suggesting similar performance as indicated by the analysis performed by Howell (Howell et al. 2019). A few projects exhibited interesting trends such as high RAP project 8441 with elevated rut depth in year 1 when compared to the other projects, while showing similar rutting by year 3. Another parameter is the IRI of the high RAP project 8438 decreasing significantly 2 years after construction, which may be credited to any preventive maintenance activity.

Equivalent cracking percentages illustrated in Figure 46 for the first 3 years of pavement service life, highlight a tight range between 0.00% and 0.44% EC for high and low RAP projects. The field sections relative to the project 8441 with 34% RAP, exceeded this range significantly and obtained 4.09% and 17.34% EC at 3 years and 4 years after construction, respectively. The same project showed the highest cracking percentage for patching equivalent to 22.53% during its first year of service, which may indicate some distress driven by unknown parameters such as existing pavement condition prior to rehabilitation, plant product or construction challenges. It should be mentioned that the FMFC samples corresponding to contract 8441, were the only samples exceeding WSDOT criterion of 175 psi, which may be strongly correlated to the high EC% observed.

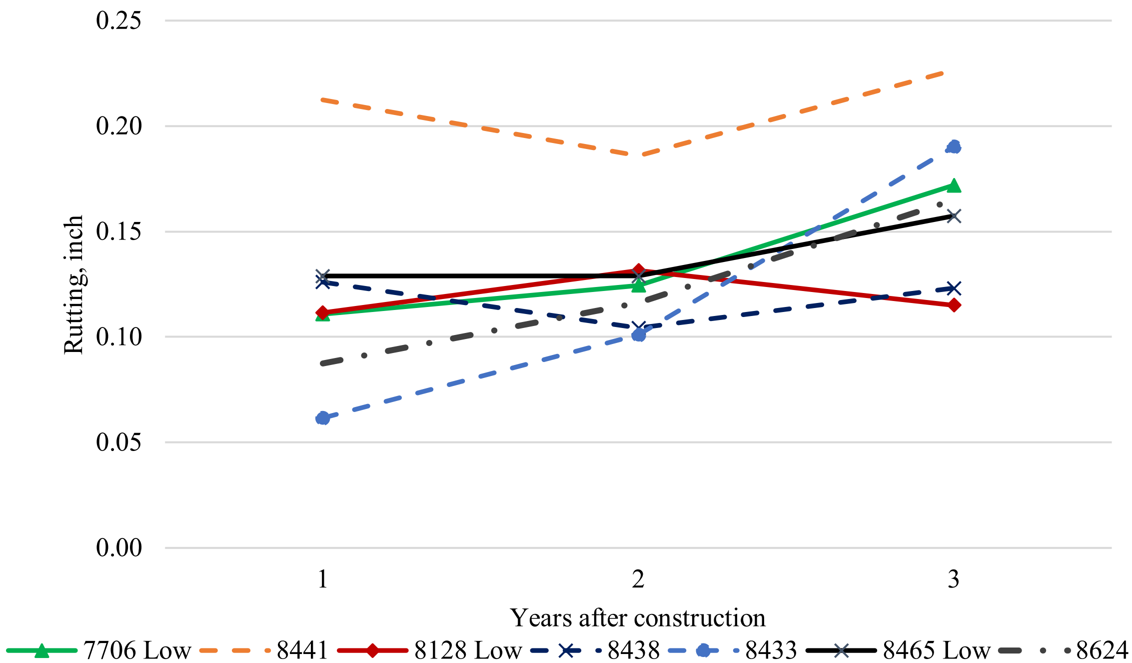


Figure 44: Rut Depth PMS Data

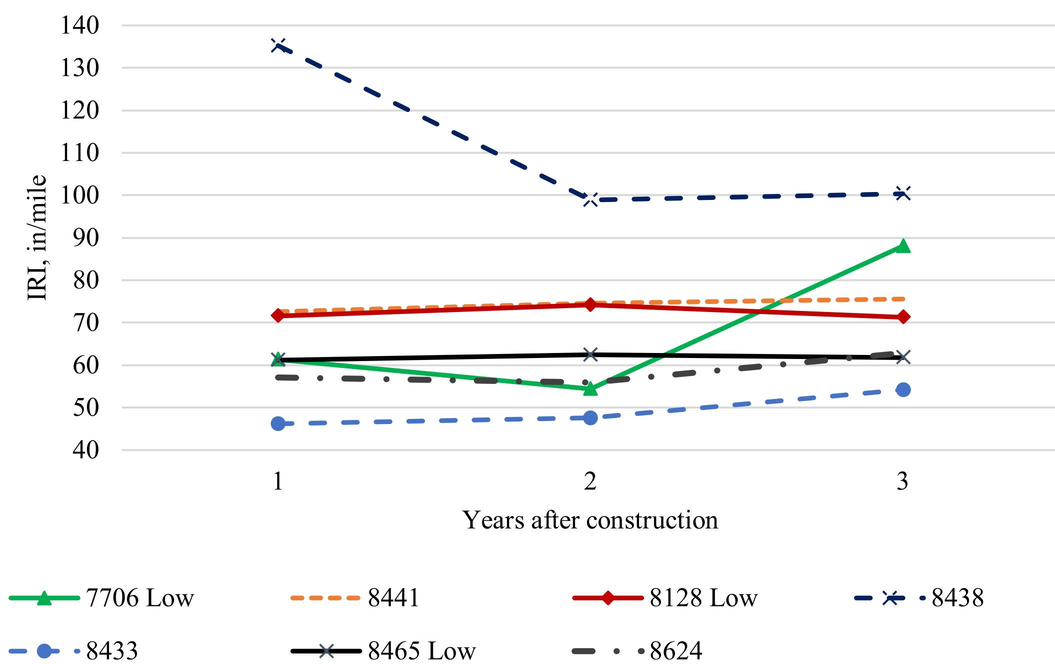


Figure 45: IRI PMS Data

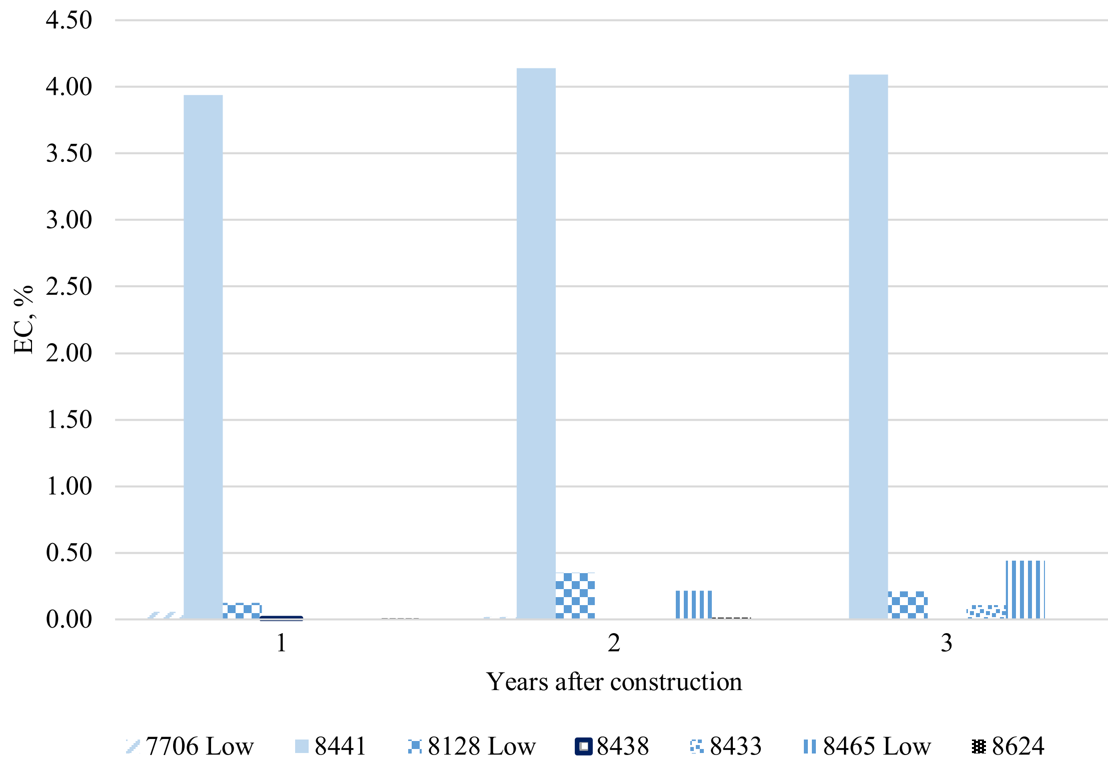


Figure 46: EC% PMS Data

8 STATISTICAL ANALYSIS

A statistical analysis was conducted on data presented in previous chapters, as shown in Figure 47 for LMLC samples, and Figure 48 for FMLC and FMFC samples. Predictor variables used in the statistical analysis were RAP dose, binder supplier, binder grade, material properties, and others. IBM® Statistical Analysis SPSS software was used to do one-way analysis of variance (ANOVA) or t-tests, which can capture statistically significant difference between two or more independent populations (IBM SPSS Statistics 2020). The t-test was used when comparing two populations, while ANOVA was done to compare three or more groups of data. The p-values were computed for each analysis, with an alpha value of 0.05, relative to 95% confidence level. A p-value greater than 0.05 indicates strong evidence supporting the null hypothesis, i.e., there is not statistically significant effect or difference, while rejecting the alternative hypothesis at 95% confidence.

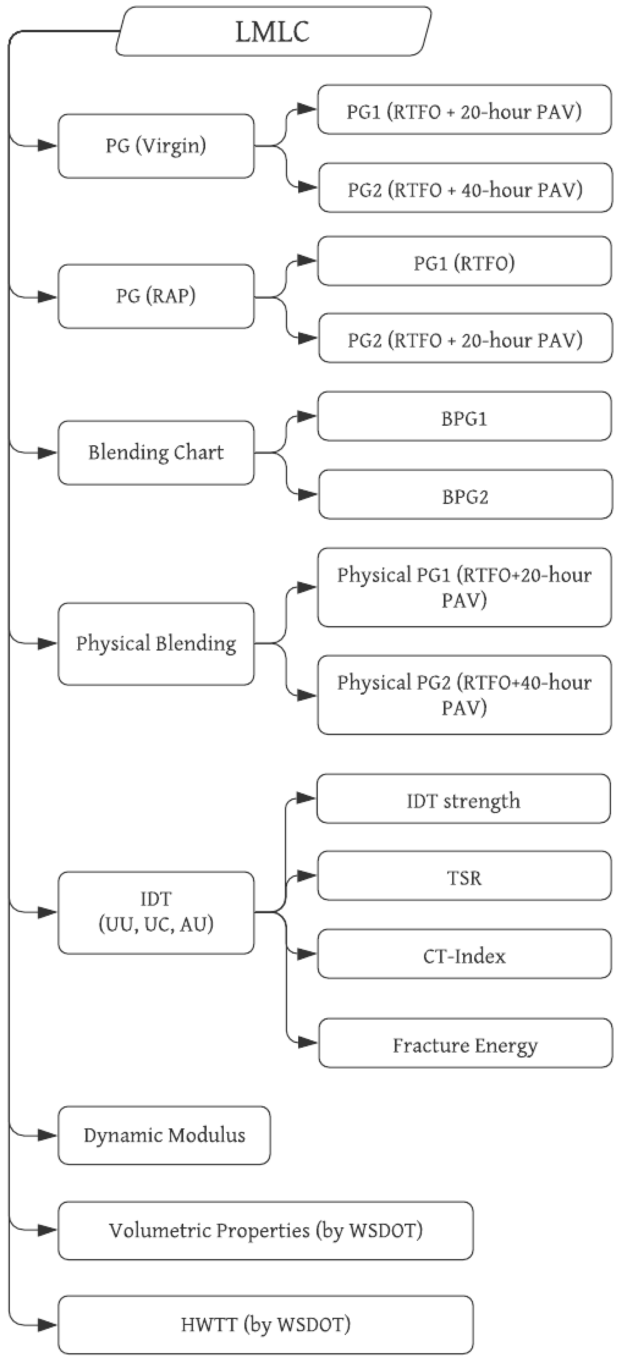


Figure 47: LMLC Experimental Data

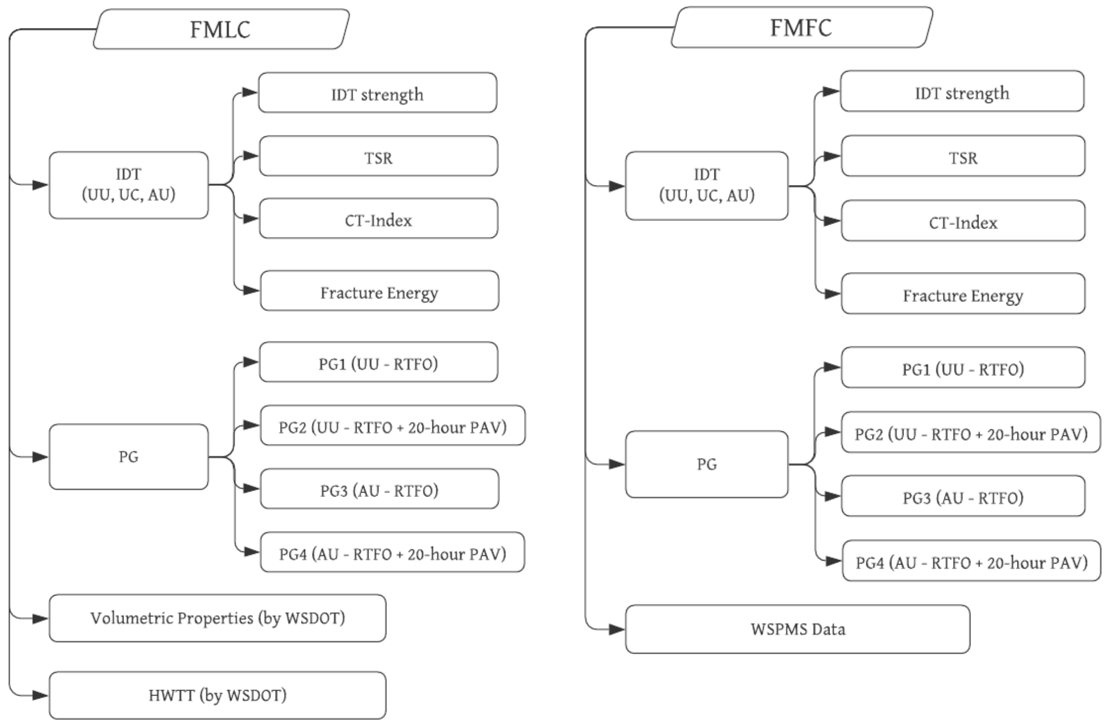


Figure 48: FMLC and FMFC Experimental Data

As part of NCHRP Project 09-57A, a ruggedness experiment for the CT-Index test was performed with statistical analysis of test results considering air voids, contact load, conditioning method, loading rate, specimen thickness, test temperature, and specimen center location as factors (NCHRP 09-57A 2019). Table 38 lists the factors included along with summary statistics. The results were analyzed separately for 12.5 mm SMA mixtures, 12.5 mm Superpave mixtures, and 9.5 mm Superpave mixtures (NCHRP 09-57A 2019). The statistical analysis emphasized the significance of air voids with a p-value ≤0.05 for all evaluated mix types.

Table 38: Statistical Significance of Effects for Ruggedness Testing of CT-Index Test (NCHRP 09-57A 2019)

Mix Type	Effect Order	Factor	|Effect|	Student's t	p-value	Half-Normal
12.5 mm SMA	7	Air Voids	207.29	2.91	0.023	1.803
	6	Contact Load	93.21	1.31	0.232	1.242
	5	Conditioning Method	85.72	1.20	0.268	0.921
	4	Loading rate	77.23	1.08	0.315	0.674
	3	Specimen thickness	48.02	0.67	0.522	0.464
	2	Test Temperature	45.69	0.64	0.542	0.272
	1	Specimen center location	31.62	0.44	0.671	0.090
12.5 mm Superpave Mixture	7	Air Voids	31.26	6.17	0.000	1.803
	6	Contact Load	9.10	1.80	0.116	1.242
	5	Conditioning Method	5.26	1.04	0.334	0.921
	4	Loading rate	3.18	0.63	0.550	0.674
	3	Specimen thickness	3.00	0.59	0.572	0.464
	2	Test Temperature	2.38	0.47	0.653	0.272
	1	Specimen center location	1.65	0.33	0.754	0.090
9.5 mm Superpave Mixture	7	Air Voids	50.32	6.67	0.000	1.803
	6	Contact Load	10.09	1.34	0.223	1.242
	5	Conditioning Method	7.47	0.99	0.355	0.921
	4	Loading rate	2.98	0.40	0.705	0.674
	3	Specimen thickness	2.94	0.39	0.708	0.464
	2	Test Temperature	2.32	0.31	0.767	0.272
	1	Specimen center location	0.67	0.09	0.932	0.090

As show in Table 39, the researchers concluded that air voids level is the only factor among the aforementioned seven factors that significantly affects the CT-Index test results and recommended keeping the current air void tolerance of ±0.5 %.

Table 39: Recommended Tolerance for the Seven Factors (NCHRP 09-57A 2019)

Factor	Tolerance used in NCHRP 09-57A	Current Requirement in ASTM WK60859	Recommended Tolerance
Specimen thickness (mm)	±2	±1	±1
Specimen location (mm)	Center or 2 mm offset	Centered in the fixture	Centered in the fixture
Air Voids (%)	±1.0	±0.5	±0.5
Loading rate (mm/min)	±2	±2	±2
Contact load (kN)	0.1 or 0	0	0
Test temperature (°C)	±1	±1	±1
Conditioning method	Air or Water	Air or Water	Air or Water

The following sections primarily focus on LMLC sample results, rather than FMLC and FMFC sample results, due to better control of sample preparation. The variability in air voids of field samples can strongly affect performance test results.

8.1 Rheological Parameters from Blending Charts versus Physically Blended Samples

A paired t-test was adopted to determine if any significant differences between rheological properties from blending charts and physically blended binder test results existed. Interestingly, the data in Table 40 captured a unique significant difference in ΔTc parameter, with a p-value of 0.02 (≤0.05), which suggests uncertainty in predicting the ΔTc parameter from blending charts. Differences ranged from -2.7 °C and +0.3°C, and in general, physically blended binders had lower (more negative) ΔTc values than those calculated from blending charts with an average difference of -0.9°C.

Table 40: Physical Blending Statistical Analysis

Blended Binder ID	Continuous High PG		Continuous Low PG		ΔTc, °C	
	Physical PG1	BPG2	Physical PG1	BPG2	Physical PG1	BPG2
9145-20-5834-S2	66.4	67.1	29.0	31.0	-6.0	-3.8
9145-25-5834-S2	66.6	68.4	27.7	30.0	-6.6	-3.9
9145-20-6428-S2	72.4	73.9	26.9	27.0	-2.7	-1.5
9262-20-58H22-S4	73.0	72.6	20.7	20.5	-3.1	-2.9
9262-25-58H22-S4	74.6	74.2	19.4	19.6	-3.7	-3.1
9262-36-58S28-S4	72.3	71.7	23.4	22.7	-2.7	-2.5
9231-20-58H22-S5	63.4	63.3	30.3	29.7	-1.1	-0.8
9231-25-58H22-S5	64.6	64.4	29.7	29.0	-1.5	-1.0
9231-40-52S28-S5	67.6	67.3	28.0	27.7	-1.9	-2.2
9231-40-52S28-S3	66.4	66.2	27.5	26.9	-4.1	-3.2
Average	68.73	68.91	26.26	26.41	-3.3	-2.5
p-value, 95% CI	0.52		0.68		0.02	

8.2 Impact of Binder Supplier

Raw material properties can play a major role in overall mixture performance, specifically in IDT strength test results. Therefore, the impact of binder supplier was analyzed based on two LMLC mixtures: 9231-40-52S28-S3+Z and 9231-40-52S28-S5+Z, having the same binder grades from two different sources. Table 41 shows a significant difference between both suppliers S3 and S5. US. Oil exhibits a statistically lower fracture energy for AU and UU samples, as well as a lower AU IDT strength value, and a lower UU CT-Index. It can be inferred that the binder source or supplier has a significant influence on cracking tests properties, under AU and UU conditions.

Table 41: Statistical Analysis of Binder Supplier Impact

Mixture Comparison	Aging Condition	Mix Properties	Significant at 0.05 alpha level?
9231-40-52S28-S3+Z and 9231-40-52S28-S5+Z	AU	CT-Index	NS
		Fracture Energy	L
		IDT strength	L
	UU	Dynamic Modulus	NS
		CT-Index	L
		Fracture Energy	L
		IDT strength	NS
	UC	CT-Index	NS
		Fracture Energy	NS
		IDT strength	NS

Note: NS: Not Significantly Different, L: Significantly Lower, H: Significantly Higher

8.3 Variability between LMLC and FMLC Cracking Test Results

It is not uncommon to observe differences in the properties of LMLC and FMLC samples (Mohammad et al. 2016). LMLC and FMLC cracking test results were statistically compared. The loose field mixture received for contract 9145 was compared with the same mix design replicated in the laboratory 9145-0-6428-S2, and with 9145-20-6428-S2 LMLC samples, since WSDOT allows incorporation of up to 20% RAP in the mixture without including the RAP in the mix design. Both contracts 9262 and 9229 refer to the same mix design ID, so that field mixture received from the different plants were compared relative to the same mix design by WSDOT. The statistical analysis is summarized in Table 42 and Table 43. The FMLC mixtures had statistically significant differences in IDT strength test parameters, compared to the mix design. Likewise, the 9231 contract FMLC had some significantly different parameters when compared to LMLC samples, within the same material properties and mix designs, which highlights on the effect of plant variability on some mixture properties. Interestingly, the IDT strength parameter was able to depict significant differences between field and laboratory replicated mixtures, while it was always greater for the field mixture. Even though some mixture properties were insignificantly different between LMLC and FMLC specimens, the statistical analysis suggests that plant production should be always controlled not to exceed the tolerance limits of variability between LMLC and FMLC, while calibrating the laboratory specification as per these allowable variabilities.

Table 42: FMLC and LMLC Comparison for 9145 and 9262 Contracts

Mixture Comparison	Aging Condition	Mix Properties	Significant at 0.05 alpha level?
9145 (FMLC) and 9145-0-6428-S2	AU	CT-Index	NS
		Fracture Energy	NS
		IDT strength	H
	UU	CT-Index	NS
		Fracture Energy	H
		IDT strength	H
	UC	CT-Index	NS
		Fracture Energy	NS
		IDT strength	NS
9145 (FMLC) and 9145-20-6428-S2	AU	CT-Index	NS
		Fracture Energy	H
		IDT strength	NS
	UU	CT-Index	NS
		Fracture Energy	H
		IDT strength	NS
	UC	CT-Index	NS
		Fracture Energy	NS
		IDT strength	L
9262 (FMLC) and 9262-36-58S28-S4+RA	AU	CT-Index	L
		Fracture Energy	L
		IDT strength	NS
	UU	CT-Index	NS
		Fracture Energy	NS
		IDT strength	H
	UC	CT-Index	L
		Fracture Energy	H
		IDT strength	H
9229 (FMLC) and 9262-36-58S28-S4+RA	AU	CT-Index	N/A
		Fracture Energy	N/A
		IDT strength	N/A
	UU	CT-Index	NS
		Fracture Energy	H
		IDT strength	H
	UC	CT-Index	NS
		Fracture Energy	H
		IDT strength	H

Note: NS: Not Significantly Different, L: Significantly Lower, H: Significantly Higher
N/A: Not applicable due to lack of materials received

Table 43: FMLC and LMLC Comparison for 9231 Contract

Mixture Comparison	Aging Condition	Mix Properties	Significant at 0.05 alpha level?
9231 (FMLC) and 9231-40-52S28-S5+Z	AU	CT-Index	NS
		Fracture Energy	NS
		IDT strength	NS
	UU	CT-Index	L
		Fracture Energy	H
		IDT strength	H
	UC	CT-Index	NS
		Fracture Energy	NS
		IDT strength	H

Note: NS: Not Significantly Different, L: Significantly Lower, H: Significantly Higher

8.4 LMLC Comparison of 20% and 25% RAP Mixtures

Since WSDOT is considering the transition from 20% to 25% RAP mixtures, ANOVA analysis was conducted on all the mixtures with 20% and 25% RAP at 95% confidence level. Binder properties results are presented in Table 44, and mixture property results are presented in Table 45. Neither binder parameters nor mixture cracking test properties considered for LMLC specimens were significantly different between 20% and 25% RAP (i.e., all p-values > 0.05).

Table 44: Binder Properties Comparison at 20% and 25% RAP

Binder Properties		20% RAP									25% RAP								
		9145-20-6428-S1	9145-20-6428-S2	9145-20-5834-S2	9262-20-58H22-S3	9262-20-58H22-S4	9262-20-58S28-S4	9231-20-58H22-S3	9231-20-58H22-S5	9231-20-52H28-S5	9145-25-6428-S1	9145-25-6428-S2	9145-25-5834-S2	9262-25-58H22-S3	9262-25-58H22-S4	9262-25-58S28-S4	9231-25-58H22-S3	9231-25-58H22-S5	9231-25-52H28-S5
High BPG1	Mean	66									68								
	p-value, 95% CI	0.33																	
Low BPG1	Mean	22									22								
	p-value, 95% CI	1.00																	
Low BPG2	Mean	22									22								
	p-value, 95% CI	1.00																	
ΔTc, BPG1, ℃	Mean	-1.69									-1.81								
	p-value, 95% CI	0.85																	
ΔTc, BPG2, ℃	Mean	-2.12									-2.32								
	p-value, 95% CI	0.76																	

Table 45: LMLC Mixture Properties Comparison at 20% and 25% RAP

Mixture Properties		20% RAP					25%RAP			
		9145-20-6428-S2	9145-20-5834-S2	9262-20-58H22-S4	9231-20-58H22-S5+Z	9231-20-52H28-S5+Z	9145-25-6428-S2	9145-25-5834-S2	9262-25-58H22-S4	9231-25-58H22-S5+Z
IDT Strength, UU, psi	Mean	118					129			
	p-value, 95% CI	0.69								
IDT Strength, UC, psi	Mean	93					100			
	p-value, 95% CI	0.73								
IDT Strength, AU, psi	Mean	151					155			
	p-value, 95% CI	0.90								
TSR, %	Mean	79					79			
	p-value, 95% CI	0.98								
CT-Index, UU	Mean	113					90			
	p-value, 95% CI	0.36								
CT-Index, UC	Mean	280					207			
	p-value, 95% CI	0.15								
CT-Index, AU	Mean	62					52			
	p-value, 95% CI	0.54								
Fracture Energy, UU, J/m2	Mean	7479					7749			
	p-value, 95% CI	0.87								
Fracture Energy, UC, J/m2	Mean	7867					8025			
	p-value, 95% CI	0.92								
Fracture Energy, AU, J/m2	Mean	7901					7814			
	p-value, 95% CI	0.95								
E*, 10Hz, 4°C, ksi	Mean	1700					1888			
	p-value, 95% CI	0.44								
E*, 10Hz, 20°C, ksi	Mean	777					923			
	p-value, 95% CI	0.43								
E*, 10Hz, 35°C or 40°C, ksi	Mean	248					299			
	p-value, 95% CI	0.50								

8.5 LMLC Performance with Varying RAP Amounts

It is well known that mixture cracking resistance is highly influenced by the RAP dosage integrated within a mixture. UU IDT strength values are plotted in Figure 49 as a function of RAP dosage and virgin binder grades along with the current WSDOT 175 psi maximum criteria. The plot explains how adding 20% and 25% RAP stiffens the mix and increases the IDT strength value, however softening the virgin binder can mitigate this additional stiffness even with high RAP % (36% to 40%). It should be mentioned that all the mixture combinations evaluated, met WSDOT criteria for a maximum dry strength of 175 psi, except for contract 9262/9229 with 25% RAP. Additionally, Table 46 and Table 47 summarize the statistical comparison between different mixture combinations and led to the following conclusions considering all the mixture properties tabulated:

- 0% and 20% RAP (with same virgin binder grade): 43% of the mixture properties were significantly different.
- 0% and 25% RAP (with same virgin binder grade): 67 % of the mixture properties were significantly different.
- 20% and 25% RAP (with same virgin binder grade): 7 % of the mixture properties were significantly different.
- High RAP Mixtures (with softer binder) and 0% RAP Mixtures: 55% of the mixture properties were significantly different.
- High RAP Mixtures (with softer binder) and 20% RAP Mixtures: 75% of the mixture properties were significantly different.
- High RAP Mixtures (with softer binder) and 25% RAP Mixtures: 75% of the mixture properties were significantly different.

- The IDT strength value captured most of the statistically significant differences at various RAP doses, exceptionally between 20% and 25% RAP mixtures.
- When identifying statistically significant differences, the IDT strength parameter was the most consistent with the dynamic modulus (E*) engineering property determined under axial dynamic loading, rather than monotonic indirect tension loading.

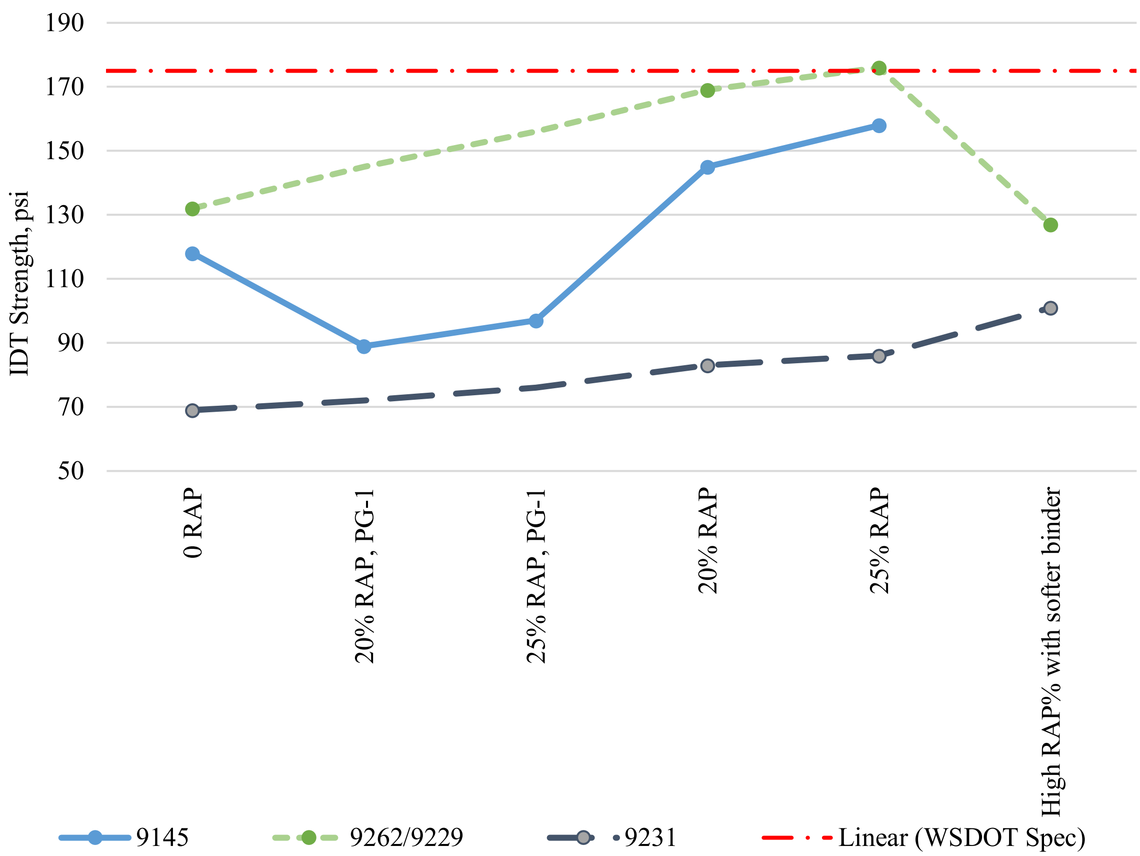


Figure 49: LMLC UU IDT Strength with RAP%

Table 46: LMLC Performance Comparison at 0%, 20%, and 25% RAP

Mixture Comparison	Aging Condition	Mix Properties	Significant at 0.05 alpha level?		
			Contract 9145	Contract 9262	Contract 9231
0% RAP Mixtures and 20% RAP Mixtures	AU	CT-Index	NS	H	NS
		Fracture Energy	NS	NS	L
		IDT strength	L	L	L
	UU	Dynamic Modulus	L	L	L
		CT-Index	NS	NS	NS
		Fracture Energy	NS	L	NS
		IDT strength	NS	L	L
	UC	CT-Index	NS	NS	NS
		Fracture Energy	NS	L	NS
		IDT strength	NS	L	NS
0% RAP Mixtures and 25% RAP Mixtures	AU	CT-Index	NS	H	NS
		Fracture Energy	NS	NS	L
		IDT strength	L	L	L
	UU	Dynamic Modulus	L	L	L
		CT-Index	NS	NS	NS
		Fracture Energy	L	L	L
		IDT strength	L	L	L
	UC	CT-Index	NS	NS	H
		Fracture Energy	NS	L	L
		IDT strength	L	L	L
20% RAP Mixtures and 25% RAP Mixtures	AU	CT-Index	NS	NS	NS
		Fracture Energy	NS	NS	NS
		IDT strength	L	NS	NS
	UU	Dynamic Modulus	NS	NS	NS
		CT-Index	NS	NS	NS
		Fracture Energy	NS	NS	NS
		IDT strength	NS	NS	NS
	UC	CT-Index	NS	NS	NS
		Fracture Energy	NS	NS	NS
		IDT strength	NS	NS	L

Note: NS: Not Significantly Different, L: Significantly Lower, H: Significantly Higher

Table 47: LMLC Performance Comparison at High RAP

Mixture Comparison	Aging Condition	Mix Properties	Significant at 0.05 alpha level?		
			Contract 9145	Contract 9262	Contract 9231
High RAP Mixtures and 0% RAP Mixtures	AU	CT-Index	N/A	NS	L
		Fracture Energy	N/A	NS	H
		IDT strength	N/A	L	H
	UU	Dynamic Modulus	N/A	H	H
		CT-Index	N/A	NS	NS
		Fracture Energy	N/A	NS	H
		IDT strength	N/A	NS	H
	UC	CT-Index	N/A	NS	NS
		Fracture Energy	N/A	H	H
		IDT strength	N/A	NS	H
High RAP Mixtures and 20% RAP Mixtures	AU	CT-Index	N/A	NS	L
		Fracture Energy	N/A	L	H
		IDT strength	N/A	L	H
	UU	Dynamic Modulus	N/A	L	H
		CT-Index	N/A	NS	NS
		Fracture Energy	N/A	L	H
		IDT strength	N/A	L	H
	UC	CT-Index	N/A	NS	NS
		Fracture Energy	N/A	L	H
		IDT strength	N/A	L	H
High RAP Mixtures and 25% RAP Mixtures	AU	CT-Index	N/A	H	NS
		Fracture Energy	N/A	L	H
		IDT strength	N/A	L	H
	UU	Dynamic Modulus	N/A	L	H
		CT-Index	N/A	NS	NS
		Fracture Energy	N/A	L	NS
		IDT strength	N/A	L	H
	UC	CT-Index	N/A	H	NS
		Fracture Energy	N/A	L	H
		IDT strength	N/A	L	H

Note: NS: Not Significantly Different, L: Significantly Lower, H: Significantly Higher
N/A: Not applicable

8.6 Effect of Softer Virgin Binder with 20% and 25% RAP

Using softer virgin binders may allow the integration of higher RAP amounts without compromising mixture performance. The effect of one grade drop (PG-1) with 20% and 25% RAP is presented in Table 48. It should be noted that the statistical analysis presented herein, relies on what WSDOT currently considers as PG-1, i.e., the binder supplier grade, in order to evaluate the impact of using PG-1 binder with 20% RAP and 25% RAP. In other words, this study includes PG6428-S2 versus PG5834-S2 (PG-1) for contract 9145, and PG58H22-S5 versus PG52H28-S5 (PG-1) for contract 9231. However, the true grade of the PG52H28-S5 turned out to be 64.0-32.7, when graded in UNR laboratories as presented in section 6.1.1. As per Table 48 results for contract 9145, virgin mixtures exhibit significantly higher CT-Index, fracture energy, IDT strength values, and dynamic modulus than 20% and 25% RAP mixtures with PG-1. The opposite was observed in contract 9231, this could be justified by the true grading previously mentioned. The PG52H28-S5 binder used by WSDOT as a PG-1, has unexpectedly higher IDT strength, CT-Index and fracture energy than the original binder PG58H22-S5.

Table 48: Effect of a Softer Virgin Binder on LMLC Performance

Mixture Comparison	Aging Condition	Mix Properties	Significant at 0.05 alpha level?		
			Contract 9145	Contract 9262	Contract 9231
0% RAP Mixtures with 20% RAP Mixtures with PG-1	AU	CT-Index	H	N/A	H
		Fracture Energy	H	N/A	L
		IDT strength	H	N/A	L
	UU	Dynamic Modulus	H	N/A	L
		CT-Index	H	N/A	L
		Fracture Energy	H	N/A	L
		IDT strength	H	N/A	L
	UC	CT-Index	H	N/A	L
		Fracture Energy	H	N/A	L
		IDT strength	H	N/A	L
0% RAP Mixtures with 25% RAP Mixtures with PG-1	AU	CT-Index	H	N/A	N/A
		Fracture Energy	H	N/A	N/A
		IDT strength	H	N/A	N/A
	UU	Dynamic Modulus	NS	N/A	N/A
		CT-Index	H	N/A	N/A
		Fracture Energy	H	N/A	N/A
		IDT strength	H	N/A	N/A
	UC	CT-Index	H	N/A	N/A
		Fracture Energy	H	N/A	N/A
		IDT strength	H	N/A	N/A

Note: NS: Not Significantly Different, L: Significantly Lower, H: Significantly
N/A: Not applicable

8.7 IDT Strength Criteria

Current WSDOT standard specifications for IDT strength is 175 psi maximum on unaged unconditioned (UU) samples. Accordingly, a correlation analysis was performed between all the parameters generated from IDT strength tests on LMLC samples, including maximum IDT strength value, CT-Index, and fracture energy, under UU and AU conditions. The highest correlation observed in Table 49 for UU IDT, was equal to 0.971 with AU IDT, followed by 0.965 with the UU fracture energy. With respect to CT-Index, the highest correlation was equivalent to -0.494 between UU IDT strength, and AU CT-Index. A preliminary criterion for AU IDT could be amplified by 30%, based on the average increase of LMLC samples in section 6.1.6. Accordingly, the preliminary AU IDT strength criterion would be a maximum of 228 psi.

Table 49: LMLC IDT Strength Test Parameters Correlation

	IDT Strength, UU	*IDT Strength, AU*	*CT-Index, UU*	*CT-Index, AU*	*Fracture Energy, UU*	*Fracture Energy, AU*
IDT Strength, UU	1					
IDT Strength, AU	**0.971**	1				
CT-Index, UU	-0.483	-0.310	1			
CT-Index, AU	-0.494	-0.377	0.892	1		
Fracture Energy, UU	**0.965**	0.988	-0.254	-0.287	1	
Fracture Energy, AU	0.835	0.917	0.041	0.002	0.942	1

8.8 ΔTc Regression with LMLC Performance

A linear regression was fitted between ΔTc and IDT strength test parameters for LMLC samples. Figure 50 through Figure 55 illustrate the different regressions with IDT strength value, CT-Index, and fracture energy under UU and AU conditions, along with the R-square for different binder grades. It should be mentioned that the regression analysis was done between the samples with similar aging conditions (i.e., BPG1 with UU specimens, and BPG3 with AU specimens). The aging conditions of BPG1 and BPG3, were previously described in the blending charts section 6.1.3. The plotted regressions can be analyzed by means of R-square value, which reflects how much of data variability was explained in the regression model.

Figure 50 and Figure 51 show that ΔTc of BPG1 correlates well with UU IDT strength with an R-square of 0.87, which is slightly lower than the R-square of the regression between ΔTc of BPG3 and AU IDT strength, of 0.88. Figure 52 and Figure 53 show that CT-index exhibited the lowest correlation with ΔTc, showing an R-square of 0.52 and 0.39 with UU and AU, respectively. Interestingly, the highest correlation observed was associated with the fracture energy, having UU and AU R-square values of 0.93 and 0.99, respectively as shown in Figure 54 and Figure 55, respectively. These observations suggest that the ΔTc parameter correlated better to the fracture energy under both aging conditions evaluated, followed by the IDT strength values, and then CT-Index. In the series of plots that follow, ΔTc showed a negative trend with IDT strength values and fracture energy, while showing a positive trend with the CT-Index. In general, all the evaluated binders supplied to WSDOT exhibited a relatively good resistance to aging related cracking and at the same time had relatively good ΔTc values.

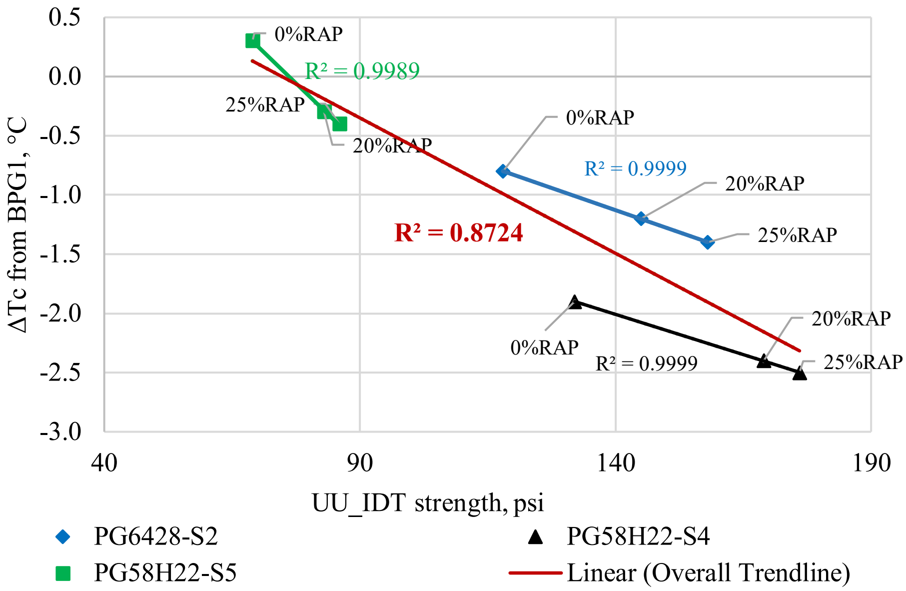


Figure 50: ΔTc versus UU LMLC IDT Strength

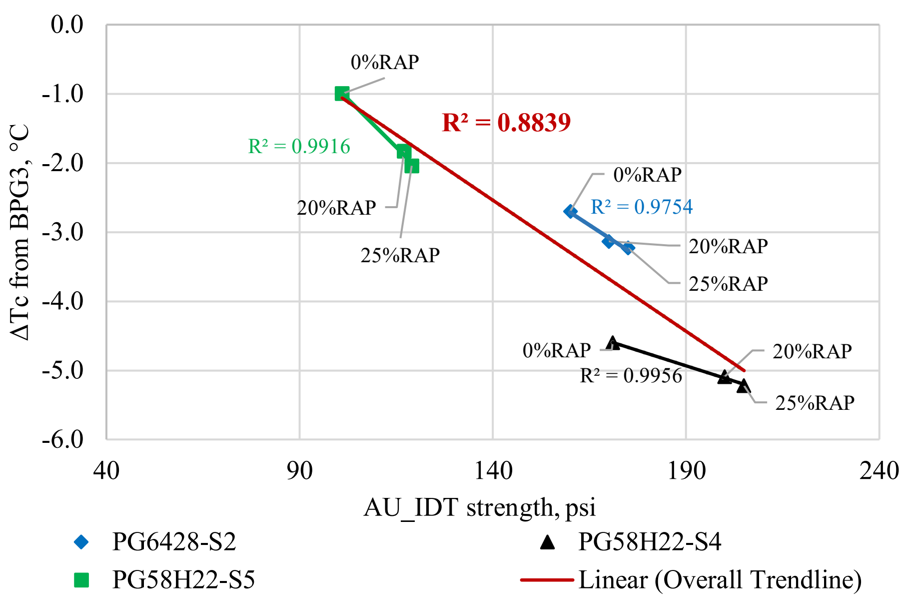


Figure 51: ΔTc versus AU LMLC IDT Strength

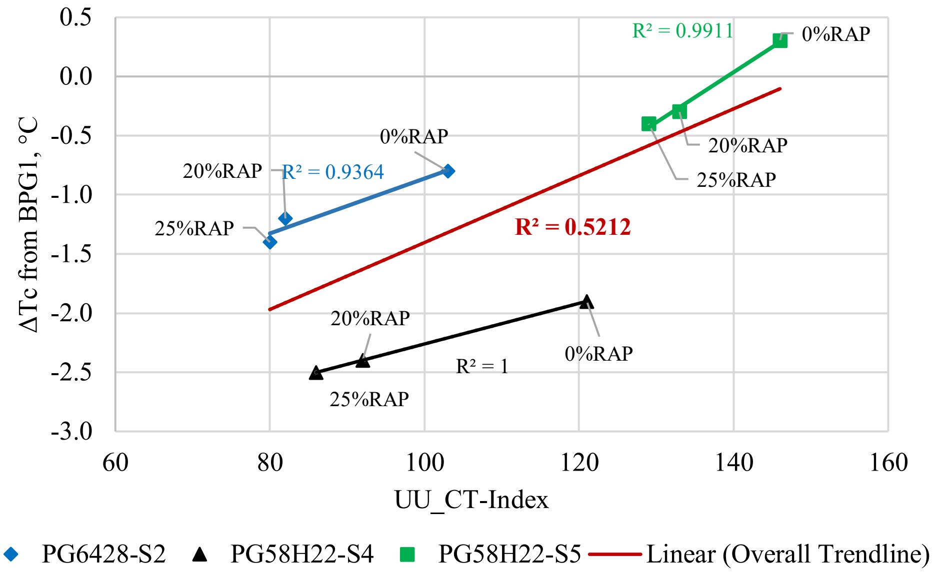


Figure 52: ΔTc versus UU LMLC CT-Index

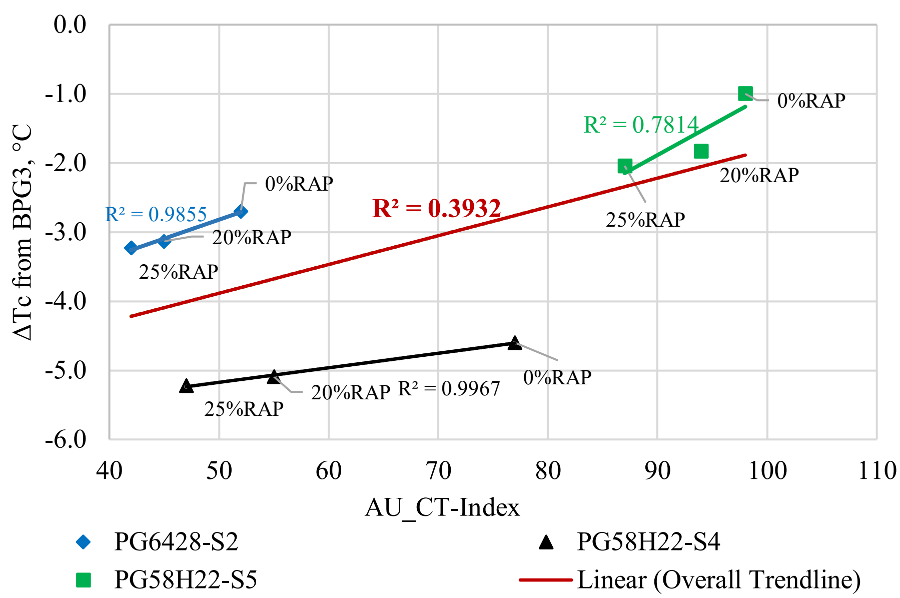


Figure 53: ΔTc versus AU LMLC CT-Index

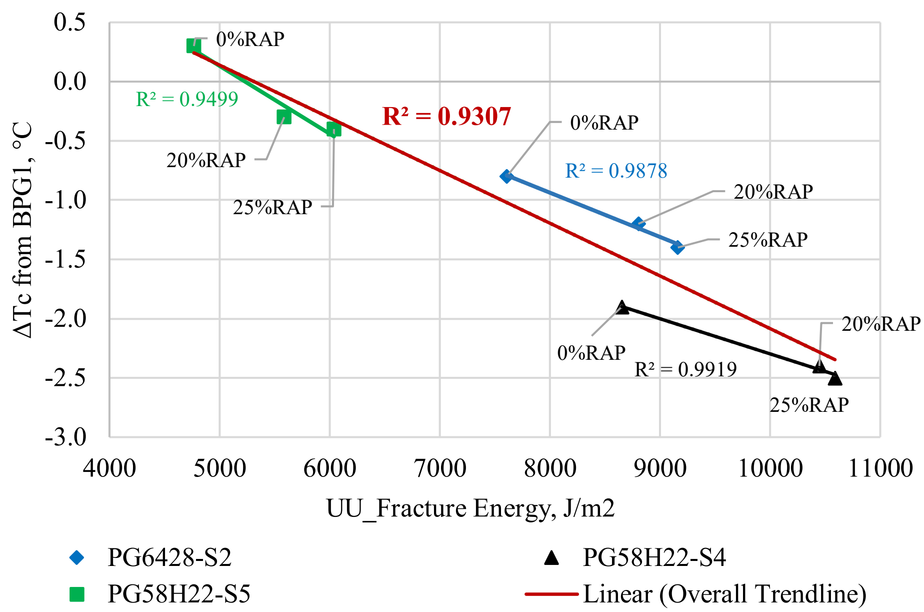


Figure 54: ΔTc versus UU LMLC Fracture Energy

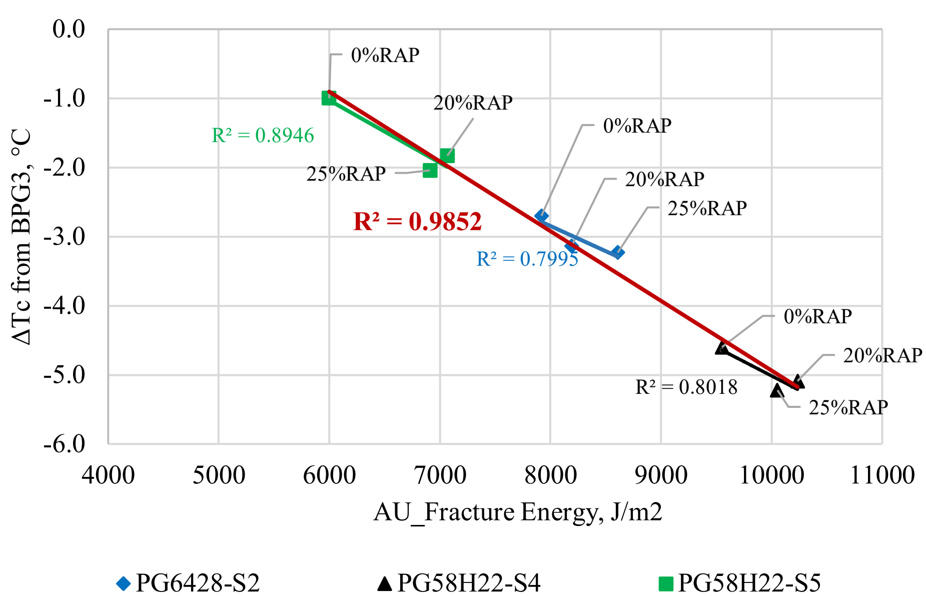


Figure 55: ΔTc versus AU LMLC Fracture Energy

8.9 ΔTc Regression with FMFC Performance

Similar to the LMLC sample regression, linear regressions were plotted in Figure 56 through Figure 61, between ΔTc of recovered binder, and FMFC IDT test parameters. The regressions with field cores had considerably lower R-square values, due to the high variability within FMFC samples. The regression analyses were conducted between the samples with similar aging conditions (i.e., PG1 with UU specimens, and PG3 with AU specimens). The aging conditions of PG1 and PG3, were previously described in the FMFC section 6.3.2. The ΔTc values of contract 8433 were very low relative to other seven contracts, so they were considered outliers and excluded from the regression graphs. Note that the values of this contract are shown on the figures via red square symbols. Considering that WSDOT current specifications require UU IDT strength value, Figure 56 represents the regression of ΔTc from PG1 with UU IDT results, that had a low R-square of 0.04. The regression between ΔTc from PG3 and AU IDT strength values, had a significantly higher R-square of 0.43 shown in Figure 57. The regression of the CT-index parameter for UU and AU samples was very weak based on their minimal R-square, as presented in Figure 58 and Figure 59. Subsequently, Figure 60 and Figure 61 show weak correlations with the fracture energy, with R-square values of 0.21 and 0.23 for UU and AU samples, respectively. Despite the overall high variability among the evaluated field cores, the regression analysis indicates that the AU IDT strength value is best correlated with ΔTc of PG3 among all IDT test results on FMFC specimens. The weak correlations observed denote that the ΔTc parameter is a better indicator for aging susceptibility, rather than a cracking index, as indicated by the Asphalt Institute and the on-going NCHRP Project 09-60 as well (Asphalt Institute 2019, Planche 2020).

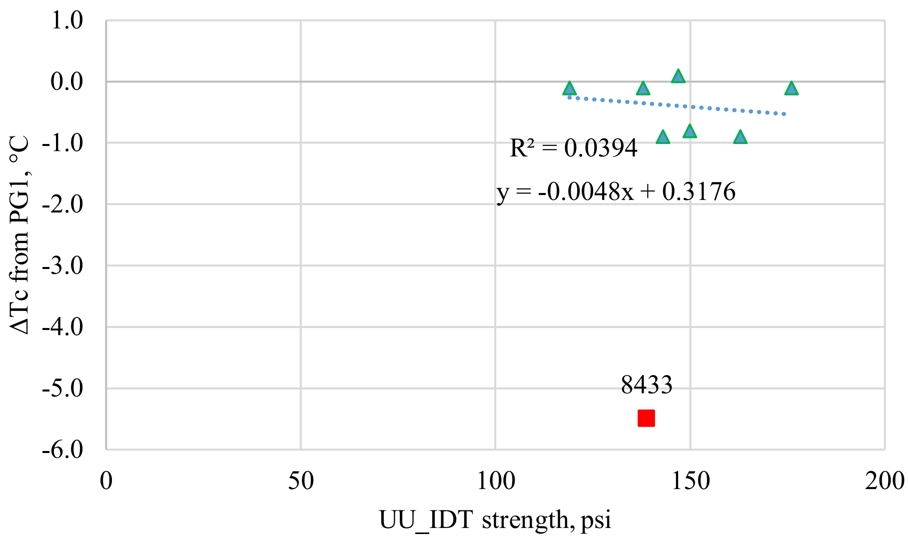


Figure 56: ΔTc PG1 versus UU FMFC IDT Strength

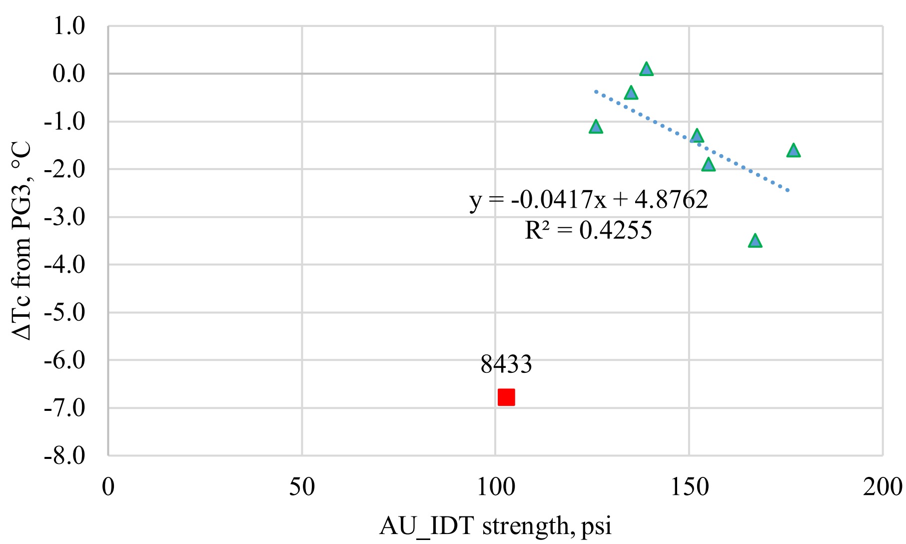


Figure 57: ΔTc PG3 versus AU FMFC IDT Strength

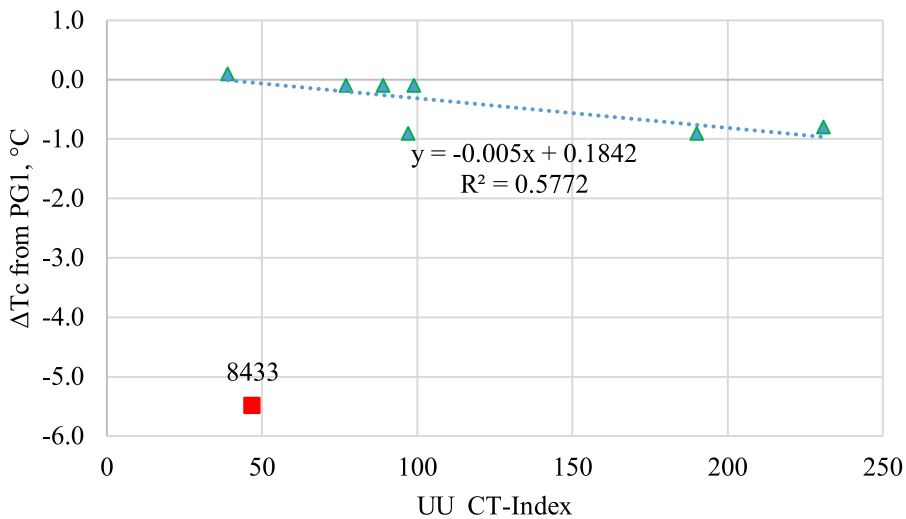


Figure 58: ΔTc PG1 versus UU FMFC CT-Index

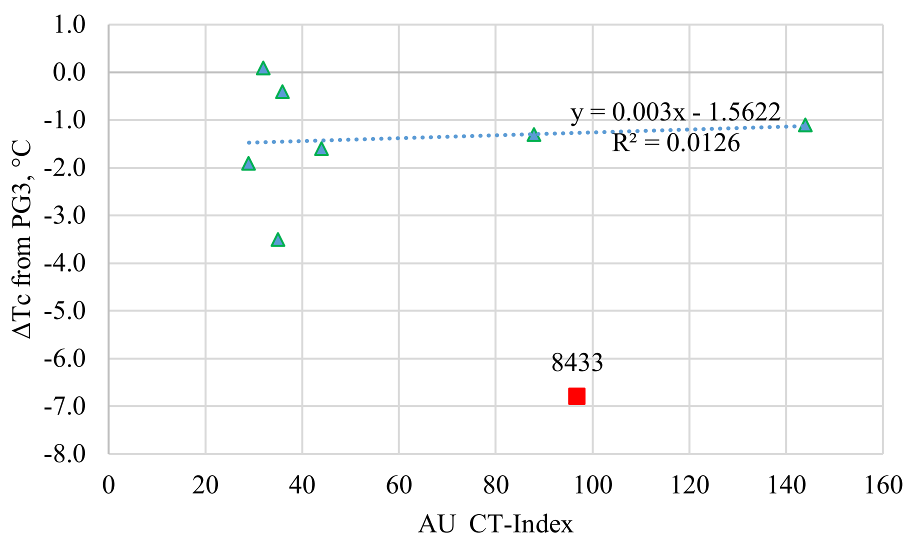


Figure 59: ΔTc PG3 versus AU FMFC CT-Index

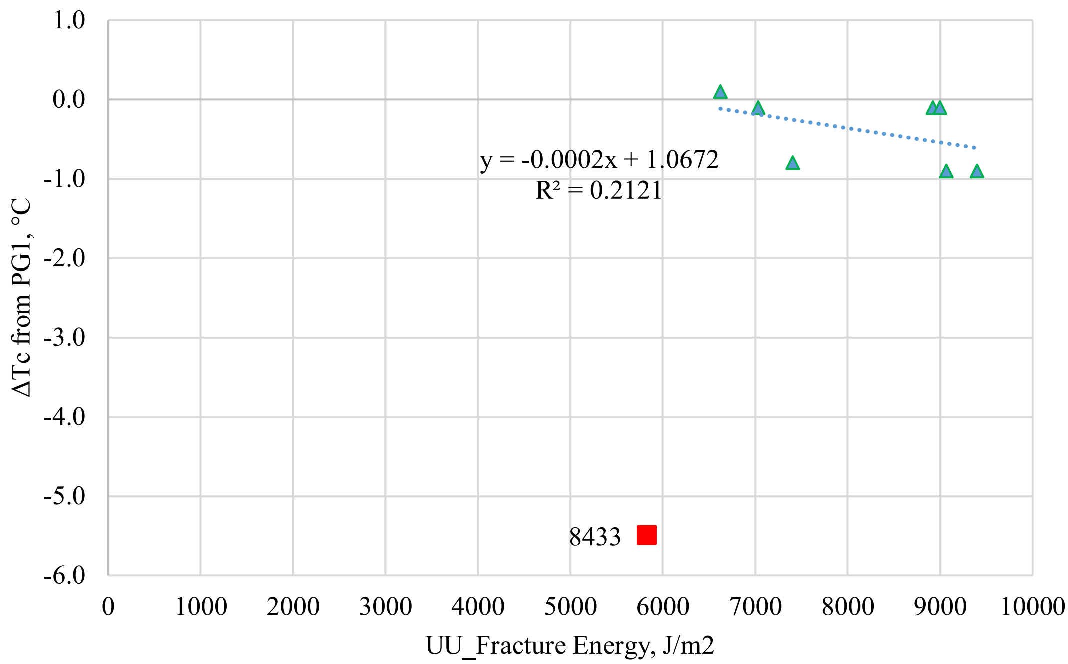


Figure 60: ΔTc PG1 versus UU FMFC Fracture Energy

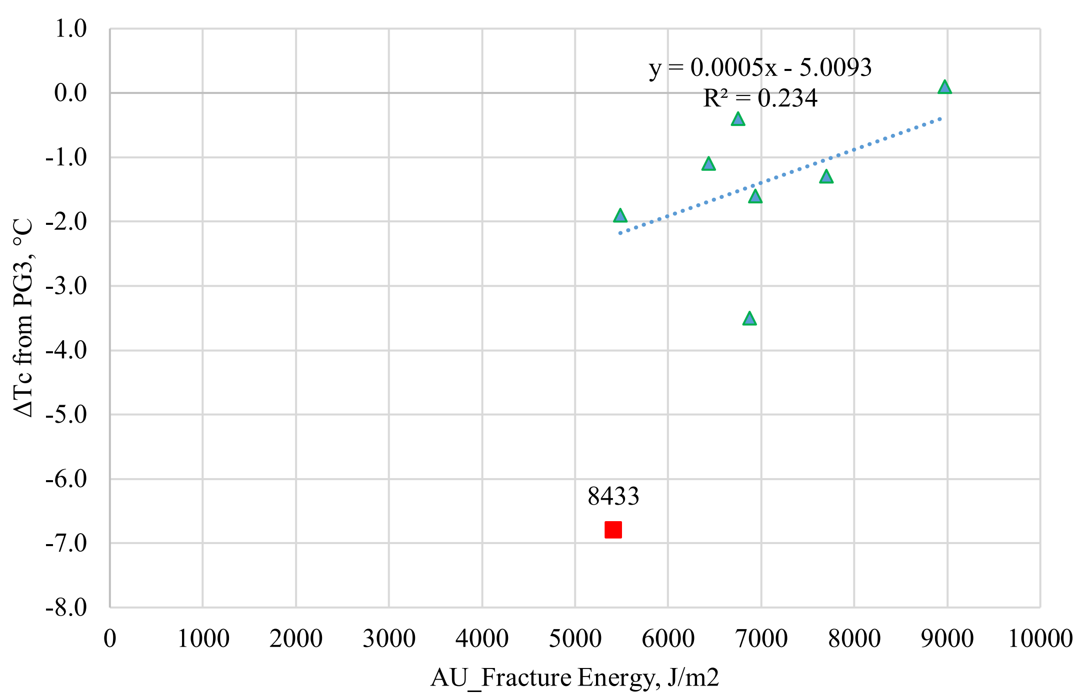


Figure 61: ΔTc PG3 versus AU FMFC Fracture Energy

8.10 PMS Data Regression with FMFC laboratory Results

In order to identify material specification criteria, correlation between pavement condition data and material properties of field pavements is needed. FMFC laboratory test results were correlated with the PMS data for the same projects, in terms of ΔTc and IDT test parameters. The laboratory data of FMFC specimens collected in 2018 were correlated to the EC% of the same year. Note that the EC% values observed among the contracts were extremely low, actually less than 1.2%. So, there was not an adequate range of EC% values to develop reliable regression analyses. The EC scale ranged between 0 and 40, since 40% EC would result in Pavement Structural Condition (PSC) equivalent to 0, as per WSDOT PMS 1993 update (Kay et al. 1993). The regression analyses were performed and as expected the observed models were not reliable as illustrated in the following figures. Figure 62 and Figure 63 illustrate the regression of EC%, with ΔTc parameter under two different conditions: RTFO on UU samples (PG1), and RTFO+20-hour PAV on UU samples (PG2). The ΔTc values of contract 8433 were very low relative to other seven contracts, so they were considered outliers and excluded from the regression graphs. However, note that the values are shown on the figures via red square symbols. The R-square values suggest better correlation between EC% with ΔTc of PG2 (R-square = 0.50), then with ΔTc of PG1 (R-square = 0.16). When correlating EC% to the IDT strength test parameters, the EC% was better correlated with CT-Index of AU samples, followed by the IDT strength values of AU samples with an R-square of 0.80 and 0.68 shown in Figure 64 and Figure 65, respectively. The correlations between EC% and the rest of IDT test parameters indicated very weak correlations. Again, it is very important to emphasize that the amount of equivalent cracking in the surveyed field sections was very minimal and the range of ΔTc values was small, resulting in unexpected and unreliable trends for the CT-Index and IDT strength values. The very small range of EC% value observed in the relatively young pavements resulted in unreliable regression models between IDT test parameters and field cracking.

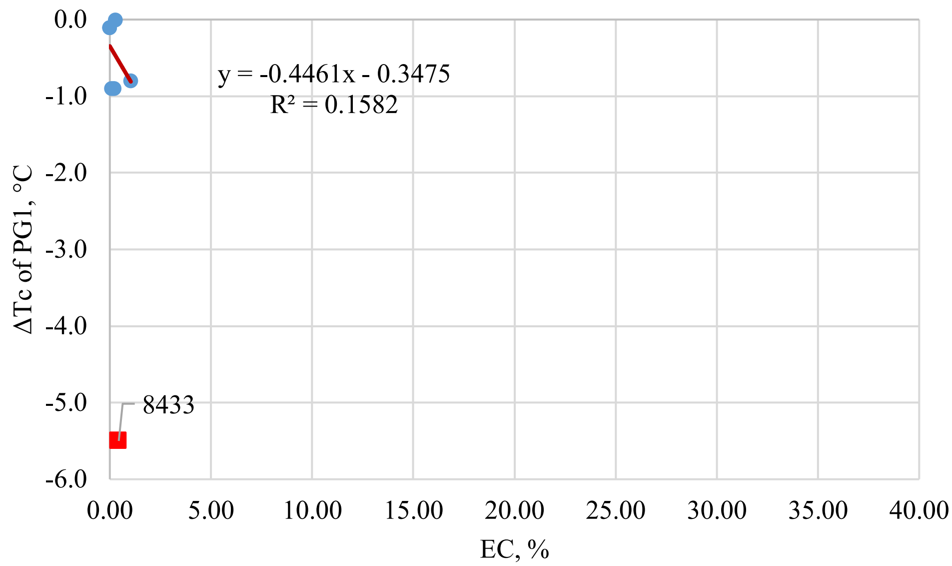


Figure 62: ΔTc PG1 Regression with EC%

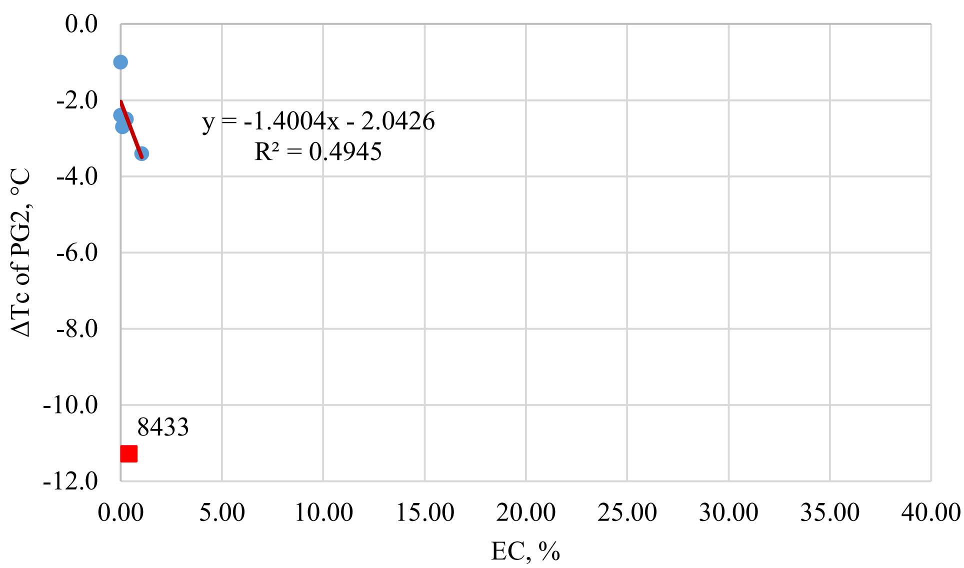


Figure 63: ΔTc PG2 Regression with EC%

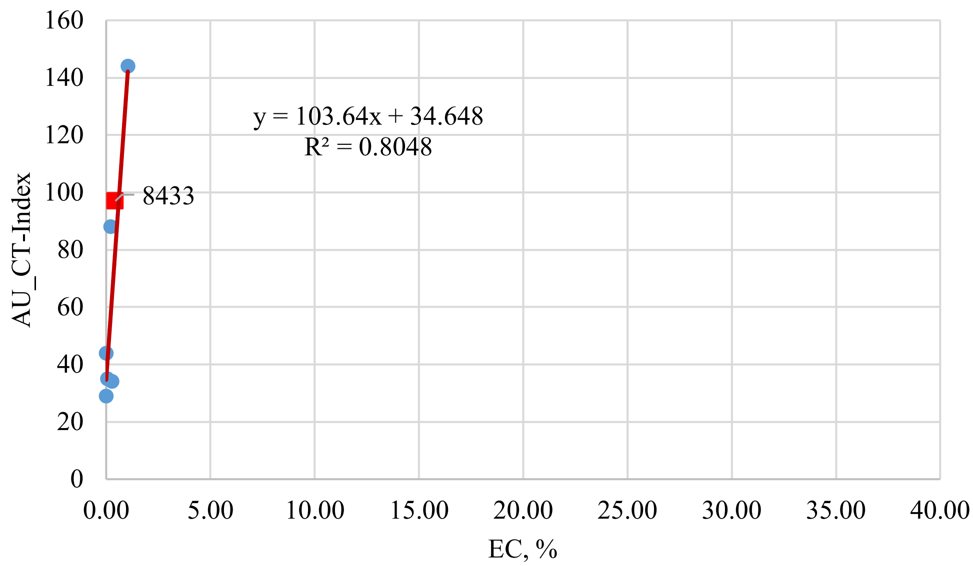


Figure 64: AU CT-Index Regression with EC%

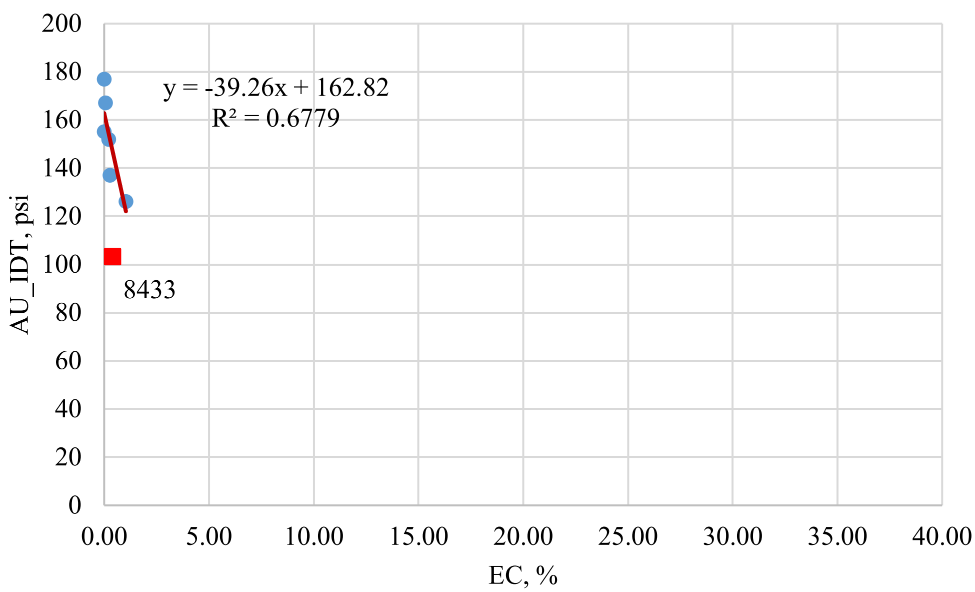


Figure 65: AU IDT Strength Regression with EC%

9 CONCLUSIONS AND RECOMMENDATIONS

The durability of Asphalt Concrete (AC) containing recycled asphalt materials has become a focal point of the asphalt industry. WSDOT has successfully been using RAP since the mid 1970's and it has been the contractor's option to use 0 to 20% RAP in dense graded mixes without the need to adjust binder properties or limit RAP to meet project specific PG requirements. In 2008, WSDOT specifications were revised to allow up to 40 percent recycled binder in mixtures with no more that 20 percent of it being from RAS. The use of RAS has been very limited, though high RAP use has been embraced by industry.

WSDOT initiated this effort, with industry input through WAPA, with the goal of optimizing the use of RAP in AC mixtures, at the same time that the Balance Mix Design (BMD) approach was being embraced nationally. The research was collaboratively conducted by the University of Washington and University of Nevada, Reno with an experimental plan that included a review of literature and assessment of the overall performance of WSDOT AC mixtures containing low and high RAP based on PMS data. Additionally, the engineering properties of raw materials, lab produced mixtures, plant produced mixtures, and field cores were determined under different aging conditions and analyzed. Emphasis was placed on virgin and RAP binder aging susceptibility and contribution to mixture rutting and cracking performance measured with the Hamburg Wheel Track (HWTT) test and Indirect Tensile (IDT) test properties. Based on analysis of the outcomes from the experimental plan the following observations are made:

1. The key points from the literature review indicated:
    a. *Improving durability*: A 2019 NCAT study evaluated adjustments adopted by DOTs, to the Superpave mix design method and their effectiveness to

enhance pavement performance and long-term durability (Tran et al. 2019). According to the list of mix design adjustments implemented by SHAs at the time of the survey, the multiple-stress creep recovery (MSCR) specification for asphalt binder was at the top of specification changes, followed by increasing the use of polymer-modified asphalt binder and decreasing the design compaction effort N_{Design}. Similar to reducing N_{Design}, reducing the design air voids, as well as increasing design VMA were some of the major AC mixture adjustments, all of which focus on increasing effective asphalt binder content in AC mixtures. Supplementing the Superpave mix design method with performance testing was identified to assure good stability and long-term durability of AC mixtures. Hence, the asphalt industry is focused on implementing BMD today. Subsequently, an NCHRP project focused on improving durability of AC mixtures by determining quality control and acceptance testing framework for BMD of recycled mixtures, using CT-Index, and I-FIT for cracking evaluation (Yin et al. 2020). Likewise, Elias et al. proposed implementing the BMD with Superpave volumetric foundation for RAP mixtures, at the local agency level. The authors recommended implementing a BMD using HWTT and CT-Index as stability and cracking indicators, respectively, while refining the HWTT and CT-Index (after long-term aging) criteria; using 50 gyrations and 4% design air voids regardless of roadway functional classification; and asphalt content, AV, VMA, and DP in acceptance.

b. *Impact of virgin and RAP binder:* Mogawer et al. illustrated that blended binder properties were significantly affected by virgin binder and RAP source, while concluding that the current MassDOT specification allowing up to 15% RAP without considering source or RAP properties was not rational (Mogawer et al. 2020). Virgin binder source significantly affected mixture cracking performance, while RAP source and dosage also impacted flexibility index (FI) and CT-Index. It was recommended that low and intermediate temperature properties be considered when incorporating RAP in AC mixtures and that performance testing be used with the aim of ensuring satisfactory long-term performance.

c. *ΔT_c parameter*: The ΔTc parameter and several rheological properties were compared with field performance data, finding that 40-hour PAV ΔTc showed good correlation with field cracking after six years of service life (Reinke 2017). A number of researchers have tried to correlate ΔTc with field performance data and mixture cracking tests results, experiencing variable degrees of success. According to the Asphalt Institute (AI), the ΔTc parameter has been implemented by ten State DOTs. The AI suggests using 40-hour instead of 20-hour PAV ΔTc to reduce the risk of accepting asphalt binders susceptible to premature embrittlement.

2. *LMLC binder properties:* The low PG for most of the eleven virgin binders evaluated was controlled by the relaxation parameter m-value after 20-hour PAV aging and all were m-value controlled after 40-hour PAV aging. The impact of the

additional 20-hour PAV aging, induced significant decreases in ΔTc (more negative).

3. *Physical blending and blending charts:* The ΔTc parameter revealed a more pronounced decrease (more negative) with aging and higher RAP amount. However, this drop reached a plateau in some cases with different RAP amounts after extended aging. A comparison between ΔTc estimated from blending charts and ΔTc measured on physically blended binders revealed that the blending charts underestimated the ΔTc (in absolute value) for 90% of the evaluated binder blends. The difference ranged from -2.7°C to +0.3°C, with an average of -0. 9°C. A statistically significant difference in ΔTc (i.e., p-value of 0.02), suggests uncertainty in predicting the ΔTc parameter from blending charts. With respect to final PG, the difference in grade between physical blending and blending charts was not as pronounced as ΔTc values.
4. *Binder source*: A statistically significant difference was captured in the IDT test results for the same binder PG binder grade supplied from different sources, which may suggest that the binder source can have a significant influence on cracking tests properties.
5. *LMLC mixture properties:* The higher RAP amounts increased the IDT strength values and lowered the CT-Index between different mixture types. However, the use of a recycling agent with high RAP mixture effectively reduced the mixture stiffness, thus lowering the relative IDT strength, and increasing CT-Index. A comprehensive examination confirmed that the short-term aged (UU) IDT strength values were better correlated to UU fracture energy than the CT-Index, which was

validated with a correlation factor of 0.965. However, the IDT strength values, and CT-index illustrated more consistent trends with increased RAP doses, compared to fracture energy. Preliminary minimum CT-Index criteria of 65 and 37 were identified for short-term aged (UU) and long-term aged (AU) specimens, respectively. The long-term aged (AU) CT-Index criteria captured more cracking deficient mixtures than the short-term aged (UU) samples. The HWTT rutting data demonstrated that even with softer virgin binder and/or recycling agent, none of the LMLC samples exceeded the current WSDOT criterion.

6. <u>*LMLC statistical comparison with varying RAP amounts:*</u> Within the same virgin binder grade, 43% of mixture properties were significantly different between 0% and 20% RAP mixtures, 67% of mixture properties were significantly different between 0% and 25% RAP mixtures, and 7% were significantly different between 20% and 25% RAP mixtures.
7. <u>*FMLC and FMFC extracted binder properties:*</u> For both FMLC and FMFC samples, 20-hour PAV aging of extracted binder further aged the binder and reduced ΔTc more than the long-term aging of compacted specimens per AASHTO R30 did.
8. <u>*FMLC mixture properties:*</u> For most mixtures lower variability was associated with IDT strength results than CT-Index and fracture energy results. When comparing the field mixtures to the replicated LMLC samples, the IDT strength parameter was most effective at identifying significant differences between FMLC and LMLC, and the FMLC IDT strengths were always greater than the short-term aged LMLC IDT strengths.

9. *FMFC (core) mixture properties:* There was not a clear trend observed between the cracking test results on cores, likely due to the higher variability in thickness and air voids among cores from the same contract, as well as a significant range of air voids (3.6% to 8.6%) between contracts. Despite this high variability, in most cases lower variability was associated with IDT strength results than CT-Index and fracture energy results. A statistical comparison did not identify a significant difference between the core cracking properties of high and low RAP contracts. It is important to recognize that the pavements were relatively young and only one had a significant amount of cracking, which was a high RAP mixture.
10. *IDT strength criteria evaluation:* The short-term aged (UU) IDT strength parameter surpassed the CT-Index and fracture energy parameters in capturing the significant differences with different RAP doses, as well as the discrepancy between field mix and laboratory mix test results, while showing minimum variability among replicates for different parameters. Additionally, the IDT strength parameter was the most consistent with the dynamic modulus (E*) engineering property, in identifying significant differences between mixtures at varying RAP dose.
11. *PMS data:* The greatest rut depth observed was 0.30 inch, on a high RAP contract, after 4 years of service, compared to 0.20 inch within low RAP contracts over the same period. Based on the available data for the first three years after construction for all the projects, some overlapping in rut depth and IRI was observed among high RAP and low RAP projects, suggesting similar performance. A comparison of equivalent cracking percentages (EC%) for the first three years of pavement service life among the projects showed a low and tight range 0.00% and 0.44% EC for high

and low RAP projects, except for a high RAP project (34% RAP) which exhibited 4.09% and 17.34% EC at 3 years and 4 years after construction, respectively. Finally, a statistical analysis comparing the pavement condition (i.e., rut depth, IRI, EC%) did not capture any statistically significant differences between high and low RAP projects. It is important to recognize the comparison was done on pavements that were all relatively young, while some cracking trends may take longer to emerge.

12. *ΔTc regression with LMLC performance*: The ΔTc parameter correlated best with fracture energy under short-term (UU) and long-term (AU) aging conditions, followed by the IDT strength values, and then the CT-Index.
13. *ΔTc regression with FMFC performance:* These regressions with field cores had considerably lower R-square values, due to the higher variability within FMFC samples. Despite the overall high variability, the regressions suggest that the long-term aged (AU) IDT strength value had the best correlation with ΔTc among all IDT test parameters for FMFC specimens.
14. *PMS data regression with FMFC:* The developed regression models are not adequately reliable due to the very small range of EC% among the projects, which is likely due to the relatively short time period over which the projects were in service.

The following short-term (within 1-2 year) recommendations are made based on the research:

1. Include RAM in all mix designs, regardless of the percentage used, or percent binder replacement.

2. Integrate ΔTc into virgin asphalt binder and virgin binder/RAM binder blend specifications.
3. Determine ΔTc by physical blending of virgin, RAP binder and RA if any, rather than by blending charts.
4. For High RAM mixtures (≥ 20% RAP or any RAS):
    a. Require that the PG of blended binder meet the project specific requirements with the RAM to be used.
    b. Integrate volumetric parameters along with HWTT and CT-Index performance testing as a BMD method for mix design and test section requirements.
    c. Implement the minimum VMA requirements in AASHTO M323 for mix design, test section and acceptance requirements.
    d. Maintain the current WSDOT HWTT test method and criteria for mix design and test sections.
    e. Maintain the current WSDOT IDT strength test and criteria for mix design and test sections, while shadowing the proposed STOA CT-Index criteria of ≥ 65 prior to full implementation.
    f. When possible, on test sections perform long-term aging of IDT/CT-index specimens to collect data for a future long-term aged CT-Index specification.
5. Collect strength, CT-Index, and fracture energy data whenever performing IDT tests.

The following long-term (within 4-8 years) recommendations are made based on the research:

1. Evaluate the preliminary short-term and long-term aged IDT strength (≤175psi and ≤230psi) and CT-Index (≥65 and ≥37) criteria identified in this research by collecting data on all WSDOT projects with the short-term recommended volumetric changes and long-term aging of IDT mix design and test strip specimens.
2. Use the data from the previous recommendation to refine the preliminary CT-Index criteria, while at the same time developing an understanding of typical variability of the parameters for WSDOT specific materials. Also validate the reliability of the relationship between short-term and long-term aged CT-Index identified in this study so it could be implemented for future acceptance criteria.
3. Analyze the impacts of recommendations 1 and 2 on acceptance and payment for the data from the entire season by comparing the current and future acceptance along with payment rates.
4. Share the information from recommendations 1-3 with industry partners via WAPA and provide training WSDOT and WAPA training on the changes and impacts of them.
5. Revise the current WSDOT 504 specifications to:
    a. Eliminate the mix design classification based on RAP/RAS content
    b. Add short-term and long-term aged CT-Index criteria to mix design and test strip acceptance criteria.

6. Revise the WSDOT 5-04 specifications to add IDT strength or CT-Index to acceptance criteria after shadow specification implementation for 1 year.

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APPENDIX A: EXPERIMENTAL PLAN

Table 50: UNR Experimental Plan-Contract 9145

Contract	Materials					Laboratory Test						Blending Chart	Physical Blending and Comparison
						Binder	Agg.	Mixture					
	Binder Grade	Binder Supplier	RAP, %	RA, %	Zyco, %	PG & ΔTc	Sieve Analysis	Extraction & Recovery	IDT	Prepare HWTT	E*		
9145 (Eastern)	PG64-28	1	0.0[1]	-	-	✓							
	PG64-28[1]	2[1]		-	-	✓[1]			✓[1]		✓[1]		
	PG58-34	2		-	-	✓							
	PG64-28	1	20.0	-	-							✓	
	PG64-28	2		-	-	✓			✓		✓	✓	✓
	PG58-34	2		-	-	✓			✓		✓	✓	✓
	PG64-28	1	25.0	-	-							✓	
	PG64-28	2		-	-				✓		✓	✓	
	PG58-34	2		-	-	✓			✓		✓	✓	✓
	RAP I					✓	✓	✓					
	Loose Field Mix					✓		✓	✓	✓			

[1]: Original Mix Design as per WSDOT JMF

[2]:Field Cores Contracts: - 7706, 8438, 8128, 8624, 8433, 8441, & 8465
RA = Recycling Agent
Zyco = Zycotherm anti-stripping Agent

Table 51: UNR Experimental Plan-Contract 9262/9229

Contract	Materials					Laboratory Test						Blending Chart	Physical Blending and Comparison
						Binder	Agg.	Mixture					
	Binder Grade	Binder Supplier	RAP, %	RA, %	Zyco, %	PG & ΔTc	Sieve Analysis	Extraction & Recovery	IDT	Prepare HWTT	E*		
9262/9229 (Western zone)	PG58H-22	3	0.0	-	-	✓							
	PG58H-22	4		-	-	✓			✓		✓		
	PG58S-28	4		-	-	✓							
	PG58H-22	3	20.0	-	-							✓	
	PG58H-22	4		-	-	✓			✓		✓	✓	✓
	PG58S-28	4		-	-							✓	
	PG58H-22	3	25.0	-	-							✓	
	PG58H-22	4		-	-	✓			✓		✓	✓	✓
	PG58S-28	4		-	-							✓	
	PG58S-28	4	36.0[1]	-	-	✓						✓	✓
	PG58S-28	3		-	-							✓	
	PG58S-28[1]	4[1]		3.1[1]	-	✓[1]			✓[1]		✓[1]		✓[1,3]
	RAP II					✓	✓	✓					
	Loose Field Mix								✓	✓			

[1]: Original Mix Design as per WSDOT JMF
[2]: Field Cores Contracts: - 7706, 8438, 8128, 8624, 8433, 8441, & 8465

[3]: Physical Blending was done evaluate the effect of RA without any comparison with blending charts.

RA = Recycling Agent
Zyco = Zycotherm anti-stripping Agent

Table 52:UNR Experimental Plan-Contract -9231, 7706, 8438, 8128, 8624, 8433, 8441, and 8465

Contract	Materials				Laboratory Test						Blending Chart	Physical Blending and Comparison
	Binder Grade	Binder Supplier	RAP, %	Zyco, %	PG & ΔTc	Sieve Analysis	Extraction & Recovery	IDT	Prepare HWTT	E*		
9231 (Western)	PG58S-28	3	0.0	-	✓							
	PG52S-28	3		-	✓							
	PG58H-22	5		-	✓							
	PG52H-28	5		-	✓							
	PG52S-28	5		-	✓							
	PG58H-22	3	20.0	-							✓	
	PG58H-22	5		-	✓						✓	✓
	PG52H-28	5		-							✓	
	PG58H-22	3	25.0	-							✓	
	PG58H-22	5		-	✓						✓	✓
	PG52H-28	5		-							✓	
	PG52S-28[1]	5[1]	40.0[1]	-	✓[1]						✓[1]	✓[1]
	PG52S-28	3		-	✓						✓	✓
	PG58H-22	3	0.0	0.10								
	PG58H-22	5						✓		✓		
	PG52H-28	5										
	PG58H-22	3	20.0	0.12								
	PG58H-22	5						✓		✓		
	PG52H-28	5						✓		✓		
	PG58H-22	3	25.0	0.12								
	PG58H-22	5						✓		✓		
	PG52H-28											
	PG52S-28[1]	5[1]	40.0[1]	0.15[1]				✓		✓		
	PG52S-28	3						✓[1]		✓[1]		
	RAP III				✓	✓	✓					
	Loose Field Mix				✓		✓	✓	✓			
2	Field Cores				✓		✓	✓				

[1]: Original Mix Design as per WSDOT JMF
[2]: Field Cores Contracts: - 7706, 8438, 8128, 8624, 8433, 8441, & 8465
Zyco = Zycotherm anti-stripping Agent

APPENDIX B: FIELD CORES DESCRIPTION

Table 53: Field Core Description

Contract No.	Location	Climatic Region	Mix Design ID	No. of samples
8441	SR 12 MP 7.6	Olympic Region, (Western Region)	MD130037	6 cores
	SR 12 MP 5.45		Contractor: Granite	5 cores
8624	SR 16 MP 24	Olympic Region, (Western Region)	MD140060	5 cores
	SR 16 MP 22.5		Contractor: Granite	5 cores
8128	SR 522 MP 23.96	Northwest Region (Western Region), up to 20% RAP,	MD130004	5 cores
	SR 522 MP 22.33			1 core
	SR 522 MP 22.00			5 cores
8438	SR 531 MP 7.75	Northwest Region (Western Region), high RAP contract	MD130029	1 core
	SR 531 MP 7.88		Contractor: Granite	5 cores
	SR 531 MP 8.45			5 cores
8465	US 101 MP 355	Olympic Region, (Western Region)		5 cores
	US 101 MP 359.75			5 cores
8433	SR 14 MP 179	South Central Region (Eastern Region)	MD130039	7 cores
	SR 14 MP 176		Contractor: Granite	7 cores
7706 (East)	I-90 WB MP 103.14	South Central Region (Eastern Region), up to 20% RAP		6 cores
7706 (West)	I-90 EB MP 103.14	South Central Region (Eastern Region), up to 20% RAP		6 cores

Table 54: Field Core Description (Continued)

Contract No.	NMAS	Virgin Binder	% RAP
8441	1/2"	-	34%
8624	1/2"	PG64-22	43%
8128	1/2"	PG64-22	Not High RAP
8438	1/2"	-	42% RAP
8465	3/8"	PG64-22	Not High RAP
8433	1/2"	PG64-28	42% RAP
7706 (East)	1/2"	PG70-28	Not High RAP
7706 (West)	1/2"	PG70-28	Not High RAP

APPENDIX C: CONTRACT 9145 MIX DESIGN

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia WA98504 / 1655 S. 2nd Ave. Tumwater WA 98512
BITUMINOUS MATERIALS SECTION REFERENCE MIX DESIGN REPORT

MATERIAL : HMA Class 1/2" - 9-03.8 - 2016 — WORK ORDER NO : 009145
DATE RECV'D : 03/27/2018 — REFERENCE NO : RD180017
REVIEWED BY : DavisSJ — MIX ID NO : MD170081
SR NO : 195 — REFERENCED FROM WORK ORDER NO : MS5839
PROJECT ENGINEER : Allen, Mark — ORG CODE : 464310 — CONTRACTOR : Shamrock Paving
SECTION : COLFAX TO SPANGLE - ADD PASSING LANES - PHASE 2

THIS REFERENCE MIX DESIGN FOR **13490 TONS** OF HMA IS BASED ON PREVIOUS USE OF AGGREGATES FROM THE SOURCE LISTED.
THIS DESIGN IS IN LIEU OF AN EVALUATION OF CURRENT STOCKPILES AND PRODUCTION.

CONTRACTOR'S MIX DESIGN TEST DATA

					Specification
Pb		5.0	5.3	6.0	
% Gmm @ Ninitial	7	83.4	84.4	86.0	≤ 90.5
% Va @ Ndesign	75	4.7	4.1	2.4	Approximate 4.0
% VMA @ Ndesign	75	14.8	14.7	14.6	≥ 14.0
% VFA @ Ndesign	75	68	72	84	65 - 78
% Gmm @ Nmax	115		97.6		≤ 98.0
Dust to Asphalt Ratio (D/A)		1.4	1.4	1.2	0.6 - 1.6
Pbe		4.2	4.5	5.1	
Gmm		2.550	2.543	2.519	
Gmb		2.429	2.439	2.459	
Gb		1.030	1.030	1.030	
Gse		2.765	2.771	2.775	
Hamburg Wheel-Test (mm)			3.6		≤ 10.0
Stripping Inflection Point			Pass		None @ 15,000
Indirect Tensile Strength (psi)			129		≤ 175

STATE MATERIALS LABORATORY VERIFICATION TEST DATA

					Specification
Pb		4.8	5.3	5.8	
% Gmm @ Ninitial	7	84.2	86.0	87.2	≤ 90.5
% Va @ Ndesign	75	7.0	4.7	3.2	Approximate 4.0
% VMA @ Ndesign	75	14.3	13.6	13.2	≥ 14.0
% VFA @ Ndesign	75	52	66	76	65 - 78
% Gmm @ Nmax	115		96.5		≤ 98.0
Dust to Asphalt Ratio (D/A)		2.0	1.6	1.5	0.6 - 1.6
Pbe		3.2	3.8	4.2	
Gmm		2.567	2.541	2.527	
Gmb		2.388	2.423	2.446	
Gb		1.030	1.030	1.030	
Gse		2.776	2.769	2.775	
Hamburg Wheel-Test (mm)			3.9		≤ 10.0
Stripping inflection Point			Pass		None @ 15,000
Indirect Tensile Strength (psi)			104		≤ 175

STRIPPING EVALUATION

% Anti-Strip :
Visual Appearance :
% Retained Strength :

STATE MATERIALS LABORATORY RECOMMENDATIONS

Asphalt Binder Supplier	**WSA**	
Asphalt Binder Grade	PG 64-28	
Percent Binder (Pb) (By Wt. Total Mix)	5.3	
% Anti-Strip (By Wt. of Asphalt Binder) / Type	0.00	
Sample Wt. (grams)	4800	(Informational Only)
Ignition Calibration Factor	0.80	(Informational Only)
Optimum Mixing Temperature (°F)	318	
Compaction Temperature (°F)	296	

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia WA98504 / 1655 S. 2nd Ave. Tumwater / WA 98512
BITUMINOUS MATERIALS SECTION REFERENCE MIX DESIGN REPORT

MATERIAL: HMA Class 1/2" - 9-03.8 - 2016 WORK ORDER NO : 009145
SAMPLE ID : 000001228cc MIX ID NO : RD180017

CONTRACTOR'S DESIGN AGGREGATE STRUCTURE AND AGGREGATE TEST DATA

Material:	3/4"-#4	3/8"-0	#40-0	Combined	Spec	Tolerance
Source:	C336	C336	GT336			
Ratio:	30.0%	67.0%	3.0%			
3/4 in	100.0	100.0	100.0	100	99 - 100	99 - 100
1/2 in	75.0	100.0	100.0	93	90 - 100	90 - 99
3/8 in	40.0	100.0	100.0	82	90 Max	76 - 88
No. 4	2.0	75.0	100.0	54		49 - 59
No. 8	1.0	46.0	100.0	34	28 - 58	30 - 38
No. 16	1.0	29.0	100.0	23		
No. 30	1.0	18.0	100.0	15		
No. 50	1.0	12.0	95.0	11		
No. 100	1.0	10.0	30.0	8		
No. 200	1.0	8.5	2.0	6.1	2.0 - 7.0	4.1 - 7.0

Gsb Coarse	2.731	2.717			
Gsb Fine		2.691	2.684		
Gsb Blend	2.731	2.697	2.684	2.707	
Sand Equivalent (SE)		79	81		45 Min
% Uncompacted Voids		47			40 Min
% Fracture	100	99			90 Min Double Face Fracture

STATE MATERIALS LABORATORY AGGREGATE TEST DATA

Gsb Coarse	2.741	2.704			
Gsb Fine		2.586	2.660	2.590	
Gsb Blend	2.741	2.615	2.660	**2.653**	
Sand Equivalent (SE)		99	99	99	45 Min
% Uncompacted Voids				44	40 Min
% Fracture	95	93		94	90 Min Single Face Fracture

COMMENTS

Remarks:

Result Code:

Remarks :

Kurt R. Williams, P.E.
State Materials Engineer
Steven J. Davis
Bituminous Materials Engineer
Date : 3/28/2018
Phone : (360) 709-5424

Billing Code
T162 - 1

APPENDIX D: CONTRACT 9262 / 9229 MIX DESIGN

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia WA 98504 / 1655 S. 2nd Ave. Tumwater WA 98512
BITUMINOUS MATERIALS SECTION REFERENCE MIX DESIGN ANTI-STRIP REPORT

MATERIAL :	HMA Class 3/8" - 9-03.8 - 2018	WORK ORDER NO :	009262
DATE RECV'D :	06/20/2018	REFERENCE NO :	RD180078
REVIEWED BY :		MIX ID NO :	MD180027
SR NO :	500	REFERENCED FROM WORK ORDER NO :	MS5839
PROJECT ENGINEER :	Figone, Lori	ORG CODE : 444301	CONTRACTOR : Granite Construction
SECTION :	LEADBETTER RD TO SE 3RD AVE PAVING AND ADA		

THIS REFERENCE MIX DESIGN FOR **4555 TONS** OF HMA IS BASED ON PREVIOUS USE OF AGGREGATES FROM THE SOURCE LISTED.
THIS DESIGN IS IN LIEU OF AN EVALUATION OF CURRENT STOCKPILES AND PRODUCTION.

CONTRACTOR'S MIX DESIGN TEST DATA

VALID FOR 2018

					Specification
Pb		5.4	5.8	6.4	
% Gmm @ Ninitial	7	85.9	86.7	87.9	≤ 90.5
% Va @ Ndesign	75	5.1	4.0	2.3	Approximate 4.0
% VMA @ Ndesign	75	16.5	16.1	15.9	≥ 15.0
% VFA @ Ndesign	75	69	75	86	65 - 78
% Gmm @ Nmax	115		97.5		≤ 98.0
Dust to Asphalt Ratio (D/A)		1.4	1.3	1.2	0.6 - 1.6
Pbe		4.9	5.2	5.9	
Gmm		2.500	2.493	2.469	
Gmb		2.373	2.392	2.413	
Gb		1.033	1.033	1.033	
Gse		2.721	2.728	2.728	
Hamburg Wheel-Test (mm)			2.9		≤ 10.0
Stripping Inflection Point			Pass		None @ 15,000
Indirect Tensile Strength (psi)			134		≤ 175

STATE MATERIALS LABORATORY VERIFICATION TEST DATA

CONTRACT 009262 ONLY

					Specification	Tolerance
Pb		5.3	6.0	6.3		
% Gmm @ Ninitial	7	83.8	85.9	86.5	≤ 90.5	
% Va @ Ndesign	75	8.1	5.2	4.3	Approximate 4.0	2.5 - 5.5
% VMA @ Ndesign	75	17.8	16.8	16.4	≥ 15.0	
% VFA @ Ndesign	75	54	69	74	65 - 78	
% Gmm @ Nmax	115		96.6		≤ 98.0	
Dust to Asphalt Ratio (D/A)		1.6	1.4	1.3	0.6 - 1.6	
Pbe		4.3	5.1	5.3		
Gmm		2.529	2.500	2.493		
Gmb		2.324	2.370	2.387		
Gb		1.033	1.033	1.033		
Gse		2.752	2.750	2.755		
Hamburg Wheel-Test (mm)			3.8		≤ 10.0	
Stripping inflection Point			Pass		None @ 15,000	
Indirect Tensile Strength (psi)			132		≤ 175	

STRIPPING EVALUATION

% Anti-Strip :
Visual Appearance :
% Retained Strength :

STATISTICAL

STATE MATERIALS LABORATORY RECOMMENDATIONS

Asphalt Binder Supplier	**TARGA SOUND**	
Asphalt Binder Grade	PG58H-22	
Percent Binder (Pb) (By Wt. Total Mix)	6.0	
% Anti-Strip (By Wt. of Asphalt Binder) / Type	0.00	
% Virgin Pb Binder (By Wt. Total Mix)	3.68	
% Binder Contributed From RAP & RAS (By % Total Binder Content)	36.5	≤ 40.0
Recycling Agent (By Wt. Virgin Asphalt Binder)	3.1	
Type Recycling Agent	Revive 1114	
Sample Wt. (grams)	4725	(Informational Only)
Ignition Calibration Factor	0.51	(Informational Only)
Optimum Mixing Temperature (°F)	300	
Compaction Temperature (°F)	277	

Page 1 of 2

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia WA 98504 / 1655 S. 2nd Ave. Tumwater / WA 98512
BITUMINOUS MATERIALS SECTION REFERENCE MIX DESIGN ANTI-STRIP REPORT

MATERIAL: HMA Class 3/8" - 9-03.8 - 2018 WORK ORDER NO : 009262
SAMPLE ID : 00000123418 MIX ID NO : RD180078

CONTRACTOR'S DESIGN AGGREGATE STRUCTURE AND AGGREGATE TEST DATA

Material:	#4-#8	#4-0	1/2"-#8	Fine Aggregate	3/8" RAP	Combined	Spec	Tolerance
Source:	OR27	OR27	Z157	G102	UNCL			
Ratio:	11.0%	37.0%	11.0%	5.0%	36.0%			
1/2 in	100.0	100.0	99.9	100.0	99.3	100	99 - 100	99 - 100
3/8 in	99.9	99.9	99.5	99.8	94.2	98	90 - 100	92 - 100
No. 4	42.4	78.3	33.7	99.2	70.1	68	90 Max	63 - 73
No. 8	9.8	51.9	3.0	97.8	50.6	44	32 - 67	40 - 48
No. 16	5.4	32.3	1.0	93.8	36.0	30		
No. 30	4.1	22.0	0.6	82.5	26.4	22		
No. 50	3.4	15.0	0.5	43.9	18.6	15		
No. 100	2.9	10.7	0.4	5.7	13.5	9		
No. 200	2.4	7.7	0.3	2.2	10.1	6.9	2.0 - 7.0	4.9 - 7.0

Gsb Coarse	2.677	2.658	2.740				
Gsb Fine		2.627		2.652			
Gsb Blend	2.677	2.634	2.740	2.652	2.753	2.693	
Sand Equivalent (SE)		80		88		81	45 Min
% Uncompacted Voids		48		50		48	40 Min
% Fracture	99		98			98	90 Min Single Face Fracture

STATE MATERIALS LABORATORY AGGREGATE TEST DATA

Gsb Coarse	2.682	2.678	2.720		2.750		
Gsb Fine		2.599		2.505		2.585	
Gsb Blend	2.682	2.616	2.720	2.505	2.750	**2.676**	
Sand Equivalent (SE)		79		84			45 Min
% Uncompacted Voids						47	40 Min
% Fracture	100	100	98				90 Min Single Face Fracture

COMMENTS

Remarks:

Result Code:

Remarks : Test data verified PG58S-28 blended with RAP binder and Revive recycling agent 1114 meets PG58H-22 specified for project.
Mix design report reflects proposed design binder adjustment from 5.8 Pb to 6.0 Pb.

Kurt R. Williams, P.E.
State Materials Engineer
Steven J. Davis
Bituminous Materials Engineer
Date : 6/20/2018
Phone : (360) 709-5424

Billing Code

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APPENDIX E: CONTRACT 9231 MIX DESIGN

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia WA 98504 / 1655 S. 2nd Ave. Tumwater WA 98512
BITUMINOUS MATERIALS SECTION REFERENCE MIX DESIGN ANTI-STRIP REPORT

MATERIAL : HMA Class 3/8" - 9-03.8 - 2018 WORK ORDER NO : 009231
DATE RECV'D : 09/20/2018 REFERENCE NO : RD180136
REVIEWED BY : DavisSJ MIX ID NO : MD180054
SR NO : REFERENCED FROM WORK ORDER NO : MS5839
PROJECT ENGINEER : Spahr, Shane ORG CODE : 412334 CONTRACTOR : Granite Construction
SECTION : STILLAGUAMISH R BR TO HILL DITCH BR PCCP REHAB & PAVING & ADA
THIS REFERENCE MIX DESIGN FOR **13000 TONS** OF HMA IS BASED ON PREVIOUS USE OF AGGREGATES FROM THE SOURCE LISTED.
THIS DESIGN IS IN LIEU OF AN EVALUATION OF CURRENT STOCKPILES AND PRODUCTION.

CONTRACTOR'S MIX DESIGN TEST DATA

VALID FOR 2018

					Specification
Pb		5.3	5.8	6.3	
% Gmm @ Ninitial	8	86.7	88.1	89.0	≤ 89.0
% Va @ Ndesign	100	5.6	3.9	2.6	Approximate 4.0
% VMA @ Ndesign	100	16.4	15.8	15.7	≥ 15.0
% VFA @ Ndesign	100	66	75	83	73 - 76
% Gmm @ Nmax	160		96.7		≤ 98.0
Dust to Asphalt Ratio (D/A)		1.1	1.0	0.9	0.6 - 1.6
Pbe		4.7	5.1	5.6	
Gmm		2.502	2.488	2.470	
Gmb		2.362	2.390	2.406	
Gb		1.030	1.030	1.030	
Gse		2.720	2.726	2.726	
Hamburg Wheel-Test (mm)			3.5		≤ 10.0
Stripping Inflection Point			Pass		None @ 15,000
Indirect Tensile Strength (psi)			97		≤ 175

STATE MATERIALS LABORATORY VERIFICATION TEST DATA

CONTRACT 009231 ONLY

					Specification	Tolerance
Pb		5.3	5.8	6.3		
% Gmm @ Ninitial	8	85.8	87.2	88.7	≤ 89.0	
% Va @ Ndesign	100	6.7	4.8	2.9	Approximate 4.0	2.5 - 5.5
% VMA @ Ndesign	100	16.0	15.5	14.9	≥ 15.0	13.5 - 100.0
% VFA @ Ndesign	100	58	70	81	73 - 76	
% Gmm @ Nmax	160		96.8		≤ 98.0	
Dust to Asphalt Ratio (D/A)		1.3	1.1	1.0	0.6 - 1.6	
Pbe		4.1	4.7	5.2		
Gmm		2.515	2.490	2.472		
Gmb		2.347	2.372	2.401		
Gb		1.030	1.030	1.030		
Gse		2.735	2.728	2.729		
Hamburg Wheel-Test (mm)			5.6		≤ 10.0	
Stripping inflection Point			Pass		None @ 15,000	
Indirect Tensile Strength (psi)			138		≤ 175	

STRIPPING EVALUATION

% Anti-Strip :
Visual Appearance :
% Retained Strength :

STATISTICAL

STATE MATERIALS LABORATORY RECOMMENDATIONS

Asphalt Binder Supplier	**HUSKY OIL**	
Asphalt Binder Grade	*PG58H-22	
Percent Binder (Pb) (By Wt. Total Mix)	5.8	
% Anti-Strip (By Wt. of Asphalt Binder) / Type	0.10	Zycotherm
% Virgin Pb Binder (By Wt. Total Mix)	3.93	
% Binder Contributed From RAP & RAS (By % Total Binder Content)	32.2	≤ 40.0
Recycling Agent (By Wt. Virgin Asphalt Binder)	0.00	
Type Recycling Agent		
Sample Wt. (grams)	4750	(Informational Only)
Ignition Calibration Factor	0.82	(Informational Only)
Optimum Mixing Temperature (°F)	300	
Compaction Temperature (°F)	279	

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Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia WA 98504 / 1655 S. 2nd Ave. Tumwater / WA 98512
BITUMINOUS MATERIALS SECTION REFERENCE MIX DESIGN ANTI-STRIP REPORT

MATERIAL: HMA Class 3/8" - 9-03.8 - 2018 WORK ORDER NO : 009231
SAMPLE ID : 000001245fd MIX ID NO : RD180136

CONTRACTOR'S DESIGN AGGREGATE STRUCTURE AND AGGREGATE TEST DATA

Material:	3/8" Chips	#4-0	Fine Aggregate	3/8" RAP	Combined	Spec	Tolerance
Source:	D217	D217	F160	UNCL			
Ratio:	17.0%	28.0%	15.0%	40.0%			
1/2 in	99.9	100.0	100.0	99.8	100	99 - 100	99 - 100
3/8 in	97.4	100.0	100.0	93.2	97	90 - 100	91 - 100
No. 4	15.5	85.6	98.9	68.9	69	90 Max	64 - 74
No. 8	6.2	53.8	94.6	51.2	51	32 - 67	47 - 55
No. 16	4.9	34.2	77.3	38.7	37		
No. 30	4.3	21.5	48.1	28.8	25		
No. 50	3.6	12.5	16.3	18.7	14		
No. 100	2.9	6.5	2.9	12.7	8		
No. 200	2.3	2.9	0.7	9.2	5.0	2.0 - 7.0	3.0 - 7.0

Gsb Coarse	2.686					
Gsb Fine		2.621	2.606			
Gsb Blend	2.686	2.621	2.606	2.738	2.675	
Sand Equivalent (SE)		55	92		69	45 Min
% Uncompacted Voids		48	43		46	44 Min
% Fracture	96				96	90 Min Double Face Fracture

STATE MATERIALS LABORATORY AGGREGATE TEST DATA

Gsb Coarse	2.683			2.715		
Gsb Fine		2.552	2.594		2.568	
Gsb Blend	2.683	2.552	2.594	2.715	**2.644**	
Sand Equivalent (SE)		56	82			45 Min
% Uncompacted Voids					44	44 Min
% Fracture	98					90 Min Double Face Fracture

COMMENTS

Remarks:

Result Code:

Remarks : Reference request was received and processed after paving had taken place.

Test data verified PGS-28 mixed with RAP binder meets PG58H-22 specifed for project.

Kurt R. Williams, P.E.
State Materials Engineer
Steven J. Davis
Bituminous Materials Engineer
Date : 9/20/2018
Phone : (360) 709-5424

Billing Code
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APPENDIX F: LABORATORY SAMPLES NOMENCLATURE

Table 55: Blending Charts Nomenclature

Contract	Materials			Nomenclature
	Binder Grade	Binder	RAP, %	
9145	PG64-28	S1	20	9145-20-6428-S1
	PG64-28	S1	25	9145-25-6428-S1
	PG64-28	S2	20	9145-20-6428-S2
	PG64-28	S2	25	9145-25-6428-S2
	PG58-34	S2	20	9145-20-5834-S2
	PG58-34	S2	25	9145-25-5834-S2
9262/ 9229	PG58H-22	S3	20	9262-20-58H22-S3
	PG58H-22	S3	25	9262-25-58H22-S3
	PG58H-22	S3	36	9262-36-58H22-S3
	PG58S-28	S3	20	9262-20-58S28-S3
	PG58S-28	S3	25	9262-25-58S28-S3
	PG58S-28	S3	36	9262-36-58S28-S3
	PG58H-22	S4	20	9262-20-58H22-S4
	PG58H-22	S4	25	9262-25-58H22-S4
	PG58H-22	S4	36	9262-36-58H22-S4
	PG58S-28	S4	20	9262-20-58S28-S4
	PG58S-28	S4	25	9262-25-58S28-S4
	PG58S-28	S4	36	9262-36-58S28-S4
9231	PG58H-22	S3	20	9231-20-58H22-S3
	PG58H-22	S3	25	9231-25-58H22-S3
	PG58H-22	S3	40	9231-40-58H22-S3
	PG52S-28	S3	20	9231-20-52S28-S3
	PG52S-28	S3	25	9231-25-52S28-S3
	PG52S-28	S3	40	9231-40-52S28-S3
	PG58H-22	S5	20	9231-20-58H22-S5
	PG58H-22	S5	25	9231-25-58H22-S5
	PG58H-22	S5	40	9231-40-58H22-S5
	PG52H-28	S5	20	9231-20-52H28-S5
	PG52H-28	S5	25	9231-25-52H28-S5
	PG52H-28	S5	40	9231-40-52H28-S5
	PG52S-28	S5	20	9231-20-52S28-S5
	PG52S-28	S5	25	9231-25-52S28-S5
	PG52S-28	S5	40	9231-40-52S28-S5

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