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Evaluation of Hybrid Rubber Modified Asphalt Mixtures and Pavements: A Case Study in Virginia

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Abstract:

Several state departments of transportation have recognized the benefits of modified asphalt mixtures in resisting multiple modes of climate- and load-induced distresses in flexible pavements. Throughout the past 50 years, asphalt binders have been modified with various components such as styrene-butadiene-styrene (SBS) polymers, ground tire rubber, chemicals (e.g., acid), recycled engine oils, etc., to achieve the desired properties. Hybrid rubber modified asphalt (HRMA) is an innovative engineered additive derived from ground tire rubber, elastomeric SBS polymers, and additive technologies. HRMA is specifically formulated to improve the high temperature stiffness and elastic properties of performance graded binders and the storage stability of modified binders.

The purpose of this study was to document and assess HRMA field trials constructed in Virginia. This study documented and evaluated the constructability and laboratory performance of two plant-produced HRMA mixtures compared with the Virginia Department of Transportation (VDOT) typical SBS-modified surface mixtures as reference mixtures. No changes from routine established practices in terms of surface preparation, production at the plant, or paving operations were reported.

The four mixtures were evaluated in terms of durability, dynamic modulus, resistance to rutting, and resistance to cracking using multi-level performance tests (basic, intermediate, advanced). All the derived observations indicated that HRMA modification could be as beneficial as regular SBS modification and could provide similar or better performance properties and characteristics for the resultant mixtures.

The study recommends that VDOT consider the use of HRMA surface mixtures as an alternative to the current use of regular SBS-modified surface mixtures on higher-volume facilities. Since the sections evaluated in this study were placed in 2021, the 2-year performance data and corresponding observations are still considered preliminary. Continued monitoring of field performance will be needed to quantify any benefit of HRMA mixtures in comparison with regular SBS-modified surface mixtures. The study also recommends additional field trials with HRMA mixtures for further performance evaluation.

FINAL REPORT

EVALUATION OF HYBRID RUBBER MODIFIED ASPHALT MIXTURES AND PAVEMENTS: A CASE STUDY IN VIRGINIA

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ABSTRACT

Several state departments of transportation have recognized the benefits of modified asphalt mixtures in resisting multiple modes of climate- and load-induced distresses in flexible pavements. Throughout the past 50 years, asphalt binders have been modified with various components such as styrene-butadiene-styrene (SBS) polymers, ground tire rubber, chemicals (e.g., acid), recycled engine oils, etc., to achieve the desired properties. Hybrid rubber modified asphalt (HRMA) is an innovative engineered additive derived from ground tire rubber, elastomeric SBS polymers, and additive technologies. HRMA is specifically formulated to improve the high temperature stiffness and elastic properties of performance graded binders and the storage stability of modified binders.

The purpose of this study was to document and assess HRMA field trials constructed in Virginia. This study documented and evaluated the constructability and laboratory performance of two plant-produced HRMA mixtures compared with the Virginia Department of Transportation (VDOT) typical SBS-modified surface mixtures as reference mixtures. No changes from routine established practices in terms of surface preparation, production at the plant, or paving operations were reported.

The four mixtures were evaluated in terms of durability, dynamic modulus, resistance to rutting, and resistance to cracking using multi-level performance tests (basic, intermediate, advanced). All the derived observations indicated that HRMA modification could be as beneficial as regular SBS modification and could provide similar or better performance properties and characteristics for the resultant mixtures.

The study recommends that VDOT consider the use of HRMA surface mixtures as an alternative to the current use of regular SBS-modified surface mixtures on higher-volume facilities. Since the sections evaluated in this study were placed in 2021, the 2-year performance data and corresponding observations are still considered preliminary. Continued monitoring of field performance will be needed to quantify any benefit of HRMA mixtures in comparison with regular SBS-modified surface mixtures. The study also recommends additional field trials with HRMA mixtures for further performance evaluation.

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INTRODUCTION

Over the last several decades, the Virginia Department of Transportation (VDOT) has evaluated the use and performance of several technologies in asphalt mixtures with the primary aim of enhancing the performance of the roadway network while simultaneously reducing environmental burdens on transportation systems. For example, asphalt mixtures that incorporate warm mix asphalt technologies and highly polymer-modified asphalt binders can support both objectives in various ways. Warm mix asphalt technologies can effectively reduce temperatures and emissions at the plant while aiding in achieving better compaction in the field (Diefenderfer, 2019). Highly polymer-modified asphalt binders can extend the performance life of asphalt materials (Habbouche et al., 2021; Habbouche et al., 2022a). Further, like many other state highway agencies, VDOT is working extensively to determine how best to incorporate recycled materials into their roads. Such materials include recycled plastic waste (Habbouche, 2022); reclaimed asphalt pavement (RAP) at higher contents both with and without recycling agents (Diefenderfer et al., 2021; Diefenderfer et al., 2023); recycled asphalt shingles, recycled tire rubber (Nair and Hossain, 2023a; Nair and Hossain, 2023b), and hybrid rubber.

The practice of modifying asphalt binders is not new and has gained popularity in recent decades. Many state departments of transportation have acknowledged the benefits of using modified asphalt mixtures to resist climate- and load-induced distress in flexible pavements. For the past 50 years, asphalt binders have been modified using various materials, including polymers, ground tire rubber (GTR), chemicals (such as acid), and recycled engine oils, to achieve the desired properties. Compared to unmodified asphalt binders, modifiers typically enhance specific physical properties and rheological performance such as ductility, relaxation spectra, and overall strength (Habbouche et al., 2020).

Over the last two decades, styrene-butadiene-styrene (SBS) polymer modifier, a well-known elastomer, has become more readily available and widely used. The physical cross-linking of SBS molecules into a three-dimensional network can create remarkable strength and elasticity in SBS-modified asphalt binders (Habbouche et al., 2022b). Compared to unmodified binders and those modified with chemically reactive polymers such as polyphosphoric acid, SBS-modified binders have shown improved performance at low temperatures. Further, the use of SBS polymer in asphalt binder could reduce the rate of microdamage accumulation, benefiting cracking resistance (Mohammad et al., 2003). Previous research has demonstrated the ability of polymer modifiers to mitigate the aging effects of oxidative hardening (Roque et al., 2004). As such, the use of polymer-modified asphalt mixtures could produce more durable asphalt pavements.

Asphalt mixtures are often improved by incorporating GTR from scrap tires. Recently, Buttlar and Rath (2021) conducted a comprehensive review of rubber-modified asphalt mixtures, covering their historical development, production methods, field performance, environmental impact, and sustainability benefits. Baumgardner et al. (2020) provided additional information on the use of rubber-modified asphalt mixtures in pavements, revealing that only 12 states have published specifications allowing GTR-modified asphalt binders for use in asphalt pavement construction. Way et al. (2011) reported that blending GTR into asphalt binder can enhance its elasticity and physical properties, leading to improved pavement performance (West et al., 2012; Willis, 2013). Moreover, studies have demonstrated that rubber-modified asphalt mixtures can deliver comparable performance to pavements constructed with SBS polymer-modified binders (Nair and Hossain, 2023a, 2023b).

Hybrid rubber modified asphalt (HRMA) is an innovative engineered additive derived from GTR, elastomeric SBS polymers, and additive technologies (mainly recycling agents / rejuvenators). HRMA is specifically formulated to improve the high temperature stiffness and elastic properties of performance graded binders and the storage stability of modified binders. Binders modified with HRMA can meet specification requirements for polymer-modified binders with dosage rates equal to or less than those of standard modification materials, such as elastomers and plastomers. HRMA can be incorporated into traditional polymer mills at the asphalt terminals. Moreover, the binder can be formulated and dosed at the plant. The resultant binder is characterized by a very high solubility (~99.9%), which makes it a more appealing candidate to deal with when compared to traditional GTR-based asphalt binders. The bitumen formulator can meet the targeted binder grade by adjusting the dosage of the HRMA with consideration of asphalt source and milling method. Typical dosage rates are 7% to 9% by weight of binder. The resultant binder (i.e., HRMA) is characterized with a very high elastic recovery (~85%), high workability, and effective compactibility. This technology has been used in numerous states including Arizona, Florida, Georgia, Illinois, Kentucky, New Hampshire, New Jersey, Ohio, Pennsylvania, Tennessee, Vermont, and Wisconsin with very promising reported laboratory and field performance (Baumgardner et al., 2021).

Currently, VDOT primarily relies on SBS-modified asphalt binders to produce and place asphalt mixtures that resist climate- and load-induced distresses on relatively higher-volume roads and facilities. VDOT has limited experience with GTR-modified asphalt mixtures but no practical experience with designing, producing, and paving HRMA mixtures. With the HRMA technology showing promising results when used in other states, this technology may provide VDOT with another alternative to modify asphalt binders and produce mixtures with a similar or better expected performance when compared to the performance of typical SBS-modified asphalt mixtures.

PURPOSE AND SCOPE

The purpose of this study was to document and assess HRMA field trials constructed in Virginia. This was achieved by documenting and evaluating the constructability, laboratory performance, predicted long-term performance by means of mechanistic-empirical (ME) simulations, and initial field performance of asphalt mixtures modified with HRMA compared with VDOT typical SBS-modified reference mixtures. An objective of this study was to assess the impact of testing on the performance properties of HRMA and their corresponding SBS-modified reference mixtures based on degree of complexity. Evaluating the cost-effectiveness of the HRMA technology was beyond the scope of this study due to limitations in available data.

METHODS

Seven tasks were performed to achieve the study objectives:

1. Document the project selection, mix design, production, and construction processes of SBS-modified reference and HRMA mixtures.

2. Sample asphalt binders and plant-produced loose asphalt mixtures, obtain producer-supplied specimens compacted during production (referred to herein as "non-reheats"), and collect as-paved material (i.e., cores) during construction.

3. Conduct extensive performance testing on virgin asphalt binders sampled at the plant and on asphalt binders extracted and recovered from loose mixtures sampled at the plant during production.

4. Conduct volumetric and multilevel laboratory performance testing on specimens fabricated from non-reheated (i.e., non-reheats) and reheated (referred to herein as "reheats") plant-produced mixtures and perform analyses to evaluate the mixtures.

5. Conduct ME simulations using FlexPave and AASHTOWare Pavement ME Design software (hereinafter "Pavement ME Design") to predict the long-term performance of the HRMA and SBS-modified reference mixtures when placed on typically encountered pavement structures in Virginia.

6. Conduct non-destructive testing using a falling weight deflectometer (FWD), ground-penetrating radar (GPR), and a profilometer to assess the overall pavement structure after paving.

7. Conduct a visual surface distress survey and collect data from VDOT's Pavement Management System (PMS) to assess the in-service conditions of the pavement structures.

Field Trials

Two field trials were planned, developed, and constructed during VDOT's 2021 construction season. Each trial included one HRMA surface mixture (SM) (referred to herein as "SM-HRMA") with one typical "SM-E" mixture (SM with SBS polymer-modified binder) serving as a reference mixture. The SM-HRMA trial mixtures were designed in a way that the HRMA binders (formulated using ~9.0% HRMA content) had to meet VDOT's requirement for a binder with a performance grade (PG) of 64E-22 (VDOT, 2020). The "E" denotes binders meeting the requirement of extremely heavy traffic using the multiple stress creep recovery (MSCR) test in accordance with AASHTO M 332, Standard Specification for Performance-Graded Asphalt Binder Using Multiple Stress Creep Recovery (MSCR) Test (AASHTO, 2020) (typically formulated using ~3.5% SBS content). The research team, the two producers (designated A and B), the HRMA modifier supplier, and the asphalt binder supplier were able to develop two HRMA mix designs, one with a nominal maximum aggregate size (NMAS) of 9.5 mm and the other with an NMAS of 12.5 mm, in accordance with VDOT specifications in terms of gradation and volumetric properties (VDOT, 2020).

The two SM-HRMA and two control SM-E mixtures were paved on continuous routes in VDOT's Northern Virginia District:

- *Route 120 / North Glebe Road (9.5 mm NMAS mixtures by Producer A):* SM-9.5E HRMA was placed on the left lane southbound of Route 120 / North Glebe Road between Arlington Boulevard and North Quincy Street. SM-9.5E (reference mixture) was placed on North Glebe Road Northbound and Southbound.

- *Route 625 / Waxpool Road Between Ruritan Circle and the Route 28 Overpass Bridge (12.5 mm NMAS mixtures by Producer B):* The left lane was paved with SM-12.5E, and the right lane was paved with SM-12.5E HRMA.

These routes were selected from VDOT's 2021 paving contracts and in a way that the long-term performance could be monitored and evaluated in the future. Table 1 summarizes the VDOT HRMA field trials constructed during the 2021 construction season.

The pre-construction conditions of the selected roadway site were documented using VDOT's PMS and a field visit / visual review. Notes/details relevant to the production and lay-down / paving operations were documented. Photographs were taken during production at the plant and during paving operations in the field; some are shown in Figure 1.

Table 1. Hybrid Rubber Modified Asphalt Field Trials (2021 Construction Season)

Producer	Mix Type	Location
A	SM-9.5E: 15% RAP + PG64E-22	Rte. 120 / Glebe Rd. MP 4.82-5.40
	SM-9.5E HRMA: 15% RAP + PG64E-22 (HRMA binder)	
B	SM-12.5E: 15% RAP + PG64E-22	Rte. 625 / Waxpool Rd. MP 8.81-9.85
	SM-12.5E HRMA: 15% RAP + PG64E-22 (HRMA binder)	

SM = surface mixture; E = extremely heavy traffic; RAP = reclaimed asphalt pavement; PG = performance grade; HRMA = hybrid rubber modified asphalt; MP = mile points.

Figure 1. Production Operations at the Plant and Paving/Construction in the Field for HRMA and Reference Mixtures. HRMA = hybrid rubber modified asphalt.

No changes from routine established practices in terms of surface preparation, production at the plant, or paving operations were reported. No practices or enforcements of specific safety, health, or environmental restrictions and no changes to current quality control / quality assurance routine practices were identified/reported with regard to using HRMA binders and mixtures.

Material Sampling

Producer A placed SM-9.5E for multiple nights as part of a regular contract / plant-mix schedule and SM-9.5E HRMA for only 2 nights through a change order. Loose mixture sampling was performed once during one night of production for SM-9.5E and once during each night of production for SM-9.5E HRMA. SM-9.5E and SM-9.5E HRMA were sampled on consecutive placement nights to ensure that the mixtures evaluated in the laboratory were placed on pavement structures with similar conditions. This provided more consistency in comparing their field performance.

Producer B placed SM-12.5E and SM-12.5E HRMA for 1 night each. The production of both mixtures happened through a change order of a plant-mix schedule that called for 12.5 mm NMAS unmodified SM (SM-12.5D). Loose mixture sampling was performed twice during each night per mixture type.

Plant-produced loose mixtures were collected for each mixture type. Loose mixtures were sampled from an approximately 3- to 5-ton quantity of mixture dumped on the ground at the plant and struck off using a loader. Loose plant-produced mixture intended for specimens compacted without reheating at the plant was taken into the producer's laboratory and immediately compacted into specimens (non-reheats). Plant-compacted specimens were provided to the Virginia Transportation Research Council (VTRC) for testing. Table 2 summarizes the sampling and testing matrix for the non-reheat mixtures and specimens.

Table 2. Sampling, Compaction, and Laboratory Testing of Non-Reheats Regular E and HRMA Mixture Specimens

Laboratory Tests, Production Details, and VTRC Log ID	Producer A			Producer B			
	SM-9.5E (Reference)	SM-9.5E HRMA		SM-12.5E (Reference)		SM-12.5E HRMA	
	1 Night	2 Nights		1 Night		1 Night	
	21-1134	Night 1 21-1138	Night 2 21-1140	Set A 21-1123	Set B 21-1124	Set A 21-1128	Set B 21-1129
Mix Design Verification							
Volumetric Properties	✓	✓	✓	✓	✓	✓	✓
Durability Assessment							
Cantabro Test	✓	✓	✓	✓	✓	✓	✓
Assessment of Rutting Performance							
APA Rut Test	✓	✓	✓	✓	✓	✓	✓
Assessment of Cracking Performance							
IDT-CT	✓	✓	✓	✓	✓	✓	✓

Non-Reheats = laboratory-compacted non-reheated specimens that were compacted on-site without reheating of the loose mixture sampled at the plant; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; VTRC = Virginia Transportation Research Council; ID = identification; APA = Asphalt Pavement Analyzer; IDT-CT = indirect tensile cracking test.

The testing on non-reheats included testing for volumetric properties by the producer and the corresponding VDOT district (if available) and VDOT balanced mix design (BMD) tests on short-term aged specimens (i.e., Cantabro test, Asphalt Pavement Analyzer [APA] rut test, and indirect tensile cracking test [IDT-CT]).

Further, loose plant-produced mixtures were placed into boxes, taken to VTRC, and stored in a climate-controlled area until further evaluated. Specimens were then fabricated by reheating the loose mixtures until they became workable, then splitting the material into appropriate quantities, then heating the material to the compaction temperature, and then compacting the material. Table 3 summarizes the testing matrix for the reheats. The testing included testing for volumetric properties, VDOT BMD tests, and additional testing on short- and long-term aged specimens.

Table 3. Sampling, Compaction, and Laboratory Testing of Reheats Regular E and HRMA Mixture Specimens

Laboratory Tests, Production Details, and VTRC Log ID	Producer A			Producer B			
	SM-9.5E (reference)	SM-9.5E HRMA		SM-12.5E (reference)		SM-12.5E HRMA	
	1 night	2 nights		1 night		1 night	
	21-1133	Night 1 21-1136	Night 2 21-1139	Set A 21-1120	Set B 21-1121	Set A 21-1125	Set B 21-1126
Mix Design Verification							
Volumetric Properties	✓	✓	✓	✓	✓	✓	✓
Durability Assessment of Short-Term Aged Asphalt Mixtures							
Cantabro Test	✓	✓	✓	✓	✓	✓	✓
Determination of Mechanical Property of Short-Term Aged Asphalt Mixtures							
Dynamic Modulus |E*| Test	✓	✓	x	✓	x	✓	x
Assessment of Rutting Performance of Short-Term Aged Asphalt Mixtures							
IDT-HT and IDEAL-RT	✓	✓	x	✓	x	✓	x
APA Rut Test	✓	✓	✓	✓	✓	✓	✓
SSR Test	✓	✓	x	✓	x	✓	x
RLT Test	✓	✓	x	✓	x	✓	x
Assessment of Cracking Performance of Short-Term Aged Asphalt Mixtures							
IDT-CT	✓	✓	✓	✓	✓	✓	✓
I-FIT	✓	✓	x	✓	x	✓	x
Direct Tension Cyclic Fatigue Test	✓	✓	x	✓	x	✓	x
Durability Assessment of Long-Term Aged Asphalt Mixtures							
Cantabro Test	x	x	x	✓	✓	✓	✓
Determination of Mechanical Property of Long-Term Aged Asphalt Mixtures							
Dynamic Modulus |E*| Test	✓	✓	x	✓	x	✓	x
Assessment of Cracking Performance of Long-Term Aged Asphalt Mixtures							
IDT-CT	✓	✓	x	✓	x	✓	x
Direct Tension Cyclic Fatigue Test	✓	✓	x	✓	x	✓	x

Reheats = Laboratory-compacted reheated specimens that were compacted after reheating of the loose mixture sampled at the plant; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; VTRC = Virginia Transportation Research Council; ID = identification; SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; IDT-HT = indirect tensile test at high temperature; IDEAL-RT = indirect tensile rutting test; APA = Asphalt Pavement Analyzer; SSR = stress sweep rutting; RLT = repeated load triaxial; IDT-CT = indirect tensile cracking test; I-FIT = Illinois flexibility index test.

Additional testing of short-term aged specimens consisted of the indirect tensile test at high temperature (IDT-HT), indirect tensile rutting test (IDEAL-RT), dynamic modulus (E*) test, stress sweep rutting (SSR) test, repeated load triaxial (RLT) test, Illinois flexibility index test (I-FIT), and direct tension cyclic fatigue test. Moreover, as part of the additional testing, loose mixtures were further aged for 8 hours at 135°C prior to compaction and then the long-term aged performance was evaluated using the dynamic modulus |E*| test, IDT-CT, and direct tension cyclic fatigue test.

BMD Specimen Designations

Up to six types of specimens were prepared by the producer and/or VTRC when BMD performance testing was conducted:

1. *Non-Reheats-VTRC:* laboratory-compacted non-reheated specimens that were compacted on-site by the producer staff without reheating of the loose mixture sampled at the plant and tested in the VTRC laboratory by VTRC staff.

2. *Non-Reheats-P:* Laboratory-compacted non-reheated specimens that were compacted on-site and prepared by the producer staff without reheating of the loose mixture sampled at the plant and tested in the producer laboratory by producer staff.

3. *Reheats-VTRC*: Laboratory-compacted reheated specimens that were compacted and tested in the VTRC laboratory by VTRC staff after reheating of the loose mixture sampled at the plant. Mixtures were not subjected to any additional oven aging.

4. *Reheats-P:* Laboratory-compacted reheated specimens that were compacted and tested in the producer laboratory by the producer staff after reheating of the loose mixture sampled at the plant. Mixtures were not subjected to any additional oven aging.

5. *Reheats-LTOA-VTRC:* Laboratory-compacted reheated specimens that were compacted and tested in the VTRC laboratory by VTRC staff after reheating of the loose mixture sampled at the plant. Mixtures were subjected to additional long-term oven aging by conditioning the loose mixtures for 8 hours at 135°C (275°F) prior to compaction.

6. *Reheats-LTOA-P:* Laboratory-compacted reheated specimens that were compacted and tested in the producer laboratory by the producer staff after reheating of the loose mixture sampled at the plant. Mixtures were subjected to additional long-term oven aging by conditioning the loose mixtures for 8 hours at 135°C prior to compaction.

The testing on non-reheats included testing for volumetric properties by the producer and the corresponding VDOT district (if available) and VDOT balanced mix design (BMD) tests on short-term aged specimens (i.e., Cantabro test, Asphalt Pavement Analyzer [APA] rut test, and indirect tensile cracking test [IDT-CT]).

Further, loose plant-produced mixtures were placed into boxes, taken to VTRC, and stored in a climate-controlled area until further evaluated. Specimens were then fabricated by reheating the loose mixtures until they became workable, then splitting the material into appropriate quantities, then heating the material to the compaction temperature, and then compacting the material. Table 3 summarizes the testing matrix for the reheats. The testing included testing for volumetric properties, VDOT BMD tests, and additional testing on short- and long-term aged specimens. Additional testing of short-term aged specimens consisted of the indirect tensile test at high temperature (IDT-HT), indirect tensile rutting test (IDEAL-RT), dynamic modulus (E*) test, stress sweep rutting (SSR) test, repeated load triaxial (RLT) test, Illinois flexibility index test (I-FIT), and direct tension cyclic fatigue test. Moreover, as part of the additional testing, loose mixtures were further aged for 8 hours at 135°C prior to compaction and then the long-term aged performance was evaluated using the dynamic modulus |E*| test, IDT-CT, and direct tension cyclic fatigue test.

BMD Specimen Designations

Up to six types of specimens were prepared by the producer and/or VTRC when BMD performance testing was conducted:

1. *Non-Reheats-VTRC:* laboratory-compacted non-reheated specimens that were compacted on-site by the producer staff without reheating of the loose mixture sampled at the plant and tested in the VTRC laboratory by VTRC staff.

2. *Non-Reheats-P:* Laboratory-compacted non-reheated specimens that were compacted on-site and prepared by the producer staff without reheating of the loose mixture sampled at the plant and tested in the producer laboratory by producer staff.

3. *Reheats-VTRC*: Laboratory-compacted reheated specimens that were compacted and tested in the VTRC laboratory by VTRC staff after reheating of the loose mixture sampled at the plant. Mixtures were not subjected to any additional oven aging.

4. *Reheats-P:* Laboratory-compacted reheated specimens that were compacted and tested in the producer laboratory by the producer staff after reheating of the loose mixture sampled at the plant. Mixtures were not subjected to any additional oven aging.

5. *Reheats-LTOA-VTRC:* Laboratory-compacted reheated specimens that were compacted and tested in the VTRC laboratory by VTRC staff after reheating of the loose mixture sampled at the plant. Mixtures were subjected to additional long-term oven aging by conditioning the loose mixtures for 8 hours at 135°C (275°F) prior to compaction.

6. *Reheats-LTOA-P:* Laboratory-compacted reheated specimens that were compacted and tested in the producer laboratory by the producer staff after reheating of the loose mixture sampled at the plant. Mixtures were subjected to additional long-term oven aging by conditioning the loose mixtures for 8 hours at 135°C prior to compaction.

Asphalt Binder Testing and Characterization

Performance Grading

Asphalt binder performance grading was performed in accordance with AASHTO M 320, Standard Specification for Performance-Graded Asphalt Binder (AASHTO, 2017), and AASHTO M 332, Standard Specification for Performance-Graded Asphalt Binder Using Multiple Stress Creep Recovery (MSCR) Test (AASHTO, 2019). Testing was performed on traditional E and HRMA binders collected from the contractor's tanks during production and on extracted and recovered (hereinafter "E&R") asphalt binders from the mixtures collected at the plant. Extraction of asphalt binder from collected mixtures was performed in accordance with AASHTO T 164, Quantitative Extraction of Asphalt Binder From Hot Mix Asphalt (HMA), Method A, using n-propyl bromide as the solvent (AASHTO, 2018). The asphalt binder was then recovered from the solvent using the Rotavap recovery procedure specified in AASHTO T 319, Quantitative Extraction and Recovery of Asphalt Binder From Asphalt Mixtures (AASHTO, 2019).

Difference in Critical Low Temperature Performance Grade (ΔT_c)

The difference in critical low temperature PG limiting temperatures, commonly referred to as ΔT_c, was calculated by subtracting the m-critical low temperature ($T_{c,m}$) from the S-critical low temperature ($T_{c,S}$), as shown in Equation 1 (Federal Highway Administration [FHWA], 2021a). Both temperatures were determined using the bending beam rheometer in accordance with AASHTO T 313, Standard Method of Test for Determining the Flexural Creep Stiffness of Asphalt Binders Using the Bending Beam Rheometer (BBR) (AASHTO, 2019). The m-critical low temperature ($T_{c,m}$) is the resulting low temperature at which the creep relaxation m-value at 60 seconds of loading is exactly equal to the specification value of 0.300. The S-critical low temperature ($T_{c,S}$) is the resulting low temperature at which the creep stiffness S-value at 60 seconds of loading is exactly equal to the specification value of 300 MPa.

$$\Delta T_c = T_{c,S} - T_{c,m} \qquad \text{[Eq. 1]}$$

Frequency Sweep

Frequency sweep tests were conducted to evaluate the traditional E and HRMA asphalt binders sampled from the contractor's tanks during production over multiple frequencies and temperatures in terms of dynamic shear modulus (G*) and phase angle (δ) master curves. The induced strains were monitored and kept within the linear viscoelastic region. Testing was performed on binder specimens at temperatures of 45°C, 55°C, 65°C, 75°C, and 85°C using a 25-mm-diameter plate with a 1-mm gap. In addition, testing was performed on binder specimens at temperatures of 5°C, 15°C, 25°C, 35°C, and 45°C using an 8-mm-diameter plate with a 2-mm gap. The two results measured at 45°C were used for verification purposes, as no differences are usually observed at this temperature regardless of the specimen geometry used (8-mm or 25-mm diameter). All specimens were evaluated at 16 frequencies ranging from 0.1 to 100 rad/s at each testing temperature. No standard currently exists for the construction of a binder master curve. In this study, the rheological software package Rheology Analysis Software (RHEA) was used to perform the shifting of the G* master curves to a reference temperature of 45°C (Abatech, 2022). The software adopts the method of free shifting to fit the frequency sweep measured data into a smooth master curve. The term "free shifting" indicates that the measured data are shifted to a master curve without a predefined shape function (Habbouche et al., 2022b).

Glover-Rowe Parameter

The Glover-Rowe (G-R) parameter was originally defined by Glover et al. (2005) and reformulated for greater practical use by Rowe et al. (2011) in a discussion (Anderson et al., 2011). The G-R parameter is determined at a temperature of 15°C and a frequency of 0.005 rad/s and is expressed using Equation 2.

$$G - R = \frac{G^*(\cos\delta)^2}{\sin\delta} \qquad \text{[Eq. 2]}$$

where

G^* = complex dynamic shear modulus, Pa
δ = phase angle, °.

The G-R parameter captures both rheological parameters needed to characterize binder viscoelastic behavior: stiffness (represented by the complex shear dynamic modulus G*) and relaxation (represented by the phase angle δ). G-R parameter values at this temperature and frequency have been shown to correlate well with ductility, thus indicating cracking resistance and binder oxidation levels (Ruan et al., 2003). The G-R parameter refers to non-load cracking at intermediate temperature, and its limits relate to specific environmental conditions. The universal limits of the G-R parameter, 180 kPa and 600 kPa, are usually used as a reference to track the effect of aging and/or rejuvenation. However, there have also been limitations with the G-R parameter measured in the dynamic shear rheometer at intermediate temperatures, particularly when correlations with modified binders were attempted (Glover et al., 2005).

Linear Amplitude Sweep (LAS) Test

The linear amplitude sweep (LAS) test was performed in accordance with AASHTO TP 101, Estimating Fatigue Resistance of Asphalt Binders Using the Linear Amplitude Sweep, to investigate the fatigue damage characterization of the evaluated binders at an intermediate temperature of interest (AASHTO, 2018). The test included a frequency sweep test at 0.1% strain over a range of frequencies from 0.2 to 30 Hz followed by an amplitude sweep oscillatory shear in strain-control mode test at a frequency of 10 Hz over a range of induced strains from 0.1% to 30%. The test was conducted at 23°C, the average of the high and low PG temperatures minus 4°C for the majority of the evaluated binders. This temperature was also selected such that the linear complex shear modulus G* fell within the range of 12 to 60 MPa at 10 Hz to mitigate any potential edge flow and/or adhesion loss (Safaei and Castorena, 2016). The binder fatigue performance parameter Nf is calculated using Equation 3.

$$N_f = \mathrm{A} * (\Upsilon_{max})^{-B} \quad \text{[Eq. 3]}$$

where

N_f = fatigue performance parameter, number of cycles to fatigue failure
Υ_{max} = maximum expected binder strain for a given pavement structure, %
A and B = modeling parameters associated with fatigue resistance of the binder.

Asphalt Mixtures Testing and Characterization

Volumetric Properties and Aggregate Gradations of Mixtures

The theoretical maximum specific gravity of each mixture was determined in accordance with AASHTO T 209, Standard Method of Test for Theoretical Maximum Specific Gravity (G_{mm}) and Density of Asphalt Mixtures (AASHTO, 2020). The asphalt binder content of each mixture was determined by the ignition method in accordance with AASHTO T 308, Standard Method of Test for Determining the Asphalt Binder Content of Asphalt Mixtures by the Ignition Method (AASHTO, 2021), and Virginia Test Method (VTM) 102, Determination of Asphalt Content From Asphalt Paving Mixtures by the Ignition Method (Virginia Test Methods, 2013a). The size distribution (gradation) of the recovered aggregate was determined in accordance with AASHTO T 11, Standard Method of Test for Materials Finer Than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing (AASHTO, 2020), and AASHTO T 27, Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates (AASHTO, 2020). Loose mixtures were conditioned at the compaction temperature and then compacted to N_{design} gyrations using a Superpave gyratory compactor (SGC) in accordance with AASHTO T 312, Preparing and Determining the Density of Asphalt Mixtures Specimens by Means of the Superpave Gyratory Compactor (AASHTO, 2019). Basic physical characteristics and volumetric parameters in terms of bulk specific gravity (G_{mb}), voids in total mixture, voids in mineral aggregate, voids filled with asphalt, fines to aggregate ratio, aggregate effective specific gravity, aggregate bulk specific gravity, absorbed asphalt binder content, effective asphalt binder content, and effective film thickness were determined.

Cantabro Mass Loss

The Cantabro mass loss was determined to evaluate the durability of asphalt mixtures in accordance with AASHTO TP 108, Standard Method of Test for Abrasion Loss of Asphalt Mixture Specimens (AASHTO, 2021). The test was performed on specimens fabricated using an SGC that were compacted from loose mixture collected at the plant during production. The loose mixtures were conditioned at the design compaction temperature prior to compaction to N_{design} gyrations. The Cantabro test specimens were 150 mm in diameter by 115 ± 5 mm in height. The test was performed by placing the specimen into an uncharged Los Angeles abrasion machine and rotating it for 300 rotations at a speed of approximately 30 rotations per minute. Three replicates were tested for each mixture, and an average mass loss was reported.

Dynamic Modulus |E*|

The dynamic modulus (E*) of specimens compacted from loose mixtures collected during production was determined using the Asphalt Mixture Performance Tester (AMPT) with a 25 to 100 kN loading capacity in accordance with AASHTO T 342, Standard Method of Test for Determining Dynamic Modulus of Hot Mix Asphalt (HMA) (AASHTO, 2019). Tests were performed on specimens 100 mm in diameter by 150 mm in height cored from the center of specimens 150 mm in diameter by 175 mm in height compacted using an SGC to 7.0 ± 0.5% air voids. Three testing temperatures (4, 20, and 45°C) and six testing frequencies ranging from 0.01 to 25 Hz were used. Tests were conducted starting from the coldest to the warmest temperature, and at each test temperature, the tests were performed starting from the highest to the lowest frequency. Load levels were selected in such a way that at each temperature-frequency combination, the applied strain was 75 to 125 microstrains. All tests were conducted in the uniaxial mode without confinement. Results at each temperature-frequency combination for each mixture type were reported for three replicate specimens.

Rutting Performance Tests

IDT-HT and IDEAL-RT

The IDT-HT was conducted by applying a constant rate of axial displacement on the diametrical plane of the test specimens. The test was conducted at 54.4°C and a loading rate of 50 ± 2 mm/min (Boz et al., 2023). The specimens were fabricated at a diameter of 150 mm and a height of 62 mm at 7 ± 0.5% target air voids. This test is currently being used as part of the BMD efforts in Virginia for A and D SMs (subjected to a traffic of 1.0 to 3.0 million equivalent single-axle loads [ESALs] and 3.0 to 10.0 million ESALs, respectively). The specimens were conditioned for 2 hours at the test temperature before being tested. Once the IDT-HT was completed, the indirect tensile strength of specimens was determined using Equation 4. A high strength value indicates a good resistance to rutting.

$$S_t = \frac{2000P}{\pi t D} \qquad \text{[Eq. 4]}$$

where

S_t= indirect tensile strength, kPa
P = maximum load, N
t = specimen thickness, mm
D = specimen diameter, mm.

The IDEAL-RT, also known as the rapid rutting test, is a monotonic-loading rutting test that was recently proposed by researchers at the Texas Transportation Institute. The IDEAL-RT is conducted in a similar manner as the IDT-HT except that a shear fixture is used in lieu of a typical IDT-HT fixture. The rutting potential of asphalt mixtures from the IDEAL-RT is quantified through the rutting tolerance (RT) index (RT_{index}), as shown in Equation 5. A higher RT index indicates a good resistance to rutting.

$$RT_{index} = 6.618 * 10^{-5} * 0.356 * \frac{P}{(t * w)} \qquad \text{[Eq. 5]}$$

where

P = peak load, N
t = specimen thickness, m
w = width of upper loading strip, 0.0191 m.

APA Rut Test

The APA rut test was performed on specimens prepared from loose mixture collected during construction in accordance with AASHTO T 340, Standard Method of Test for Determining the Rutting Susceptibility of Hot Mix Asphalt (APA) Using the Asphalt Pavement Analyzer (APA) (AASHTO, 2019). This test simulates rutting in the laboratory by applying a loaded wheel back and forth over a pressurized rubber tube located along the surface of the test specimen at a temperature of 64°C. After 8,000 cycles were applied, the deformation of the specimen was measured. The APA rut test was performed on specimens 150 mm in diameter by 75 ± 2 mm in height compacted using an SGC to 7 ± 0.5% air voids.

Stress Sweep Rutting (SSR) Test

The SSR test assesses the rutting susceptibility of asphalt mixtures by applying repeated cyclic loading to confined cylindrical test specimens at two temperatures in accordance with AASHTO TP 134, Standard Method of Test for Stress Sweep Rutting (SSR) Test Using Asphalt Mixture Performance Tester (AMPT) (AASHTO, 2021). The low temperature and high temperature were determined for the project location using LTPPBind, Version 3.1, at the location of interest. Confined specimens were loaded for 200 cycles each at three increments of deviator stress. The SSR test results were used to calculate the average permanent strain (in percent) and produce the rutting strain index (RSI) parameter calculated by the FlexMAT rutting analysis (FHWA, 2021b). Test results were also used to generate a permanent strain shift model that can be used with the FlexPAVE analysis to model rutting in the pavement layer. A lower RSI indicates a relatively more resistance to rutting (FHWA, 2021b).

Repeated Loading Triaxial (RLT) Test—Confined Flow Number

The rutting characteristics of specimens prepared from loose mixture collected during construction were evaluated using the RLT test in accordance with NCHRP Report 719, *Calibration of Rutting Models for Structural and Mix Designs* (Von Quintus et al., 2012). The RLT test specimens were 100 mm in diameter by 150 mm in height and were cored from the center of an SGC specimen 150 mm in diameter by 175 mm in height. All test specimens were compacted to an air void level of 7.0 ± 0.5%. The RLT test was conducted by applying a repeated deviator stress of 482 kPa, a static confining pressure of 69 kPa, and a contact stress of 24 kPa. The deviator stress was applied through a pulse load with repeated loading and unloading periods. Each loading cycle consisted of 0.1 second of loading followed by a rest period of 0.9 seconds. The axial deformation after each pulse was measured, and the axial resilient strain (ε_r) was calculated. In addition, the cumulative permanent strain (ε_p) was calculated. The RLT test was conducted at three different temperatures: 30, 40, and 50°C. The Franken model, expressed in Equation 6, was used to model the permanent strain-loading cycle relationship numerically (Von Quintus et al., 2012). This well-suited mathematical model combines a power model, which characterizes the primary and secondary stages, and an exponential model, which fits the tertiary stage.

$$\varepsilon(N)=A*N^{B}+C*(e^{D*N}-1) \qquad \text{[Eq. 6]}$$

where

$\varepsilon(N)$ = permanent axial strain expressed in mm/mm
N = number of loading cycles
A, B, C, and D = regression constants.

Cracking Performance Tests

Indirect Tensile Cracking Test (IDT-CT)

The IDT-CT was conducted at 25°C on specimens prepared from loose mixture collected during construction in accordance with ASTM D8225-19, *Standard Test Method for Determination of Cracking Tolerance Index of Asphalt Mixture Using the Indirect Tensile Cracking Test at Intermediate Temperature* (ASTM, 2019). Tests were performed at a loading rate of 50 ± 2 mm/min on specimens 150 mm in diameter by 62 mm in height compacted with a SGC to 7 ± 0.5% air void content. The cracking tolerance (CT) index was then calculated from the test load-displacement curve using Equation 7. A higher CT index value indicates a better resistance to cracking.

$$CT\ index = \frac{G_f}{|m_{75}|} * \left(\frac{l_{75}}{D}\right) * \left(\frac{t}{62}\right) \qquad \text{[Eq. 7]}$$

$$m_{75} = \left|\frac{p_{85}-p_{65}}{l_{85}-l_{65}}\right| \qquad \text{[Eq. 8]}$$

where

CT index = cracking tolerance index expressed in Equation 7
G_f = total area under the load-displacement curve divided by the product of the specimen thickness [t] and diameter [D], kN/mm
m_{75} = slope of interest expressed in Equation 8
p_{85} = 85% of the peak load (P_{max}) at the post-peak stage, kN
p_{75} = 75% of P_{max} at the post-peak stage, kN
p_{65}= 65% of P_{max} at the post-peak stage, kN
l_{85} = displacement corresponding to p_{85}, mm
l_{75} = displacement corresponding to p_{75}, mm

l_{65} = displacement corresponding to p_{65}, mm
D = specimen diameter, mm
t = specimen thickness, mm.

Illinois Flexibility Index Test (I-FIT)

An additional cracking test, the I-FIT, was conducted in accordance with AASHTO TP 124, Standard Method of Test for Determining the Fracture Potential of Asphalt Mixtures Using Semicircular Bend Geometry (SCB) at Intermediate Temperature (AASHTO, 2020). Tests were conducted at 25ºC. All specimens had air voids within 7.0 ± 0.5%. Similar to the IDT-CT, the I-FIT derives from the crack growth theory. The main output of the I-FIT is the flexibility index (FI). The FI is calculated using the slope of the post-peak curve at the inflection point (m) and the fracture energy (G_f) as shown in Equation 9. The calculated FI is an index that shows an asphalt mixture's overall capacity to resist damage related to cracking. The higher the FI, the better a mixture can resist crack propagation under tensile stress.

$$FI = 0.01\frac{G_f}{m} \quad \text{[Eq. 9]}$$

where

G_f = fracture energy, kN/mm
m = slope of the post-peak curve at the inflection point.

Texas Overlay Test (OT)

The Texas overlay test (OT) was performed only on field cores in accordance with Tex-248-F, *Test Procedure for Overlay Test*, to evaluate the mixtures' resistance to reflective cracking (Texas Department of Transportation, 2019). The horizontal opening and closing of joints and cracks that existed underneath a new asphalt overlay were specifically simulated. The OT fixture was designed to increase the functionality of the AMPT by enabling it to determine the reflective cracking susceptibility of asphalt mixtures. The typical OT specimens were 150 mm long by 75 mm wide. Although the procedure calls for a thickness of 37.5 mm for Laboratory-made specimens, the samples tested in this study had a thickness equal or lower to 37.5 mm, depending on the cores from which they were trimmed. Once prepared, each OT specimen was glued on two metallic plates using epoxy. The OT test was conducted in a controlled displacement mode at a loading rate of 1 cycle per 10 seconds with a maximum displacement of 0.6350 mm at 25 ± 0.5°C. Each cycle consisted of 5 seconds of loading and 5 seconds of unloading. The number of cycles to failure was defined as the number of cycles to reach a 93% drop in initial load, which is measured from the first opening cycle. If a 93% reduction in initial load is not reached within a specified number of cycles (5,000), the test stops automatically.

A power function defined in Equation 10 was used to fit the load reduction curve function of the number of loading cycles to determine the crack propagation rate (CPR) (Garcia et al., 2016). The critical fracture energy (G_c) at the maximum peak load of the first loading cycle was determined using Equation 11. G_c was considered the energy required to initiate cracking.

$$NL = N^{CPR} \quad \text{[Eq. 10]}$$

$$G_c = \frac{W_c}{b*c} \quad \text{[Eq. 11]}$$

where

NL = normalized crack driving force or load at each loading cycle, kN
N = number of loading cycles
CPR = crack propagation rate
G_c = critical fracture energy, kN-mm^2
W_c = fracture area at the maximum peak load of the first loading cycle
b = specimen width, mm
h = specimen height, mm.

Direct Tension Cyclic Fatigue Test

The simplified viscoelastic continuum damage test, known as the direct tension cyclic fatigue test, was performed using the AMPT in accordance with AASHTO TP 107, Standard Method of test for Determining the Damage Characteristic Curve and Failure Criterion Using the Asphalt Mixture Performance Tester (AMPT) Cyclic Fatigue Test (AASHTO, 2021). The cyclic fatigue test was performed on specimens 100 mm in diameter by 130 mm in height cored from samples 150 mm in diameter by 175 mm in height compacted from loose mixtures collected during construction. All test specimens were compacted to 7.0 ± 0.5% air voids. The developed damage characteristic curves were then used with the viscoelastic material properties (i.e., |E*|) to obtain the fatigue behavior of the asphalt mixtures. To define the fatigue performance, a fatigue cracking index parameter, referred to as the apparent damage capacity (S_{app}), is usually used. The calculation of S_{app} was conducted with FlexMAT for Cracking, an Excel-based tool provided by the FHWA (FHWA, 2019). The higher the S_{app} value, the more resistance to cracking.

Evaluation of Field Cores

Field core samples were collected from each project during construction. Core locations were randomly stratified along the length and width of the section. The following properties were measured for each core: in-place layer thickness, air voids, permeability in accordance with VTM-120 (Virginia Test Methods, 2000), and resistance to cracking by means of the IDT-CT in accordance with ASTM D8225-19 (ASTM, 2019) and the OT in accordance with Tex-248-F (Texas Department of Transportation, 2019).

Mechanistic-Empirical Simulations

Laboratory-measured performance metrics and mixture volumetric properties can be coupled in a full mechanistic analysis framework. This is a vital step to quantify and evaluate the impact of using regular E and HRMA mixtures on the overall performance of pavements. Two well-known frameworks were considered in this study: FlexPave and Pavement ME Design.

FlexPave is a specialized finite element program designed to predict the performance of asphalt pavement throughout its lifespan, specifically regarding fatigue and rutting. To achieve accurate predictions, the program considers various factors, including the three-dimensional response of moving loads, viscoelastic material properties, and climate data. In addition, the program imports data from FlexMAT cracking and FlexMAT rutting, volumetric information, and details on unbound materials, enabling it to establish materials equivalent to those used on typical pavement structures in Virginia. FlexPave also integrates a pavement temperature database, which accommodates the weather conditions on the site.

The *Guide for the Mechanistic-Empirical Design of New & Rehabilitated Pavement Structures* (hereinafter "the MEPDG") (Applied Research Associates, Inc., 2004) Was written as part of NCHRP Project 1-37A with the objective of providing the highway community with a cutting-edge tool for designing new and rehabilitated pavement structures. The MEPDG employs a calculated mechanistic response, combined with empirical results from pavement test sections in the Long-Term Pavement Performance Program, to predict the performance of pavement structures. The MEPDG determines pavement responses, such as stresses, strains, and deflections, based on inputs such as traffic, climate, and material parameters, to forecast the pavement damage over time for asphalt pavements. Subsequently, transfer functions relate computed pavement responses, such as pavement damage, to observed pavement distresses. The MEPDG principles were integrated into Pavement ME Design.

Two pavement structures were considered in this study. Structure (i), shown in Figure 2a, simulates a typical pavement structure in Virginia. This structure consisted of three asphalt layers: a 2.0-inch SM layer, a 2-inch intermediate mixture (IM) layer, and a 7-inch base mixture (BM) layer.

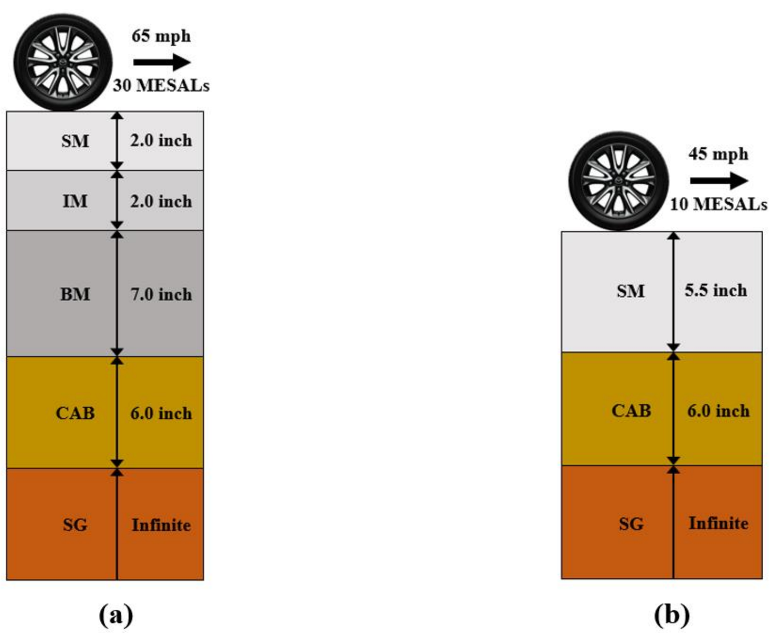


Figure 2. Pavement Structures for Mechanistic-Empirical Simulations: (a) Structure (i); (b) Structure (ii). SM = surface mixture; IM = intermediate mixture; BM = base mixture; CAB = crushed aggregate base; SG = subgrade; MESALs = million equivalent single-axle loads.

The structure was built on top of a 6-inch 21-B base layer and an infinite subgrade. Structure (ii), shown in Figure 2b, simulates an experimental test section with a 5.5-inch SM on top of a 6-inch 21 B base layer and infinite subgrade. The simulation results will enable an extensive exploration and comprehension of the performance of traditional E and HRMA mixtures beyond the laboratory level and extending to the structural level under simulated traffic loading.

Non-Destructive Pavement Testing

Ride Quality, Rut Depth, and Mean Profile Depth Testing

Ride quality, rut depth, and mean profile depth (MPD) data were collected using a VDOT pavement profiler (South Dakota type) in accordance with VTM-106, Determining Pavement Roughness and Rut Depth Using an Accelerometer Established Inertial Profile Referencing System (Virginia Test Methods, 2013b). MPD is a measure of macrotexture that can be calculated from a pavement profile in accordance with ASTM E1845, Standard Practice for Calculating Paving Macrotexture Mean Profile Depth (ASTM, 2015; Flintsch et al., 2003). The MPD is defined as the difference in height between the pavement profile and a horizontal line passing through the top of the highest peak. The MPD typically ranges from 400 to 2,500 microns (0.4 to 2.5 mm) for asphalt pavement surfaces. High values for the MPD generally indicate a higher percentage of aggregate with a positive texture (Rada et al., 2013). Pavement surface texture influences many different pavement-tire interactions. Good skid resistance results from controlling the microtexture and macrotexture of a pavement surface.

Falling Weight Deflectometer (FWD) Testing

FWD testing to assess structural capacity was performed in accordance with ASTM D4694, Standard Test Method for Deflections From a Falling-Weight-Type Impulse Load Device (ASTM, 2020). Deflection testing was conducted at four load levels (6,000; 9,000; 12,000; and 16,000 lbf) using a 250-ft spacing. Following two unrecorded seating drops, four deflection basins were recorded at each load level.

Ground Penetrating Radar (GPR) Testing

GPR testing was conducted with a 2 GHz horn antenna and an SIR-30 computer manufactured by GSSI. The vehicle was equipped with an electronic distance measuring instrument mounted to the rear wheel, providing synchronous distance data as the GPR data were collected, and a GPS unit, providing high-resolution, differentially corrected geospatial information. The data collection and recording were controlled by the SIR-30 GPR system operated from within the survey vehicle. The data were collected at a rate of 1 scan per foot of travel. GPR data were processed with RADAN 7 software.

Distress Survey

A visual distress evaluation was conducted before paving, and Figure 3 shows the pavement surface on Route 120 / Glebe Road. This section was an existing composite pavement (asphalt over jointed concrete pavement), and the distress on this section before paving included transverse reflective cracks, longitudinal cracks (both on and outside the wheel path), and some localized alligator cracks. The section on Route 625 / Waxpool Road was an asphalt pavement, and the major distresses on the existing pavement included cracking and longitudinal joint deterioration.

A visual distress survey was conducted periodically for the two sections (Route 120 / Glebe Road and Route 625 / Waxpool Road) after paving. An early-life performance baseline was also established through VDOT's PMS after the mixtures were placed. The surface distresses collected for VDOT's PMS included transverse cracking, longitudinal cracking, reflective transverse cracking, reflective longitudinal cracking, alligator cracking, longitudinal joint cracking, patching, potholes, delamination, bleeding, and rutting. In VDOT's PMS, three condition indices are used to rate pavement sections based on the observed distresses. The first is the load related distress rating (LDR), which measures pavement distresses caused by traffic loading. The second is the non-load related distress rating (NDR), which measures pavement distresses that are not load related, such as those caused by environmental or climatic conditions. These two condition indices range from 0 to 100, where 100 signifies a pavement having no distresses. The third is the critical condition index (CCI), which is the lesser of the LDR and the NDR. In addition to storing the individual distress data, VDOT's PMS calculates and stores the LDR, the NDR, the CCI, and the international roughness index (IRI) for all sections.

Figure 3. Reflective Cracking on the Route 120 / Glebe Road Existing Pavement Section

RESULTS AND DISCUSSION

Laboratory Evaluation of Asphalt Binder

Performance Grading

Table 4 summarizes the properties of the original binders and their corresponding E&R asphalt binders. All four original binders met VDOT specifications (VDOT, 2020). It should be noted that typical GTR-modified binders are known to show relatively higher viscosities when compared to typical E SBS-modified binders. However, in this case, the HRMA binders exhibited relatively lower viscosity, thus highlighting their high workability and the effective compactability of the corresponding mixtures. The same batch of HRMA binder was sent to the two producers at the same time. The HRMA binder was stored in the contractors' tanks until completion of the job. SM-9.5E HRMA was produced about 3 weeks after SM-12.5E HRMA due to inclement weather. No changes were observed due to storage in the properties of the original binders and those after aging using a pressurized aging vessel (PAV) for 20 hours. A slight decrease in the MSCR properties (i.e., higher non-recoverable creep compliance at 3.2 kPa $J_{nr3.2}$ and lower percent recovery [%R]) was observed due to the extended storage. The E&R process and the inclusion of 15% RAP in the mixture have caused changes in the binder properties when compared to the original binders.

Table 5 presents the $T_{c,S}$, $T_{c,m}$, and ΔT_c values for all evaluated original and E&R binders after 20 hours PAV and for the original binders after 40 hours PAV. All original binders had ΔT_c values ranging from −1.9°C to −1.3°C and −4.7°C to −3.2°C after 20 hours and 40 hours PAV, respectively, with none exceeding the cracking warning limit of −2.5°C after 20 hours PAV and the traditional cracking zone of −5.0°C after 40 hours PAV (Diefenderfer et al., 2023; Yang et al., 2022). This indicates a promising and comparable resistance of HRMA binders to non-load related cracking. A decrease in ΔT_c values was observed for the E&R binders when compared with the original binders, which can be attributed to the E&R process and the inclusion of 15% RAP content in the produced mixtures.

Aging Assessment of Evaluated Asphalt Binders

The G* and δ master curves of virgin E and HRMA asphalt binders were determined by means of frequency sweep tests at various testing temperatures and loading frequencies. Four aging conditions were considered: original; short-term aged by means of a rolling thin film oven (RTFO) (hereinafter "RTFO"); and long-term aged for 20 or 40 hours using a PAV (referred to herein as "20-hour PAV" and "40-hour PAV," respectively). Figure 4 shows the G* and δ master curves of the virgin binders for SM-9.5E, SM-9.5 E HRMA, SM-12.5E, and SM-12.5E HRMA after 20-hour PAV long-term aging. The G* and δ master curves for the same evaluated binders at the remaining aging stages are provided in Appendix A.

Table 4. Properties of Evaluated E and HRMA Asphalt Binders (Original and Extracted/Recovered)

Property	Test Results								Specification[a]	Test Method
	SM9.5E		SM9.5E HRMA		SM-12.5E		SM-12.5E HRMA			
	Original	E&R	Original	E&R	Original	E&R	Original	E&R		
Original Binder										
Viscosity at 135°C, Pa.s	--	--	1.663	--	--	--	1.663	--	Max. 3.000[b]	AASHTO T 136
Dynamic Shear, G*/sinδ at 76°C and 10 rad/s, kPa	1.64	--	1.46	--	1.33	--	1.40	--	Min. 1.000	AASHTO T 315
Rolling Thin Film Oven (RTFO) Residue, AASHTO T 240										
Mass Loss, %	-0.267	--	-0.377	--	-0.279	--	-0.361	--	|Max.| 1.00	AASHTO T 240
Non-Recoverable Creep Compliance,[c] $J_{nr\,3.2}$ at 3.2 kPa at 64°C, kPa^{-1}	0.13	0.36	0.27	0.81	0.19	0.31	0.19	0.21	Max. 0.5	AASHTO T 350
Non-Recoverable Creep Compliance Difference,[c] $J_{nr\,diff}$ at 3.2 kPa at 64°C, %	37.8	22.8	38.5	35.5	30.3	18.1	32.9	14.1	--	AASHTO T 350
Creep Recovery,[c] $R_{3.2}$ at 3.2 kPa at 64°C, %	75.1	57.2	56.3	31.5	67.6	50.4	62.8	53.3	--	AASHTO T 350
Pressure Aging Vessel (PAV) Residue, AASHTO R 28										
Dynamic Shear G*sinδ, max. 6,000 kPa, test temp. at 10 rad/s, °C	20.9	20.1	18.4	17.9	23.8	23.3	19.5	21.1	--	AASHTO T 315
Creep Stiffness[d] at 60 s, S, test temp. −12°C, Mpa	156	130	114	102	190	135	120	123	Max. 300	AASHTO T 313
Creep Relaxation[d] at 60 s, m-value, test temp. −12°C, Mpa	0.331	0.332	0.352	0.347	0.317	0.302	0.340	0.318	Min. 0.300	AASHTO T 313
Continuous Grade										
Performance Grade	80.8-25.9	76.3-26.6	79.3-28.3	73.7-27.8	79.1-24.1	79.5-22.5	79.6-27.7	84.4-24.5	--	AASHTO M 320
Performance Grade	64E-22	64E-22	64E-22	64E-22	64E-22	64E-22	64E-22	64E-22	--	AASHTO M 322

Red text indicates value that did not meet allowable listed binder requirements. E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; E&R = extracted and recovered; -- = data not available.

[a] Specifications apply only for original binders sampled from the contractors' tanks.

[b] The viscosity must be less than or equal to 3.0 Pa.s; however, the Engineer may increase the viscosity limit to 5.0 Pa.s if the binder supplier and contractor agree that the binder is suitably workable.

[c] Multiple stress creep recovery (MSCR) test on RTFO residue should be performed at the performance grade based on the environmental high pavement temperature (i.e., 64°C for E and HRMA binders).

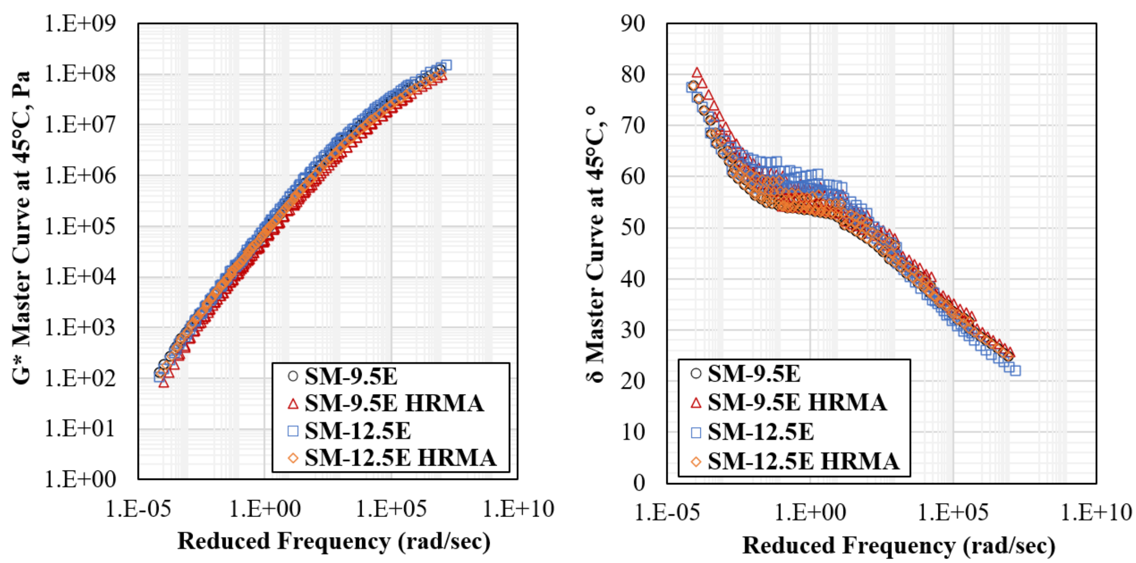
[d] Testing temperature is 10°C warmer than the actual low performance grade.

Table 5. Summary of Critical Low Temperatures for All Evaluated Original and Extracted/Recovered (E&R) Asphalt Binders

Property	Mixture ID							
	SM-9.5E		SM-9.5E HRMA		SM-12.5E		SM-12.5E HRMA	
	Original	E&R	Original	E&R	Original	E&R	Original	E&R
After 20 Hours PAV								
$T_{c,S}$, °C	-27.2	-28.6	-30.0	-29.9	-26.0	-28.2	-29.5	-29.6
$T_{c,m}$, °C	-25.9	-26.6	-28.3	-27.8	-24.1	-22.5	-27.7	-24.5
ΔTc, °C	-1.3	-2.0	-1.7	-2.2	-1.9	-5.7	-1.8	-5.2
After 40 Hours PAV								
$T_{c,S}$, °C	-26.1	--	-28.9	--	-24.8	--	-28.2	--
$T_{c,m}$, °C	-22.3	--	-25.7	--	-20.1	--	-25.0	--
ΔTc, °C	-3.8	--	-3.2	--	-4.7	--	-3.2	--

E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; PAV = pressure aging vessel; $T_{c,S}$ = stiffness critical low temperature; $T_{c,m}$ = m-value critical low temperature; -- = data not available.

All evaluated binders exhibited typical shapes of G* master curves. The stiffness (G*) increased with the increase in frequency for all considered frequency values (low, intermediate, and high). Moreover, regardless of the aging temperature, the trajectory of the G* master curves for the same binder evaluated at different aging conditions showed similar values for the glassy modulus; however, no specific work was done to confirm this observation. Higher G* values were observed with the increase in aging temperature and aging duration irrespective of the evaluated asphalt binder (i.e., regular E vs. HRMA), simulating more potential oxidation of the asphalt binder and brittleness of the corresponding mixture in the field when subjected to warmer climatic conditions. As seen in Figure 4a, the HRMA binder exhibited slightly lower G* values than its reference E binders regardless of the testing frequency or loading rate.



(a) (b)

Figure 4. Performance Test Data in Terms of Master Curves at 45°C for All Evaluated Asphalt Binders at 20-Hour PAV Aging Conditions: (a) Dynamic Shear Modulus (G*); (b) Phase Angle (δ). SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; PAV = pressure aging vessel.

All evaluated binders exhibited typical shapes of δ master curves for SBS-modified asphalt binders. At higher frequencies, lower phase angle values were observed, simulating more elastic behavior for the asphalt binders at lower temperatures, at which the binders are more susceptible to cracking. With the decrease of the reduced frequency (warmer temperatures), the phase angle values started increasing to reach a plateau at intermediate reduced frequencies. At this peak, the polymer properties overcame the properties of the base asphalt binder content; the phase angle values were observed to increase again at low reduced frequencies. As seen in Figure 4b, the HRMA binder exhibited similar or slightly lower δ values when compared with its reference E binders regardless of the testing frequency or loading rate.

A black space diagram is constructed by plotting G* vs. δ values, as shown in Figure 5. However, for rheological significance, the black space diagram of an evaluated binder is represented by a point indicating G* vs. δ at a particular temperature and frequency. In this study, the specific temperature and frequency were selected to be similar to those of the traditional G-R parameter, 15°C and 0.005 rad/s, respectively. Data corresponding to the four aging conditions are shown in Figure 5. The dashed orange and dashed green dotted lines show the current PG boundaries for G* and δ for the RTFO and 20-hour PAV aging conditions, respectively. It can be seen that all binders fell well within these criteria, although with aging, the binder G* increased and the δ decreased. The diagram also shows a damage zone where cracking likely begins because of brittle rheological behavior defined by the G-R parameter between 180 kPa, the onset of cracking (dashed black line), and 600 kPa, significant cracking (solid black line), that correlates to low ductility values of 5 cm to 3 cm, respectively. It is anticipated that a lower G* and a lower δ represent lower susceptibility to cracking.

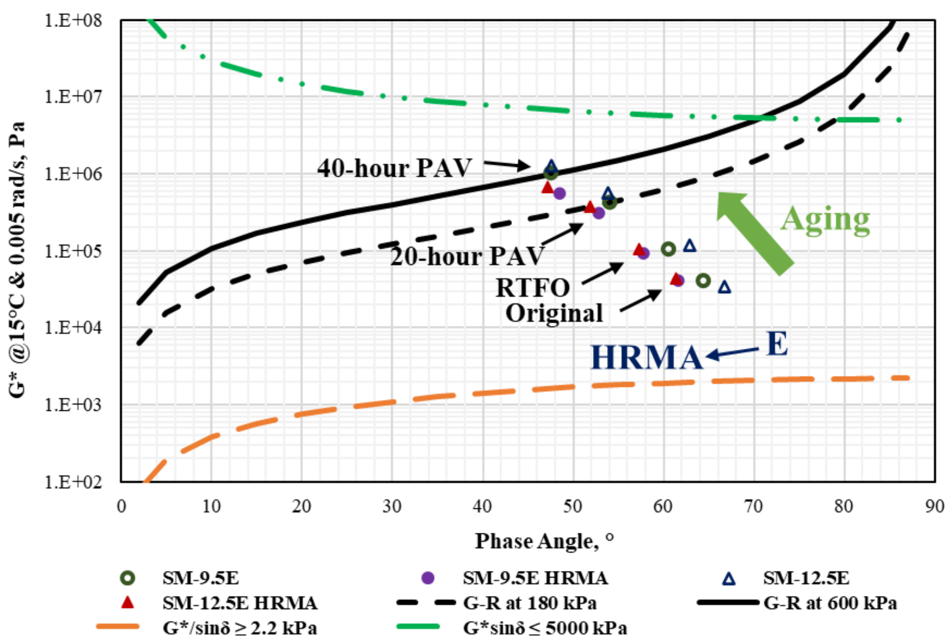


Figure 5. Black Space Diagram in Terms of Dynamic Shear Modulus (G*) at 15°C and 0.005 rad/s for All Evaluated Binders at Original, RTFO, 20-Hour PAV, and 40-Hour PAV Aging Conditions. SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; δ = phase angle; RTFO = rolling thin film oven; PAV = pressure aging vessel.

Overall, HRMA binders exhibited lower G* and lower δ values than their reference E binders, regardless of the aging conditions. In addition, a steeper slope between G* and δ represents lower susceptibility to long-term aging and resistance to the loss of flexibility. Overall, HRMA exhibited steeper slopes between G* and δ than its reference E binders. The HRMA binders were also noted to have a greatly reduced range of data compared to the reference E binders, which is an indication of the overall resistance to aging, again noting the same aging protocol for all binders. Finally, the data showed that the traditional E binders were the first to reach the G-R criterion of 180 kPa after 20-hour PAV aging and the G-R criterion of 600 kPa after 40-hour PAV aging.

Assessment of Cracking Performance for Evaluated Asphalt Binders

The fatigue life of a given asphalt binder can be predicted using the LAS test results coupled with the viscoelastic continuum damage model. Figure 6 shows the predicted fatigue life (N_f) for the four E and HRMA evaluated asphalt binders at 5% and 10% induced strain. These relatively higher amplitude strains are more likely to provide more reasonable ranking results for long-term aged binders (i.e., 20-hour PAV and 40-hour PAV) (Mannan et al., 2015). Regardless of the binder type (E vs. HRMA), N_f decreased with the increase in induced strain, which indicates that the LAS seems to be sensitive to modified asphalt binders. Further, for a given aging condition, HRMA binders exhibited significantly higher N_f values when compared with their reference E binders, confirming the superior cracking performance of asphalt binders and mixtures expected with HRMA modification.

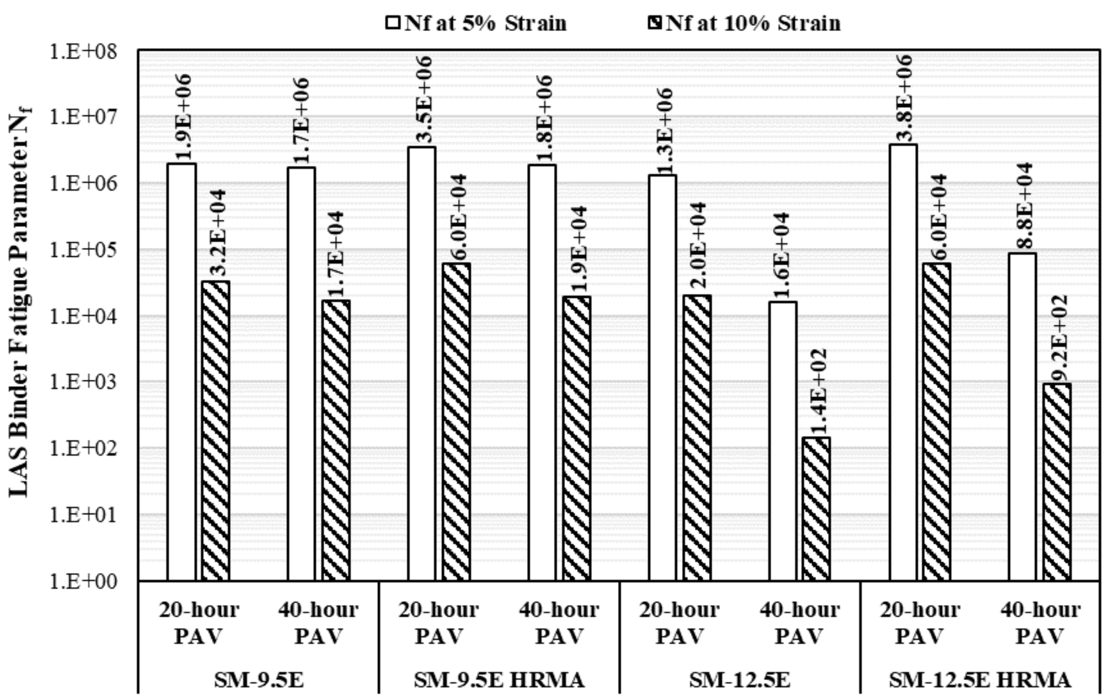


Figure 6. LAS Binder Fatigue Parameter for E and HRMA Evaluated Asphalt Binders at the 20-Hour and 40-Hour PAV Aging Conditions at 5% and 10% Induced Strain. LAS = linear amplitude sweep; PAV = pressure aging vessel; SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt.

Laboratory Evaluation of Asphalt Mixtures

The properties of the overlying asphalt layer are important to the performance of the overall system. Throughout the evaluation of regular E and HRMA plant-produced asphalt mixtures, test specimens were compacted to an air void level of 7.0 ± 0.5%.

Volumetric Properties and Aggregate Gradations of Mixtures

Tables 6 and 7 summarize the aggregate gradation and volumetric properties, respectively, determined for the four mixtures. The results compared well with the job-mix formula and the quality control and acceptance data available from the producers and VDOT districts. These data are provided in Appendix B.

Table 6. Aggregate Gradations for Evaluated E and HRMA Mixtures

Mixture ID	**SM-9.5E**	**SM-9.5E HRMA**	**SM-12.5E**	**SM-12.5E HRMA**
Sieve Size	**Percent Passing**			
¾ inch (19.0 mm)	100.0	100.0	100.0	100.0
½ inch (12.5 mm)	100.0	99.9	97.7	96.1
3/8 inch (9.5 mm)	96.7	97.1	85.7	83.3
No. 4 (4.75 mm)	56.6	58.2	51.6	52.6
No. 8 (2.36 mm)	37.3	37.4	32.8	34.6
No. 16 (1.18 mm)	26.3	25.3	22.4	23.5
No. 30 (600 µm)	19.7	18.5	16.1	16.1
No. 50 (300 µm)	14.7	13.7	11.8	10.6
No. 100 (150 µm)	10.2	9.5	8.3	6.6
No. 200 (75 µm)	6.1	6.0	5.6	3.9

SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt.

Table 7. Volumetric Properties for Evaluated E and HRMA Mixtures

Mixture ID	**SM-9.5E**	**SM-9.5E HRMA**	**SM-12.5E**	**SM-12.5E HRMA**
Composition				
RAP Content, %	15	15	15	15
Asphalt Binder	PG 64E-22	PG 64E-22 HRMA	PG 64E-22	PG 64E-22 HRMA
Volumetric Property				
N_{design}, gyrations	50	50	50	50
NMAS, mm	9.5	9.5	12.5	12.5
Asphalt Binder Content, %	5.6	5.8	5.5	5.4
Rice SG (G_{mm})	2.560	2.573	2.707	2.712
Aggregate Bulk SG (G_{sb})	2.800	2.827	2.961	2.981
VTM, %	3.1	3.6	4.5	5.6
VMA, %	16.3	17.3	17.5	18.7
VFA, %	81.2	79.2	74.1	70.0
FA Ratio	1.11	1.04	1.08	0.74

RAP = reclaimed asphalt pavement; PG = performance grade; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; N_{design} = number of Superpave design gyrations; NMAS = nominal maximum aggregate size; SG = specific gravity; VTM = voids in total mixture; VMA = voids in mineral aggregate; VFA = voids filled with asphalt; FA = fines to asphalt ratio.

The major change noted among mixtures was a slight drop in the percent passing the No. 200 sieve for SM-12.5E HRMA when compared with SM-12.5E. A lower percent passing was observed for sieves No. 4 and No. 8 for the SM-12.5E plant-produced mixtures compared to the corresponding job-mix formula, irrespective of the testing party (Producer, VDOT, or VTRC). Similarly, a lower percent passing the No. 200 sieve was observed for the SM-12.5E HRMA plant-produced mixtures compared to the corresponding job-mix formula, regardless of the testing party (Producer, VDOT, or VTRC). A slight increase in voids leading to higher VMA and lower VFA values was observed for HRMA mixtures when compared with the E reference mixtures, which could be attributable to the partial inclusion and effect of GTR on the mixture.

Durability Assessment of Mixtures

Figure 7 shows the Cantabro mass loss of the Non-Reheats-VTRC and Reheats-VTRC specimens for the four evaluated mixtures. Producer B compacted and evaluated Non-Reheats-P, Reheats-P, and Reheats-LTOA-P specimens in terms of Cantabro mass loss for each of the two sets for SM-12.5E and SM-12.5E HRMA. Overall, Reheats-VTRC and Reheats-P specimens exhibited a similar or relatively higher Cantabro mass loss when compared with their corresponding Non-Reheats-VTRC and Non-Reheats-P, respectively, which could be attributable to reheating and its stiffening effect on the evaluated asphalt mixtures. It should be mentioned that the impact of reheating was much more limited for the 9.5 mm mixtures when compared with the 12.5 mm mixtures. Further, Reheats-P specimens exhibited a statistically similar Cantabro mass loss when compared with Non-Reheats-VTRC specimens. Finally, not much change was observed for Reheated-LTOA-P specimens when compared with other types of specimens, which could confirm a long-term durability of VDOT SBS- and HRMA-modified mixtures.

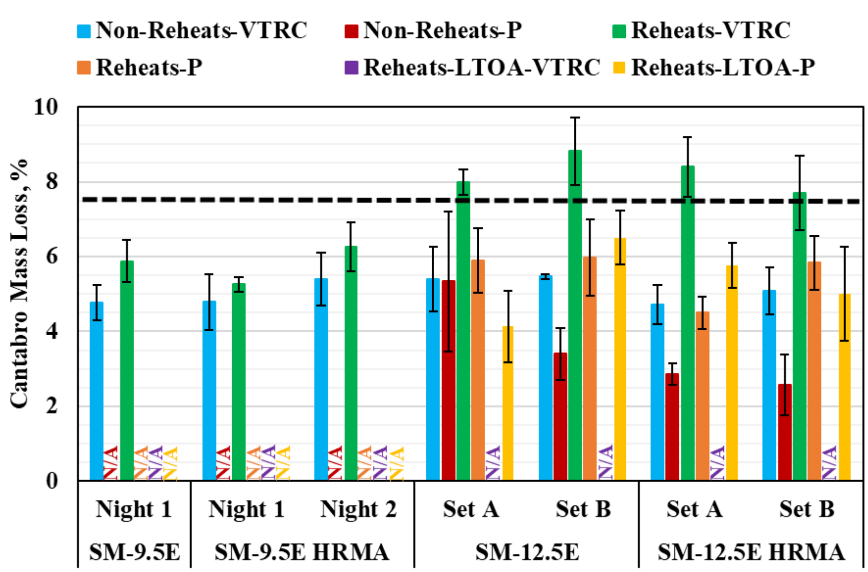


Figure 7. Performance Test Data for Cantabro Mass Loss of E and HRMA Mixtures. I-bars indicate mass loss variability ± 1 standard deviation. SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; VTRC = Virginia Transportation Research Council; P = producer; LTOA = long-term oven aged. Dashed black line = VDOT's balanced mix design limit for unmodified asphalt mixtures.

The dashed black line in Figure 7 refers to VDOT's BMD criterion of 7.5% for Cantabro mass loss (Diefenderfer et al., 2021). All evaluated mixtures exhibited a mass loss lower than 7.5% regardless of the type of specimens being tested except for reheats-VTRC. The reason remains unknown. It should be mentioned that the 7.5% threshold is valid only for A and D mixtures (mixtures subjected to a traffic of 1.0 to 3.0 million ESALs and 3.0 to 10.0 million ESALs, respectively) and is provided for reference purposes only. Currently, VDOT does not specify a pass/fail criterion for the Cantabro mass loss of modified mixtures subjected to heavy traffic.

Mechanical Property of Mixtures

Figures 8a and 8b show the dynamic modulus |E*| master curves at a reference temperature of 21.1°C for the four evaluated mixtures at short- and long-term aging stages, respectively. The data in Figure 8 show that the E and HRMA mixtures exhibited similar |E*| values at intermediate and high frequencies, with HRMA mixtures exhibiting slightly softer behavior at relatively low frequencies. This observation is valid for both evaluated NMASs (i.e., 9.5 and 12.5 mm) and both aging stages (STOA vs. LTOA).

Figures 9a and 9b show the phase angle (δ) master curves at a reference temperature of 21.1°C for the four evaluated mixtures at short- and long-term aging stages, respectively. The data in Figure 9a show that SM-9.5E and SM-9.5E HRMA exhibited a similar phase angle (denoting a similar elastic behavior) at intermediate and high reduced frequencies, with HRMA showing lower phase angles at lower reduced frequencies. For the 12.5 mm mixtures, SM-12.5E HRMA exhibited significantly lower phase angle values than SM-12.5E, indicating a potential more pronounced elastic behavior. After long-term aging, 9.5 and 12.5 mm HRMA mixtures exhibited similar phase angle values when compared with their 9.5 and 12.5 mm control mixtures.

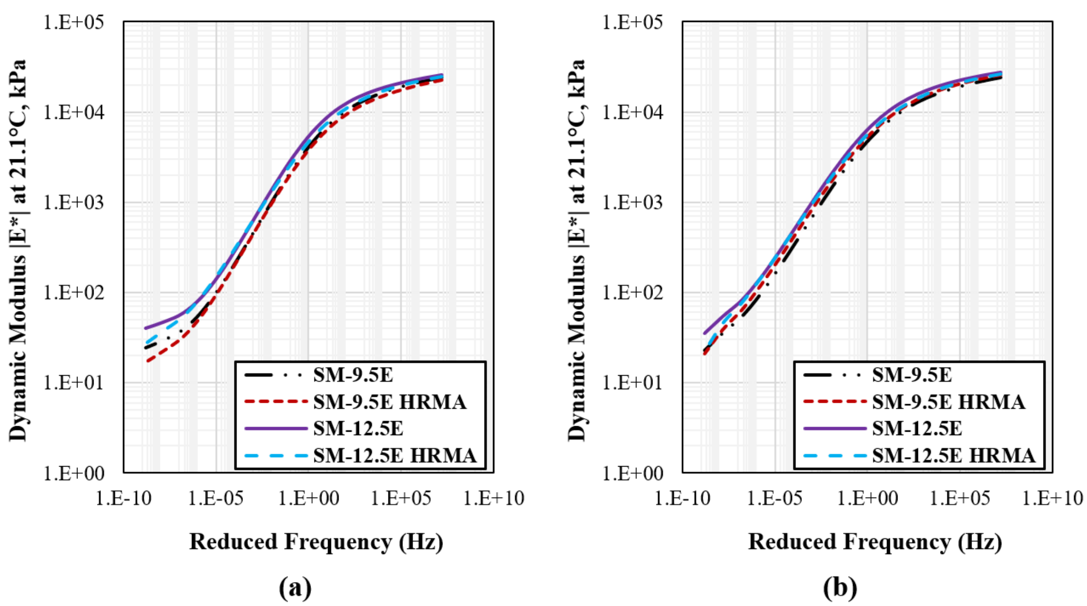


Figure 8. Dynamic Modulus |E*| Master Curves for E and HRMA Mixtures: (a) short-term oven aged; (b) long-term oven aged. SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt.

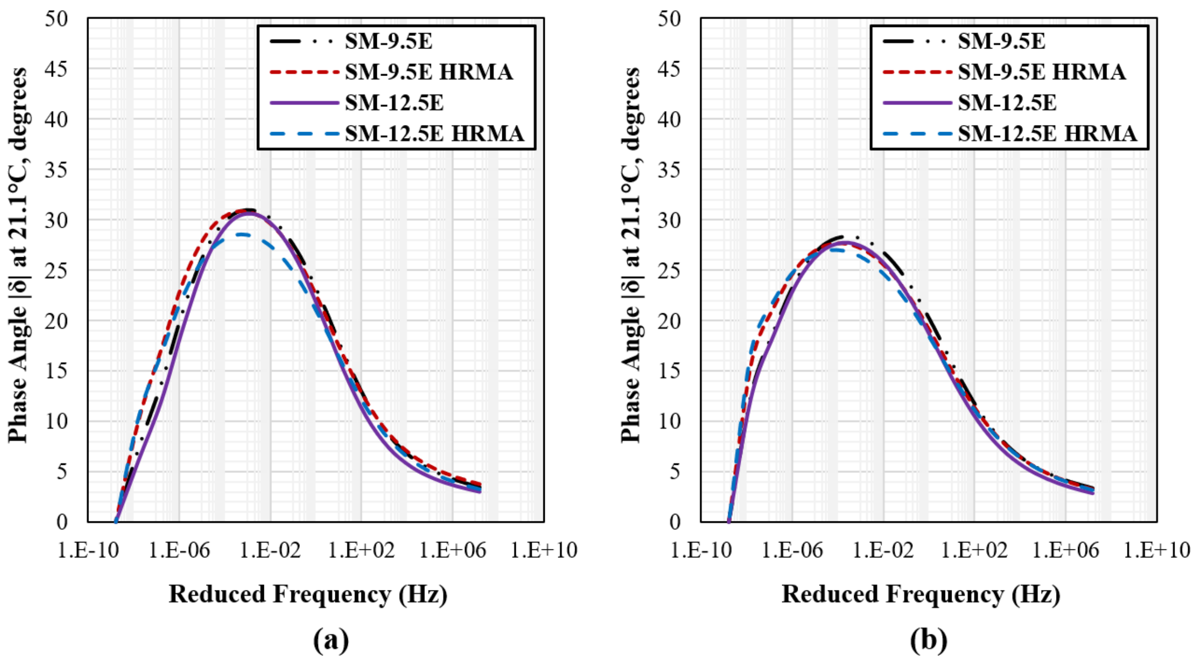


Figure 9. Phase Angle (δ) Master Curves for E and HRMA Mixtures: (a) short-term aged; (b) long-term oven aged. SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt.

Assessment of Rutting Performance for Evaluated Mixtures

Five performance tests were considered to assess the resistance of E and HRMA mixtures to rutting. These tests belong to three levels of testing: basic, intermediate, and advanced. The basic level included the IDT-HT and IDEAL-RT, characterized by a short time for specimen preparation and testing without the requirement of any specific cutting, coring, and gluing. The intermediate level included the APA rut test, which needs longer times for specimen preparation and testing. The advanced level included the SSR and RLT tests, which require more specimen preparation and test/analysis time including time for cutting and/or coring to prepare specimens and multiple days to complete and analyze the test results.

IDT-HT and IDEAL-RT Results and Analyses

Figures 10a and 10b show the S_t and RT index values for the four evaluated mixtures. Only Reheats-VTRC specimens were considered for these two tests. The mean S_t values for the four mixtures ranged from 216.3 to 359.5 kPa, with a coefficient of variation ranging from 4.6% to 13.0%. The mean RT index values for the four mixtures ranged from 86.6 to 140.8 kPa, with a coefficient of variation ranging from 5.7% to 12.5%. Overall, HRMA mixtures had better repeatability in terms of the IDT-HT and IDEAL-RT results when compared to E mixtures. S_t and RT index values were in full agreement. Moreover, SM-9.5E HRMA had a mean S_t and RT index lower than its control SM-9.5E. An opposite trend was observed for SM-12.5E and SM-12.5E HRMA. The dashed purple line in Figures 10a and 10b indicates VDOT's BMD criteria of 133 kPa and 72 for the IDT-HT S_t and RT index (Boz et al., 2023). All evaluated mixtures had test results greater than the recommended thresholds.

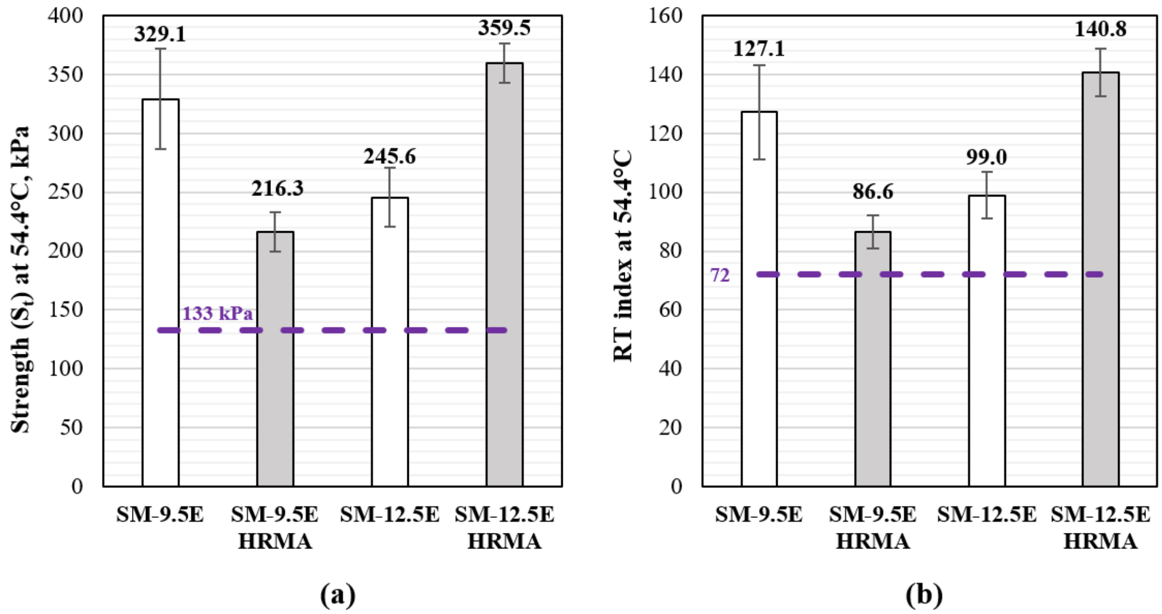


Figure 10. Performance Test Data of E and HRMA Mixtures: (a) IDT-HT; (b) IDEAL-RT. I-bars indicate parameter variability ± 1 standard deviation. SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; IDT-HT = indirect tensile test at high temperature; IDEAL-RT = indirect tensile rutting test; RT = rutting tolerance. Dashed purple line = VDOT's balanced mix design limit for unmodified asphalt mixtures.

It should be mentioned that these thresholds are valid only for A and D mixtures and are provided here for reference purposes only. Currently, VDOT does not specify a pass/fail criterion for the IDT-HT S_t and RT index of modified mixtures subjected to heavy traffic.

APA Rut Test Results and Analyses

Only two types of specimens were considered for the APA rut test: Non-Reheats-VTRC and Reheats-VTRC. Figure 11 shows the APA rut depth for the four evaluated mixtures at 64°C and after being subjected to 8,000 loading cycles. Overall, low APA rut depth values (<4.0 mm) were observed for the four evaluated mixtures, with reheating having a minimal impact on the resistance to rutting. Slightly higher average APA rut depth values were observed for SM-9.5E HRMA Night 1 and Night 2 and SM-12.5E HRMA Set A mixtures when compared to their corresponding control mixtures, which could be attributable to the variability of the test itself with no solid performance-based justification. The dashed black line in Figure 11 indicates VDOT's BMD criterion of 8.0 mm for the APA rut depth at 64°C and after 8,000 loading cycles (Diefenderfer et al., 2021). It should be mentioned that this threshold is valid only for A and D mixtures and is provided here for reference purposes only. Currently, VDOT does not specify an APA rut test pass/fail criterion for modified mixtures subjected to heavy traffic when the test is performed in accordance with AASHTO T 340. All evaluated mixtures had test results lower than the recommended threshold. Overall, the APA rut test results indicated that the evaluated mixtures are not expected to exhibit excessive rutting in the field.

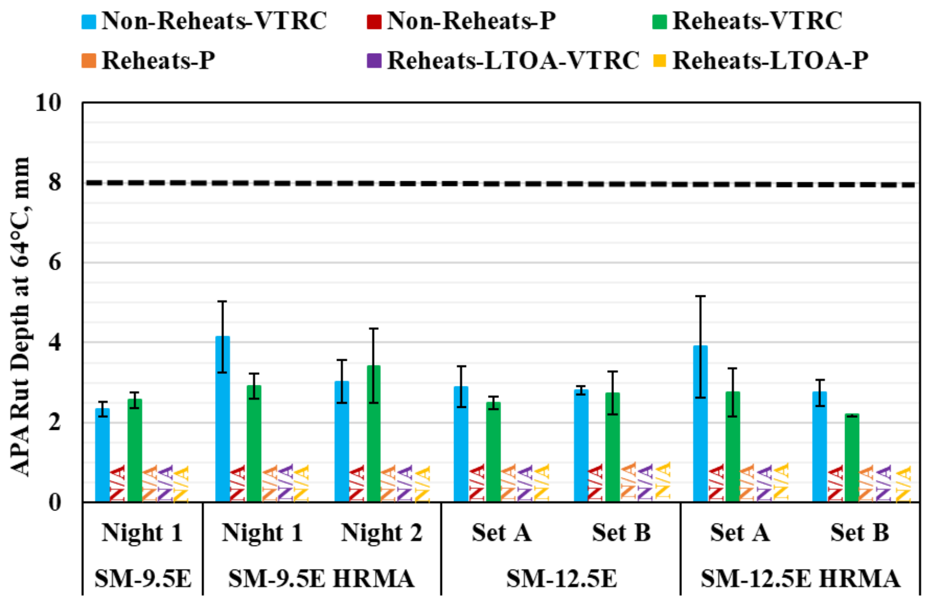


Figure 11. Performance Test Data for APA Rut Depth of E and HRMA Mixtures. I-bars indicate rut depth variability ± 1 standard deviation. SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; VTRC = Virginia Transportation Research Council; P = producer; LTOA = long-term oven aged; APA = Asphalt Pavement Analyzer. Dashed black line = VDOT's balanced mix design limit for unmodified asphalt mixtures.

SSR Test Results and Analyses

Figure 12a shows the RSI values for the four mixtures. Overall, 12.5 mm mixtures exhibited a lower RSI than 9.5 mm mixtures, typically expected for mixtures with a greater NMAS. Regardless of the NMAS, HRMA mixtures exhibited slightly lower RSI values than E mixtures, thus indicating a promising/better resistance to rutting. All evaluated mixtures exhibited RSI values ranging from 2% to 4%, the recommended threshold values for the heavy traffic category (10 to 30 million ESALs) (FHWA, 2021b).

RLT Test Results and Analyses

Figure 12b shows the rutting relationship at 50°C for all evaluated E and HRMA mixtures. The rutting relationships at the three testing temperatures for all evaluated mixtures are shown in Appendix C. Overall, a greater rutting sensitivity to temperature was observed for SM-9.5E and SM-12.5E HRMA than for SM-9.5E HRMA and SM-12.5E. A lower rutting characteristic indicates a lower accumulated permanent strain with loading, thus indicating a better resistance to rutting. Further, a flatter curve indicates a lower susceptibility of the asphalt mixtures to rutting by repeated loading. Overall, the HRMA mixtures (SM-9.5E HRMA and SM-12.5E HRMA) exhibited similar rutting relationships (by means of the intercept with the vertical y-axis), higher than SM-9.5E and lower than SM-12.5E. More important, the HRMA mixtures exhibited flatter rutting relationships than the E mixtures, indicating a lower susceptibility to rutting by repeated loading and thus a generally better rutting resistance.

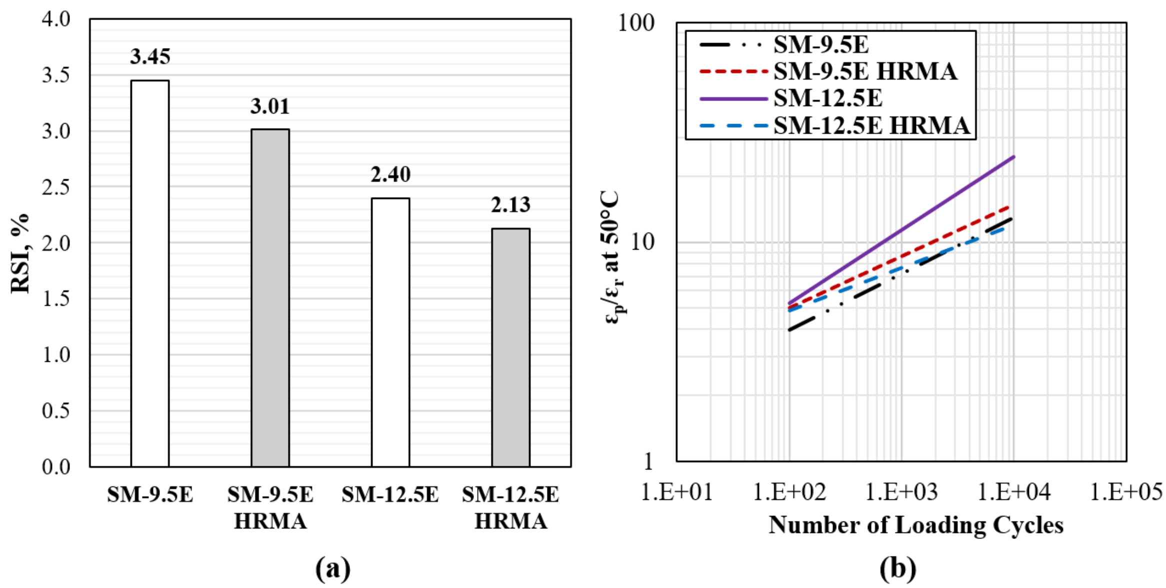


Figure 12. Performance Test Data of E and HRMA Mixtures: (a) SSR test; (b) RLT test at 50°C. SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; RSI = rutting strain index; SSR = stress sweep rutting; RLT = repeated load triaxial test.

Assessment of Cracking Performance for Evaluated Mixtures

Three performance tests were considered to assess the resistance of E and HRMA mixtures to cracking. These tests belong to three levels of testing complexity: basic, intermediate, and advanced. The differences among these levels were highlighted in the previous section. The basic, intermediate, and advanced levels included the IDT-CT, I-FIT, and direct tension cyclic fatigue test, respectively.

IDT-CT Results and Analyses

The IDT-CT was conducted in the laboratory on six types of specimens fabricated from the mixtures at the plant: Non-Reheats, Reheats-VTRC, and Reheats-LTOA-VTRC for all evaluated mixtures and Non-Reheats-P, Reheats-P, and Reheats-LTOA-P provided by Producer B for SM12.5E and SM-12.5E HRMA. Figure 13 shows the CT index at 25°C for all types of specimens for all evaluated mixtures. Overall, non-reheats exhibited similar or higher CT index values regardless of mixture type, thus indicating a negative impact of reheating on the cracking performance of asphalt mixtures. HRMA mixtures exhibited relatively similar or greater CT index values than E mixtures, thus indicating a better resistance to cracking with the use of HRMA modification. The dashed black line in Figure 13 indicates VDOT's BMD criterion of 70 for the CT index at 25°C (Diefenderfer et al., 2021). It should be mentioned that this threshold is valid only for A and D mixtures and is provided for reference purposes only. Currently, VDOT does not specify a pass/fail criterion for the CT index of modified mixtures subjected to heavy traffic. All evaluated mixtures had test results similar to or greater than the recommended threshold. Overall, the IDT-CT results indicated that the evaluated mixtures are expected to resist cracking in the field. Moreover, similar CT index values were observed for the four mixtures after long-term oven aging when compared with only reheating (short-term aging) regardless of mixture type (i.e., E vs. HRMA).

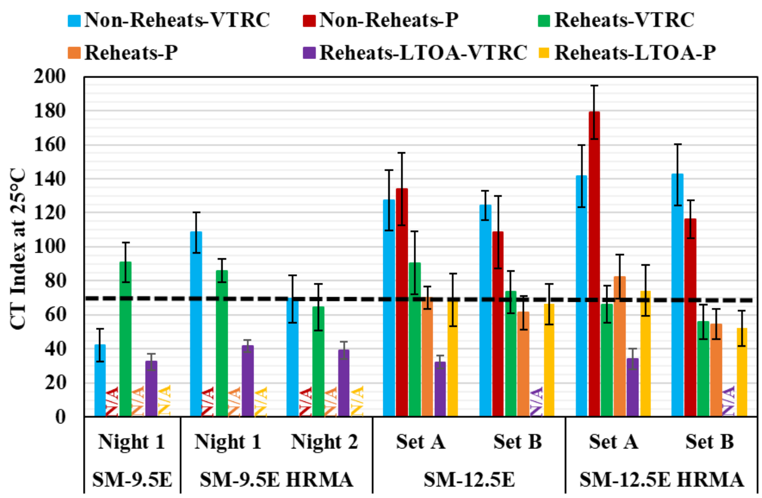


Figure 13. Performance Test Data for IDT-CT of E and HRMA Mixtures. I-bars indicate CT index variability ± 1 standard deviation. SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; VTRC = Virginia Transportation Research Council; P = producer; LTOA = long-term oven aged; IDT-CT = indirect tensile cracking test; CT = cracking tolerance. Dashed black line = VDOT's balanced mix design criterion of 70 for the CT index at 25°C.

I-FIT Results and Analyses

Figure 14 shows the FI values for all evaluated mixtures. Although the values are relatively lower when compared to typical FI values for A and D SMs in Virginia, it can be observed that the HRMA mixtures exhibited similar or greater FI values than E mixtures, indicating a similar resistance to cracking in the field.

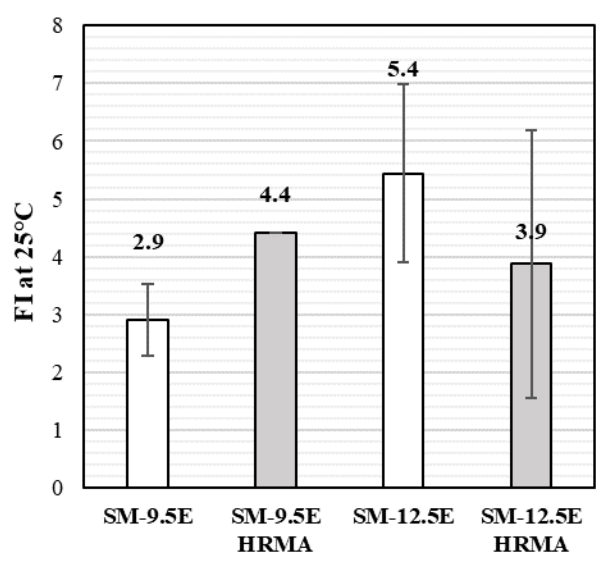
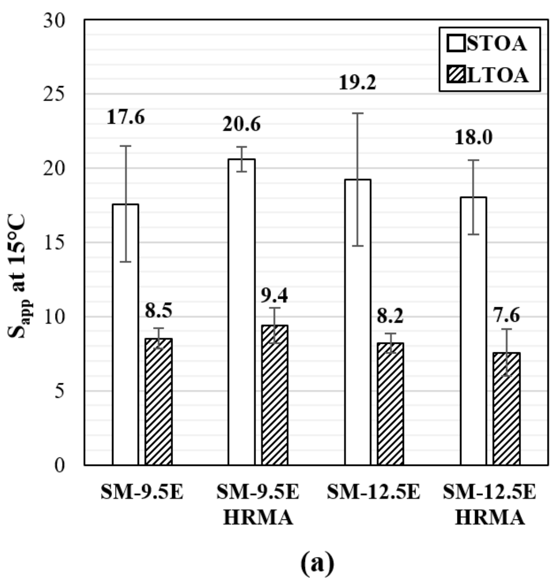


Figure 14. Performance Test Data for I-FIT of E and HRMA. I-bars indicate FI variability ± 1 standard deviation. I-FIT = Illinois flexibility index test; FI = flexibility index; SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt.

Direct Tension Cyclic Fatigue Test Results and Analyses

Figure 15a shows the S_{app} values for all E and HRMA mixtures at both aging stages (STOA and LTOA). All mixtures exhibited similar S_{app} values regardless of the type of modification and aging level, thus indicating similar resistance to cracking. A similar drop in S_{app} values was observed for all mixtures between the STOA and LTOA levels. When the fatigue life of mixtures is considered, stiffness often is not the only factor to be considered. The toughness of material, which refers to its ability to absorb energy without fracturing, is often necessary to be used as another aspect of analysis. The D_R parameter can be used as an indicator of toughness. Figure 15b shows the D_R parameter for all evaluated mixtures after short- and long-term oven aging. Similar or greater D_R values were observed for HRMA mixtures than for reference E mixtures. Figures 16a and 16b show the damage characteristic curves for all mixtures after short- and long-term oven aging, respectively. The damage characteristic curve is strain independent for a given asphalt mixture, and its position captures the mixture's stiffness.

For STOA, HRMA mixtures showed higher damage curves than E mixtures, indicating a relatively stiffer behavior. For STOA, 12.5 mm mixtures exhibited higher damage curves than 9.5 mm mixtures; this was in line with the observations derived from the dynamic modulus |E*| test results. Much lower and steeper curves were observed for LTOA mixtures than for STOA mixtures, which shows the impact of aging. For LTOA, the same observations are valid for the 12.5 mm mixtures; however, SM-9.5E exhibited a lower damage curve than SM-9.5E HRMA. All of these observations indicate that HRMA modification could be as beneficial as regular SBS modification and could provide similar and better performance properties and characteristics for the resultant mixtures.



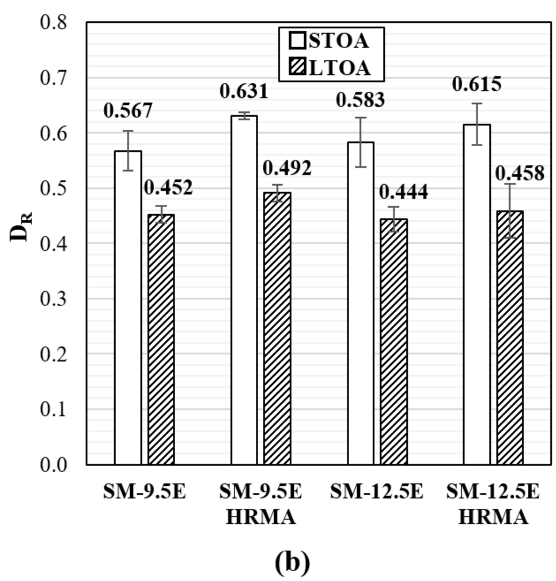


Figure 15. Cyclic Fatigue Performance Test Data of E and HRMA Mixtures: (a) S_{app} at 15°C; (b) D_R. I-bars indicate parameter variability ± 1 standard deviation. SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; STOA = short-term oven aged; LTOA = long-term oven aged.

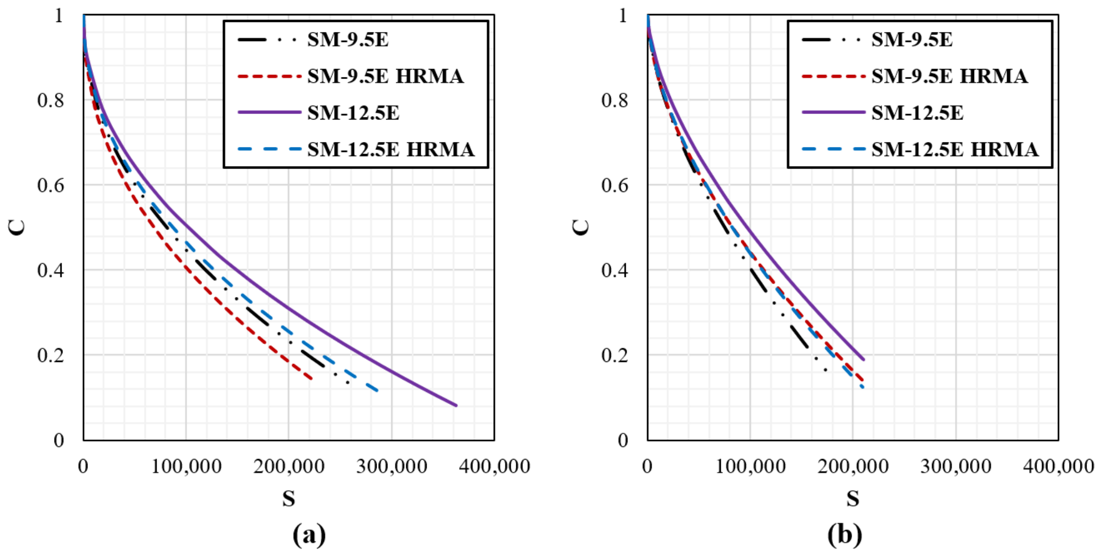


Figure 16. Cyclic Fatigue Performance Test Data of E and HRMA Mixtures in Terms of C vs. S Curves: (a) STOA; (b) LTOA. C = material integrity; S = damage; SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; STOA = short-term oven aged; LTOA = long-term oven aged.

Summary of Performance of Reheated Mixtures

Table 8 presents a summary overview of performance trends for reheated mixtures. It summarizes various properties of interest, including durability, mechanical properties, resistance to rutting, and resistance to cracking, for SM-9.5E HRMA and SM-12.5E HRMA compared to their corresponding reference mixtures, SM-9.5E and SM-12.5E. Table 8 showcases the changes in these properties compared to the control, denoted by arrows indicating improvements (↑), declines (↓), or no change (↔) in each category. Overall, the evaluated mixtures demonstrated similar or improved performance properties and characteristics with HRMA modification.

Table 8. Summary of Performance Trends for Reheated Mixtures

Property of Interest	Mixture ID			
	SM-9.5E	SM-9.5E HRMA	SM-12.5E	SM-12.5E HRMA
Durability (by means of Cantabro test)	C	↔	C	↔
Mechanical property (advanced):				
- Dynamic modulus |E*|	C	↔	C	↔
- Phase angle (δ)		↔		↑
Resistance to rutting (by means of):				
- IDT-HT (basic)	C	↓	C	↑
- IDEAL-RT (basic)		↓		↑
- APA rut test (intermediate)		↔		↔
- SSR test (advanced)		↑		↑
- RLT test (advanced)		↓		↑
Resistance to cracking (by means of):				
- IDT-CT (basic)	C	↔	C	↔
- I-FIT (intermediate)		↑		↔
- Direct tension fatigue test (advanced)		↔		↔

SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; C = control; IDT-HT = indirect tensile test at high temperature; RT = rutting tolerance; APA = Asphalt Pavement Analyzer; SSR = stress sweep rutting; RLT = repeated load triaxial; IDT-CT = indirect tensile cracking test; I-FIT = Illinois flexibility index test; ↑ = property of interest improved compared to control; ↓ = property of interest declined compared to control; ↔ = no change in property of interest compared to control.

Evaluation of Field Cores

Field core samples were collected from each project during construction. Core locations were randomly stratified along the length and width of the section. Table 9 summarizes the in-place layer thicknesses, air void levels, and permeability values. Two major changes were noted among the evaluated E and HRMA mixtures. The in-place density of E mixtures was higher than for HRMA mixtures regardless of the NMAS (i.e., 9.5 mm and 12.5 mm). The measured permeability values of E cores were lower than for HRMA cores regardless of the NMAS. However, the average of the permeability results for all evaluated mixtures was lower than the VDOT threshold value of $150*10^{-5}$ cm/s at 7.5% air voids (VDOT, 2000).

An initial effort was made to determine the Cantabro mass loss for field cores. However, field cores were too thin for testing, so no Cantabro data are available. The 150-mm-diameter cores had a thickness lower than the typical thickness of 62 mm. The research team acknowledges the variation that might be induced with high variations from the target heights; therefore, the data generated were used for comparison purposes, especially with plant-produced laboratory-compacted specimens, to assess the impact of specimen preparation type (laboratory vs. field compaction) and other components such as in-place densities.

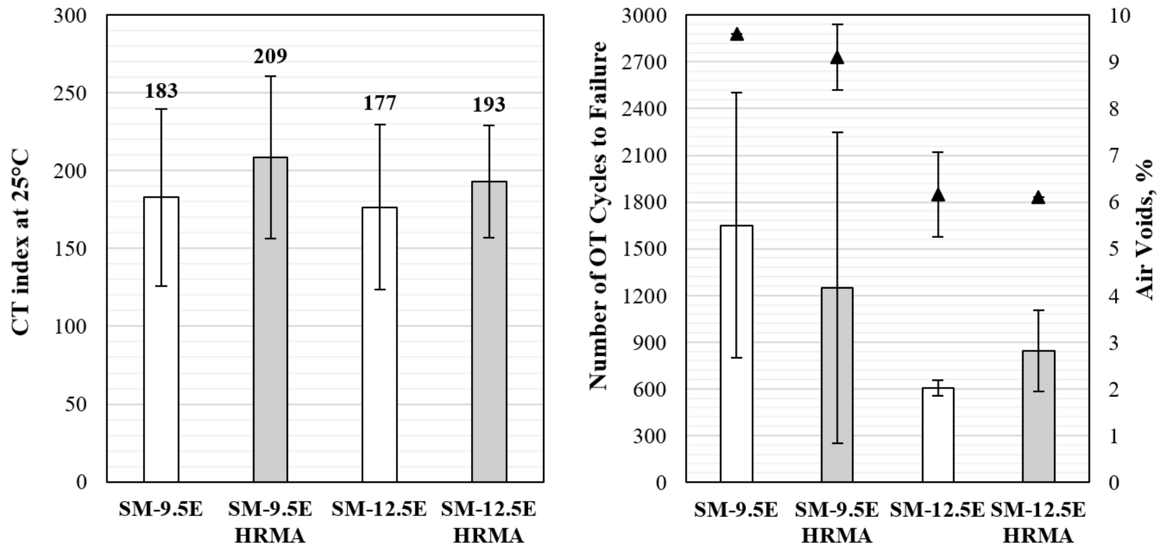
Figure 17a shows the CT index values of all cores by mixture type. All mixtures had similar values. With regard to the mean/average values, HRMA mixtures had slightly higher CT index values than their corresponding reference E mixtures. Figure 17b shows the number of cycles to failure at 25°C for the cores of all evaluated mixtures. A higher number of OT cycles to failure indicates a better resistance to reflective cracking. A similar number of OT cycles to failure was observed for the HRMA mixtures and their corresponding E reference mixtures. It should be noted that the OT is well known for its high variability.

The OT data were further analyzed to quantify the resistance of the evaluated mixtures to cracking initiation (using G_c) and cracking propagation (using the CPR) (Garcia et al., 2016). A greater G_c value indicates that the evaluated mixture is tough and requires high initial energy to initiate a crack. On the other hand, a greater CPR value indicates that the evaluated mixture is more susceptible to cracking (a fast crack propagation indicates a shorter reflective cracking life).

Table 9. Summary of In-Place Layer Thickness, Air Void Level, and Permeability for Core Samples

Mixture Type / Property Measured	Layer Thickness (mm)			In-Place Air Voids (%)			Permeability ($*10^{-5}$ cm/s)		
	Avg.	CI	Target	Avg.	CI	Range	Avg.	CI	Target
SM-9.5E	34.3	1.6	38.1	7.1	1.3	5.8 to 8.5	76	71	150
SM-9.5E HRMA	39.7	3.2	38.1	7.5	0.9	6.6 to 8.4	147	87	
SM-12.5E	52.9	3.6	50.8	6.0	0.8	5.2 to 6.8	25	34	
SM-12.5E HRMA	44.5	3.8	50.8	7.7	1.3	6.9 to 8.4	115	78	

Avg. = average; CI = 95% confidence interval; SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt.



(a) **(b)**

Figure 17. Cracking Performance Test Data of E and HRMA Field Cores: (a) IDT-CT; (b) Texas OT. I-bars indicate parameter variability ± 1 standard deviation. SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; IDT-CT = indirect tensile cracking test; OT = overlay test; CT = cracking tolerance.

Figure 18 shows a design interaction graph plotting G_c vs. CPR for the E and HRMA evaluated field cores. Four categories were identified on this interaction plot:

1. *Tough-Crack Resistant:* characterized with relatively higher G_c values and relatively lower CPR values simulating a good resistance in both crack initiation and crack propagation.

2. *Tough-Crack Susceptible:* characterized with relatively higher G_c values and relatively higher CPR values simulating a good resistance in crack initiation but susceptible to crack propagation.

3. *Soft-Crack Resistant*: characterized with relatively lower G_c values and relatively lower CPR values simulating softness and susceptibility to crack initiation but a slowdown of the propagation of the crack.

4. *Soft-Crack Susceptible:* characterized with relatively lower G_c values and relatively higher CPR values simulating a significant poor resistance to both crack initiation and crack propagation.

A preliminary threshold for a CPR of 0.5 was proposed by Garcia et al. (2016). Moreover, preliminary limits for G_c were identified: an upper limit of 3 to screen the asphalt mixtures with high brittleness potential and a lower limit of 1 to guarantee a minimum stability under traffic of the evaluated mixtures. It should be noted that these thresholds were used for comparison purposes only. Independent efforts should consider defining new thresholds specifically for E and HRMA mixtures.

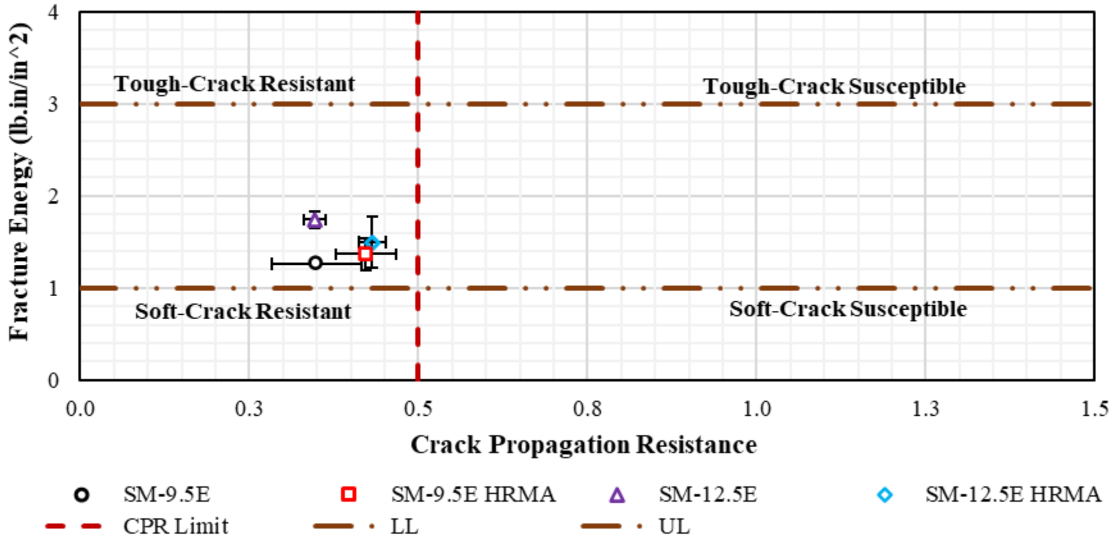


Figure 18. Cracking Performance Test Data for OT of E and HRMA Field Cores at 25°C in Terms of Interaction Plot. I-bars indicate parameter variability ± 1 standard deviation. CPR = crack propagation resistance; LL = lower limit; UL = upper limit; SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt.

As seen in Figure 18, all E and HRMA mixtures / field cores had a CPR value lower than 0.5, indicating good cracking resistance. Moreover, all mixtures had a G_c from 1 to 3, indicating good resistance to crack initiation. SM-9.5E showed the most soft-crack-resistant behavior and SM-12.5E showed the most soft-crack-susceptible behavior among the four evaluated mixtures.

Mechanistic-Empirical Simulations

Figures 19a and 19b show the fatigue performance in terms of top-down damage for Structure (i) and in terms of total damage for Structure (ii) over a 30-year span, respectively. It should be noted that the single change for each of the considered pavement structures was limited to the material selected in the surface layer (i.e., SM-9.5E vs. SM-9.5E HRMA vs. SM-12.5E vs. SM-12.5E HRMA).

For Structure (i), HRMA mixtures exhibited a similar fatigue performance when compared to that of the corresponding E reference mixtures regardless of the mixture's NMAS. For Structure (ii), SM-12.5E and SM-12.5E HRMA exhibited a similar fatigue performance; however, SM-9.5E showed a lower percent total damage when compared to SM-9.5E HRMA.

Figures 20a and 20b show the rutting performance in terms of total rut depth progression over the 30-year span life for the evaluated mixtures. All mixtures displayed a rapid increase in rutting depth within the first 2 years, followed by a gradual and consistent rate in rutting depth throughout the analysis duration.

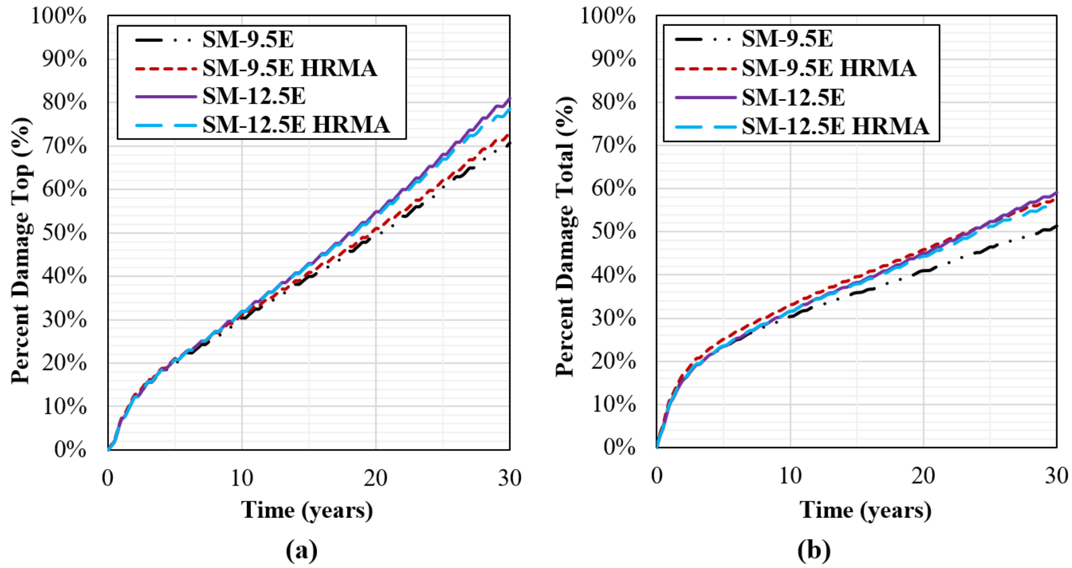


Figure 19. FlexPave Simulation Fatigue Performance Results: (a) Structure (i); (b) Structure (ii). SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt.

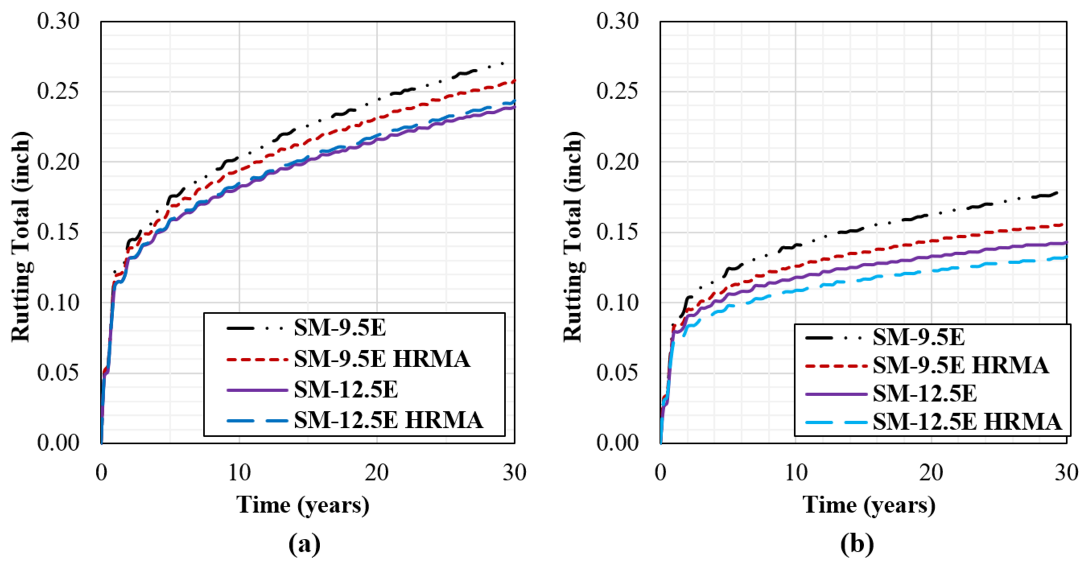


Figure 20. FlexPave Simulation Total Rutting Performance Results: (a) Structure (i); (b) Structure (ii). SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt.

For Structure (i), the use of HRMA mixtures resulted in a rutting performance similar to or lower than that for the corresponding E reference mixtures. For Structure (i), both SM-9.5E HRMA and SM-12.5E HRMA had a lower total rutting depth than SM-9.5E and SM-12.5E. The results of ME simulations by means of Pavement ME Design are shown in Appendix C. Overall, similar and/or better cracking and rutting predicted performance was observed for HRMA mixtures than for E reference mixtures.

Non-Destructive Pavement Testing

Ride Quality, Rut Depth, and Mean Profile Depth Results and Analyses

The ride testing, rut depth, and MPD results for Route 120 / Glebe Road are shown in Figures 21 through 23, respectively. From Figure 21 it can be seen that the average IRI value was above 100 in/mi and the results were comparable for the E (reference) and HRMA sections. In general, IRI values more than 100 in/mi indicate rough pavement. Lower rut depth values (average value of 0.06 inch) were observed for both the E and HRMA sections (Figure 22). MPD values relate to macrotexture; the results are shown in Figure 23. In general, average MPD values of 0.7 mm were observed, which are comparable to typical values for asphalt pavement surfaces in Virginia. Similar findings were observed for Route 625 / Waxpool Road; IRI rut depth and macrotexture results were comparable for the E and HRMA sections. The results are provided in Appendix D.

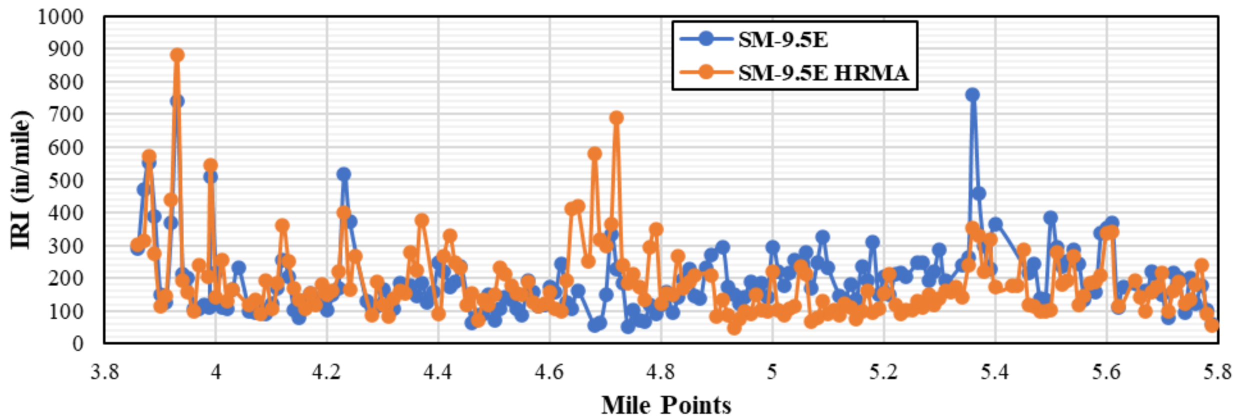


Figure 21. International Roughness Index (IRI) Results for Route 120 / Glebe Road

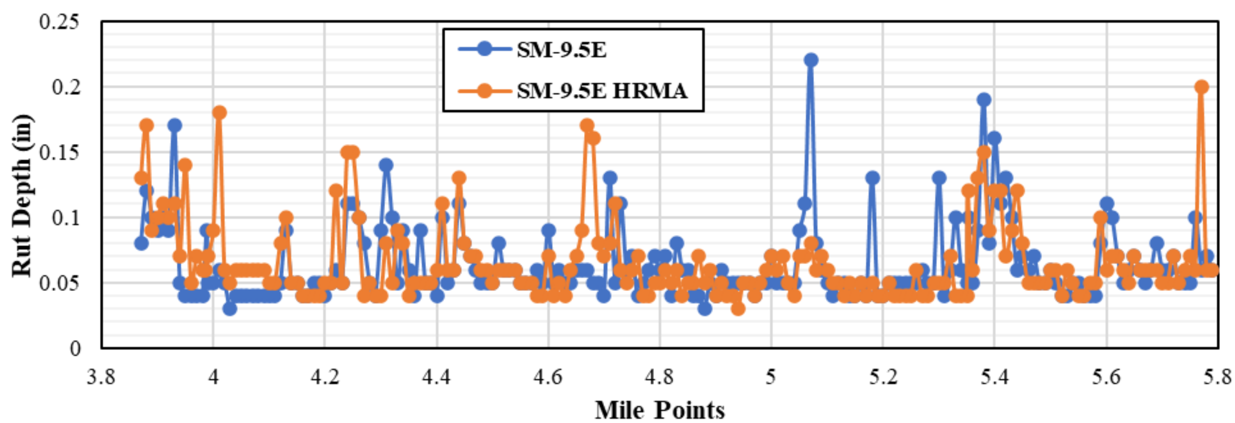


Figure 22. Rut Depth Results for Route 120 / Glebe Road

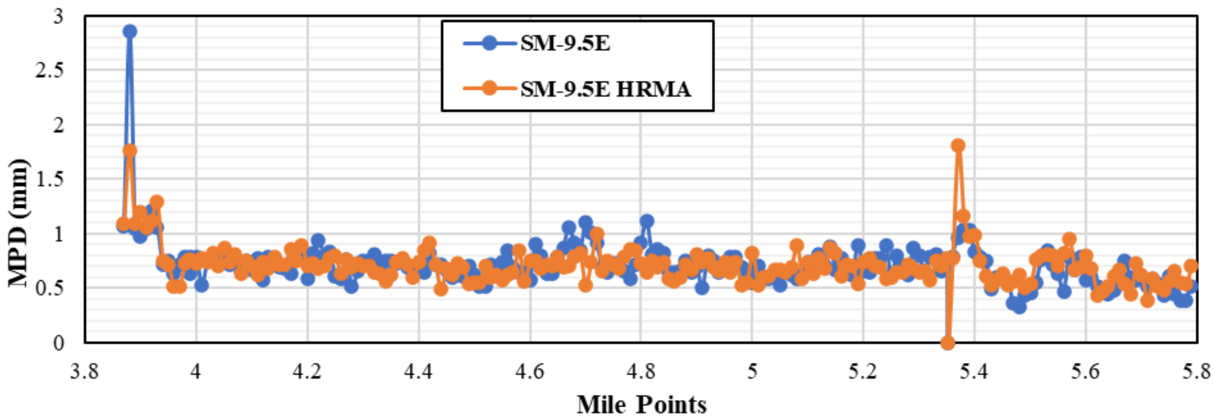


Figure 23. Mean Profile Depth (MPD) Results for Route 120 / Glebe Road

FWD Test Results and Analyses

FWD deflection data for the first sensor (D1) and the last sensor (D9) for 9,000-lb load levels are shown in Figures 24 and 25 for Route 120 / Glebe Road. D1 denotes the deflection at the loading plate, and D9 denotes the deflection at a distance of 72 inches from the loading plate. The D1 parameter is an indicator of the overall structural capacity of the pavement system, and the D9 parameter is an indicator of the quality of the pavement foundation (subgrade). The results in Figure 24 show the deflection of D1 to be uniform (with an average of approximately 5 mils using a 9,000-lb load level) with the exception of deflection at station 1,000 ft, which showed a deflection value of 17 mils. Earlier studies showed that these deflection values indicate a strong structural pavement (Diefenderfer et al., 2019; Pierce et al., 2017). Route 103 / Glebe Road was an existing composite section, and lower deflection values were expected. Similar lower deflection values (<9 mils) were observed for Route 625 / Waxpool Road (asphalt pavement section), as shown in Figure 25.

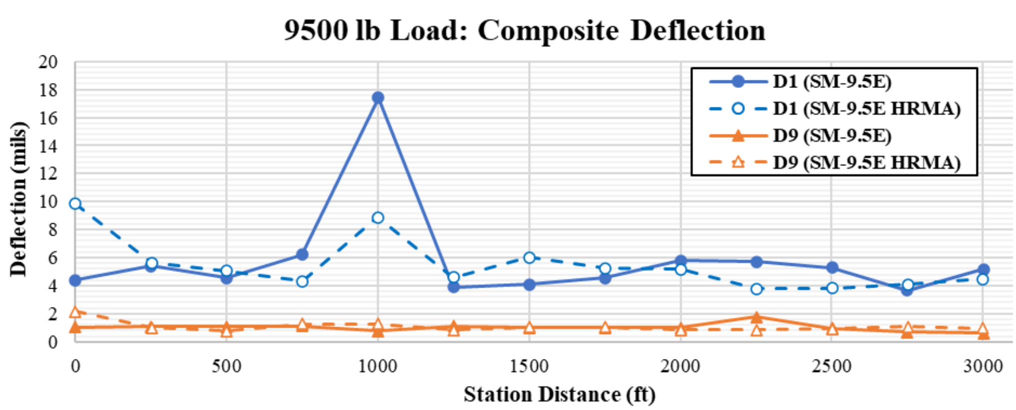


Figure 24. Deflection Results for Route 120 / Glebe Road From Falling Weight Deflectometer (FWD) Testing. D1 = deflection at loading plate measured with the first sensor; D9 = deflection at 72 inches from loading plate measured with the last sensor.

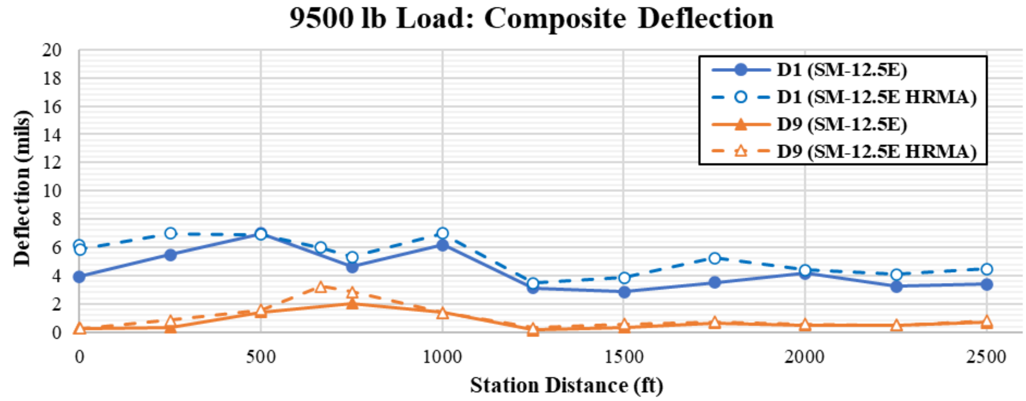


Figure 25. Deflection Results for Route 625 / Waxpool Road From Falling Weight Deflectometer (FWD) Testing. D1 = deflection at loading plate measured with the first sensor; D9 = deflection at 72 inches from loading plate measured with the last sensor.

GPR Test Results and Analyses

An example of the GPR data indicating the different layers for Route 625 / Waxpool Road is shown in Figure 26. Full-depth cores were not taken to confirm the thickness of the layers for this section. Further, the PMS construction history, usually used for comparison purposes, was also not available for this section.

An example GPR scan for Route 120 / Glebe Road is shown in Figure 27. Three distinct layers were shown in the analysis. The PMS construction history for this section showed 5 in of asphalt over an 8-in jointed reinforced concrete pavement with a 6-in crushed aggregate base.

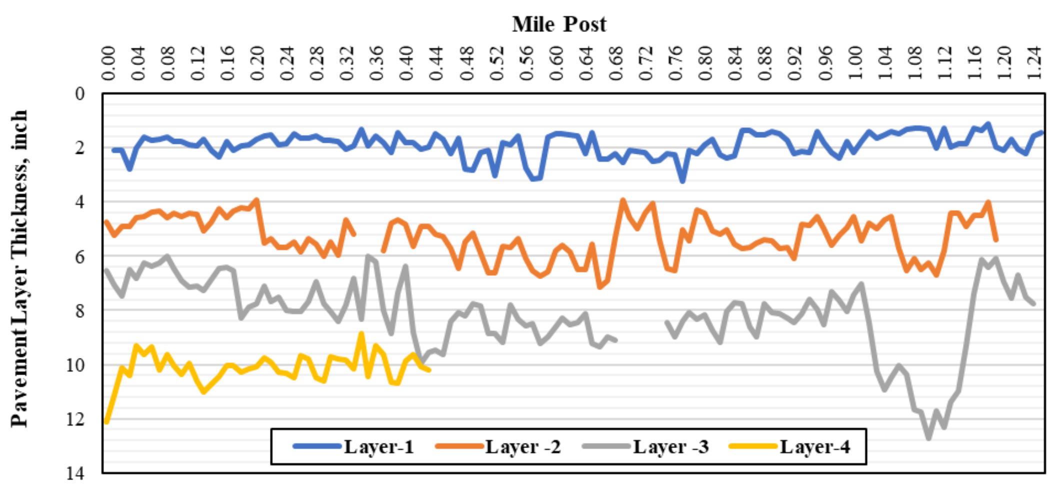


Figure 26. Pavement Layer Thickness Data for Ground Penetrating Radar (GPR) Analysis for Route 625 / Waxpool Road

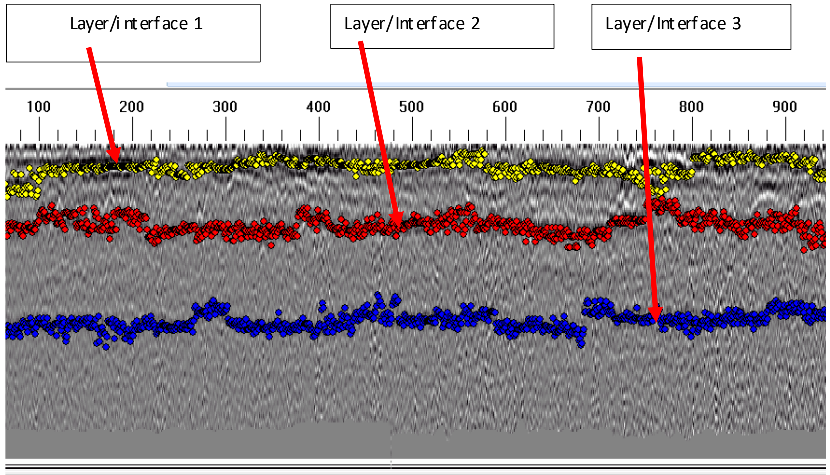


Figure 27. Screenshot of Detected Pavement Layers After Ground Penetrating Radar Analysis for Route 120 / Glebe Road

In-Service Conditions of Evaluated Pavement Structures

The HRMA and E sections are in very good condition (after 2 years), as shown in Figure 28. Since the evaluated sections were placed during the 2021 construction season, the 2-year performance data and corresponding observations are still considered preliminary. The research team will continue to monitor the performance of the HRMA and E sections evaluated in this study.

The PMS data (before and after paving) for the Route 120 / Glebe Road and Route 625 / Waxpool Road sections are shown in Table 10. There are no major distresses reported for these two sections. As mentioned before, the IRI value was high for constructed sections, indicating a rough pavement.

(a) **(b)**

Figure 28. Photographs Taken 19 Months Post-Paving: (a) Route 120 / Glebe Road; (b) Route 625 / Waxpool Road.

Table 10. PMS Performance of Pavements: HRMA Mixtures Placed in 2021

Section ID	Rte. 120 / Glebe Rd.		Rte. 625 / Waxpool Rd.	
Mixture Type	SM-9.5E HRMA		SM-12.5E HRMA	
Year / PMS Property	2021 (Before Paving)	2022	2021 (Before Paving)	2022
CCI	27	87	44	92
LDR	67	100	52	95
NDR	27	87	44	92
IRI (in/mi)	221	208	178	143
Rut depth (in)	0.19	0.06	0.17	0.06

PMS = pavement management system; ID = identification; SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; CCI = critical condition index; LDR = load related distress rating; NDR = non-load related distress rating; IRI = international roughness index.

CONCLUSIONS

- *HRMA asphalt binders can be formulated at the terminal to meet VDOT "E" specifications when products of proper quality are used. No changes from routine established practices in terms of formulation and dosage at the plant were reported for HRMA binders.*

- *No changes from standard practices in terms of surface preparation, production at the plant, or paving operations were reported for HRMA mixtures.*

- *The results for the binders tested in this study indicated that the modification of asphalt binders with HRMA can improve their resistance to cracking and loss of flexibility while also reducing their susceptibility to long-term aging.*

- *Based on the results for the mixtures tested in this study, HRMA modification could be as beneficial as traditional SBS modification and could provide similar and better performance properties and characteristics for the resultant mixtures. Further, there was no significant difference in the performance of HRMA and traditional E mixtures based on the three levels of testing complexity considered in this study.*

- *Based on the results of the ME simulations in this study, HRMA modification can provide a similar or extended predicted in-service life when compared to traditional E mixtures.*

- *Based on the results for the sections evaluated in this study, use of the HRMA mixtures provides similar or better functional surface characteristics (IRI, rut depth, and texture) when compared to regular polymer-modified mixtures.*

- *Both HRMA sections are in very good condition. Since both sections were placed in 2021, performance data are considered preliminary. Continued monitoring of the performance of the sections will be needed to quantify accurately any potential cost-savings in comparison with other polymer-modified surface mixtures.*

RECOMMENDATIONS

1. *VDOT's Materials Division and VDOT districts should consider the use of HRMA surface mixtures as an alternative to the current use of regular SBS-modified surface mixtures on higher-volume facilities.* The data presented and discussed in this report showed similar or better laboratory performance, predicted long-term performance through ME analysis, and initial field performance of HRMA mixtures when compared to typical SBS-modified surface mixtures.

2. *VTRC should continue to monitor the performance of the HRMA sections evaluated in this study.* This will help in predicting the service life of the HRMA overlays in this study in a more accurate manner as the existing sections continue to age.

3. *VDOT districts should consider conducting additional field trials using HRMA mixtures for the purpose of a benefit-cost evaluation.* Control sections with traditional SBS-modified E mixtures should be included in these field trials for comparison of laboratory and field performance. This will help in evaluating the cost-effectiveness of using HRMA mixtures as part of future projects when a more accurate representation of material costs is available.

IMPLEMENTATION AND BENEFITS

Researchers and the technical review panel (listed in the Acknowledgments) for the project collaborate to craft a plan to implement the study recommendations and to determine the benefits of doing so. This is to ensure that the implementation plan is developed and approved with the participation and support of those involved with VDOT operations. The implementation plan and the accompanying benefits are provided here.

Implementation

With regard to Recommendation 1, VTRC will work with VDOT's Central Office Materials Division to modify the most recent version of the VDOT Special Provision for Type E Asphalt Concrete Mixtures in order to include the possible use of HRMA technology as another alternative for binder modification by means of wet process. This work is expected to be completed by spring 2025.

With regard to Recommendation 2, VTRC will monitor the performance of HRMA sections in Virginia for the next 3 to 5 years in order to capture a more representative documentation of field performance for this type of paving material. VTRC will coordinate and collect performance data for the sections evaluated in this study annually from VDOT's PMS and share the data with VDOT's Northern Virginia District Materials Division and VDOT's Central Office Materials Division. This effort will be documented as part of a technical assistance project or a technical brief.

With regard to Recommendation 3, VTRC will work with VDOT districts to identify additional field projects for using HRMA technology in the 2024 and 2025 construction seasons.

Benefits

This project assessed the viability of using HRMA technology as an additional tool to modify asphalt binders in Virginia. Currently, more 400,000 tons (based on 2021 paving data) of typical SBS-modified asphalt surface mixtures are being produced and placed every year in Virginia. Any potential shortage of SBS polymers will cause a significant impact on the asphalt industry. Such a shortage could lead to an increase in the cost of asphalt mixture production; could affect the quality of the produced asphalt mixtures, resulting in reduced performance and durability; and could lead to delays in road construction and maintenance projects as suppliers might not be able to meet the demand for these materials. Therefore, HRMA additives can be used as an alternative and sustainable solution to modify asphalt mixtures in Virginia. HRMA mixtures were found to provide a similar or even better performance when compared to SBS-modified asphalt mixtures, while also being environmentally friendly. These mixtures feature the use of GTR, which can support sustainability in several ways such as reducing the amount of waste tires in landfills, increasing the cost savings for VDOT, and reducing the carbon footprint associated with road construction and maintenance activities.

Implementing Recommendation 1 will help to increase the use of HRMA technology in asphalt mixtures. *Implementing Recommendations 2 and 3* will help in assessing the benefit-cost of the use of these mixtures in VDOT pavements.

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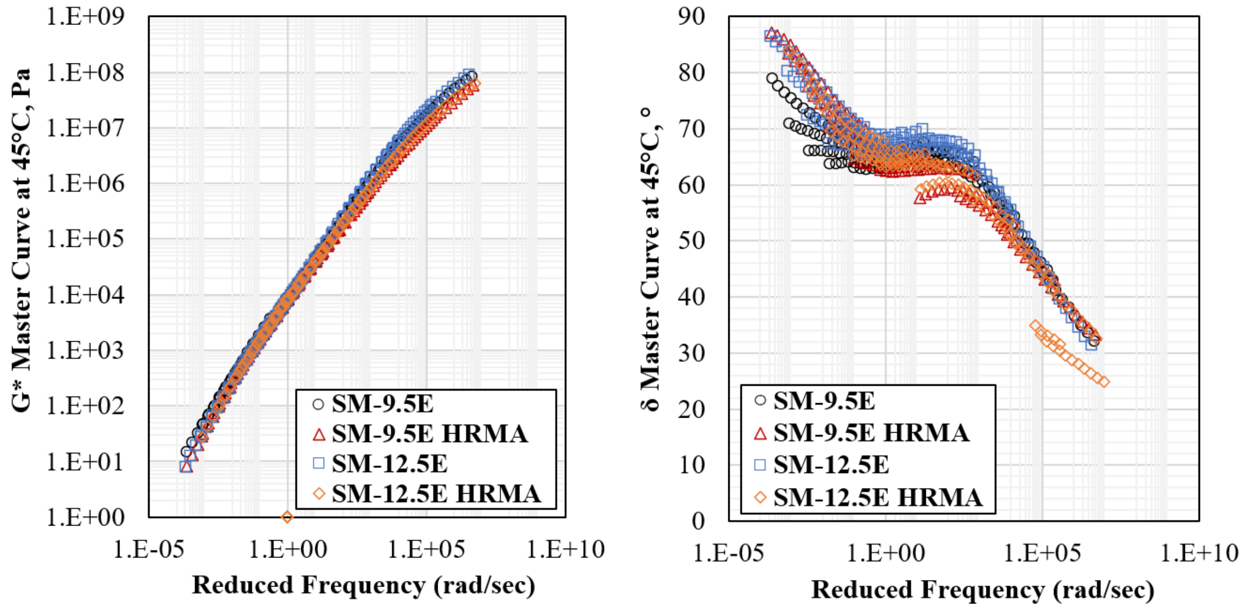
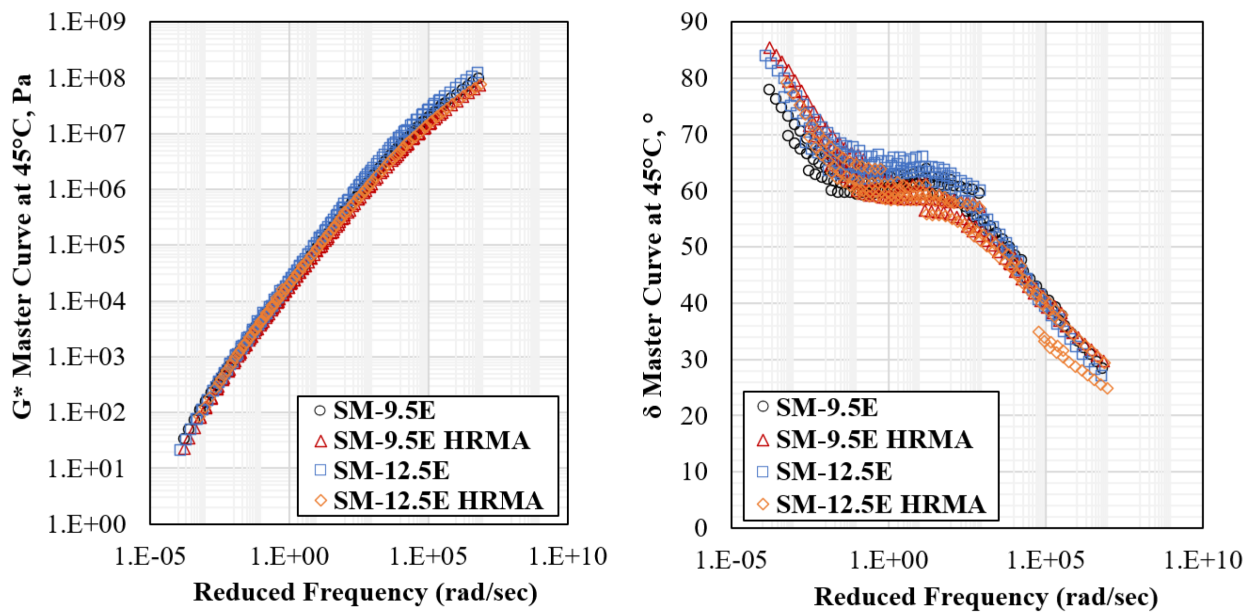
APPENDIX A

ASPHALT BINDER G* and δ MASTER CURVES



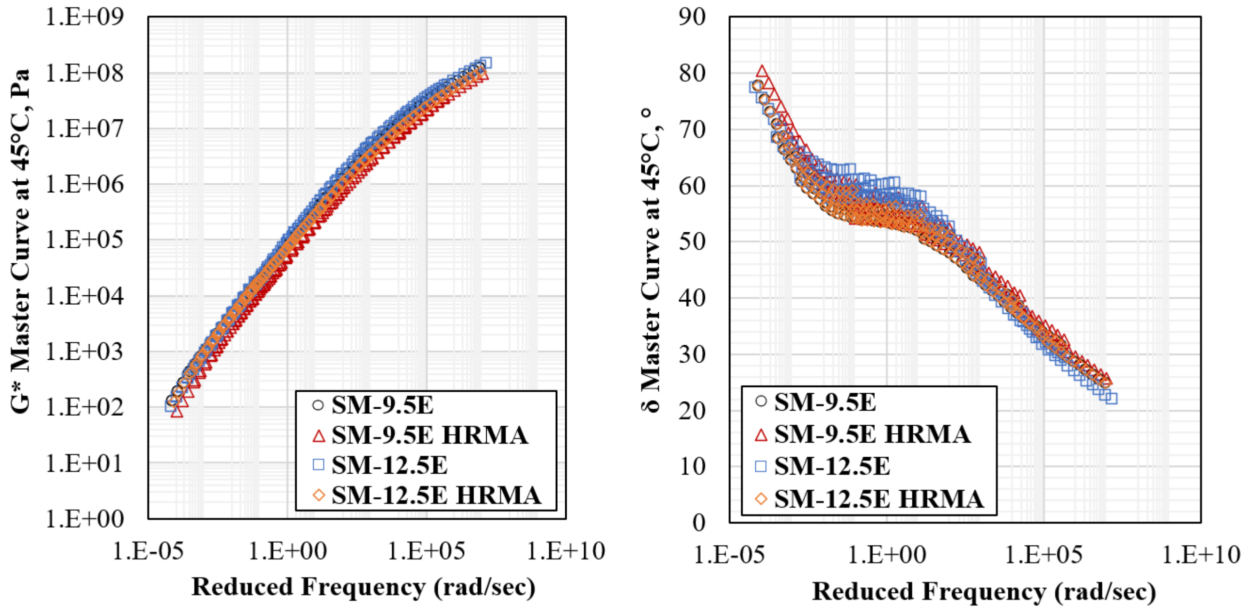
(a) (b)

Figure A1. Performance Test Data in Terms of Master Curves at 45°C for All Evaluated Asphalt Binders at Original Conditions: (a) Dynamic Shear Modulus (G*); (b) Phase Angle (δ). SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt.



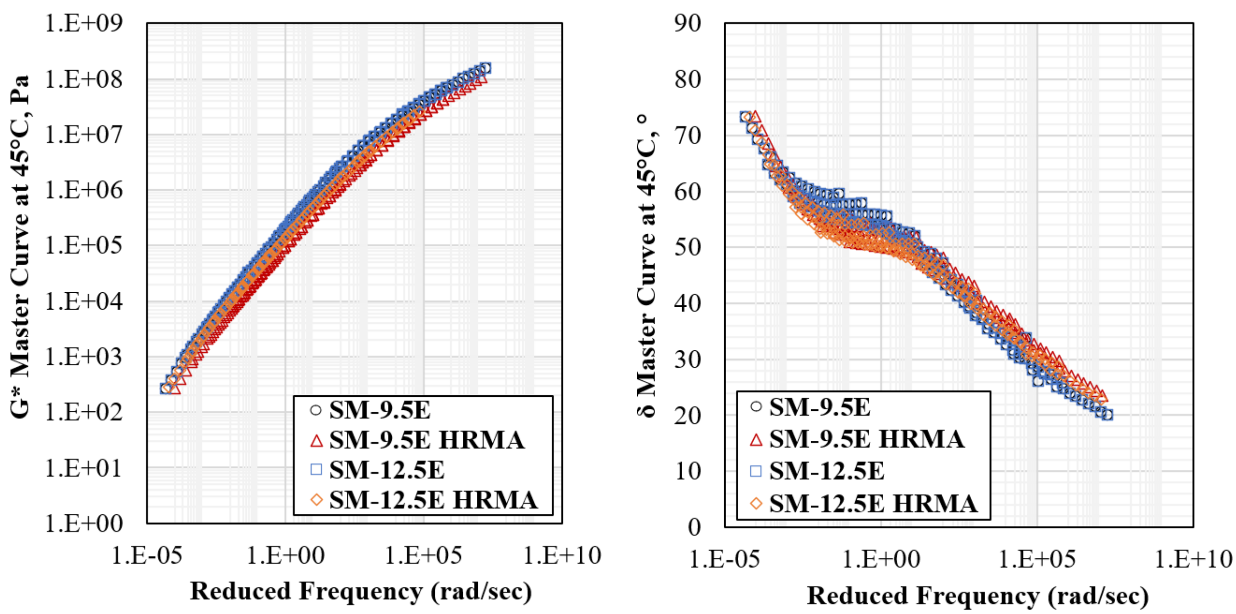
(a) (b)

Figure A2. Performance Test Data in Terms of Master Curves at 45°C for All Evaluated Asphalt Binders at Short-Term Aged RTFO Conditions: (a) Dynamic Shear Modulus (G*); (b) Phase Angle (δ). SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; RTFO = rolling thin film oven.



(a) (b)

Figure A3. Performance Test Data in Terms of Master Curves at 45°C for All Evaluated Asphalt Binders at 20-Hour PAV Aging Conditions: (a) Dynamic Shear Modulus (G*); (b) Phase Angle (δ). SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; PAV = pressure aging vessel.



(a) (b)

Figure A4. Performance Test Data in Terms of Master Curves at 45°C for All Evaluated Asphalt Binders at 40-Hour PAV Aging Conditions: (a) Dynamic Shear Modulus (G*); (b) Phase Angle (δ). SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; PAV = pressure aging vessel.

APPENDIX B

VOLUMETRIC PROPERTIES AND GRADATIONS

Table B1. Volumetric Properties and Gradations for SM-9.5E

Mixture Type	SM-9.5E				
Virgin Binder Grade	PG 64E-22				
RAP Content, %	15				
Additives	Warm mix additives				
Sample			**Set 1**		
	JMF	**Process Tolerance**	**Producer**	**VDOT**	**VTRC**
Property					
NMAS, mm	9.5	-	9.5	9.5	9.5
Asphalt Content, %	5.90	±0.60	5.69	5.64	5.61
Rice SG (G_{mm})	2.500	-	2.560	2.557	2.560
VTM, %	3.5	2.0-5.0	3.4	3.0	3.1
VMA, %	17.2	Min. 16.0	16.9	16.3	16.3
VFA, %	77.6	70.0-85.0	80.0	82.0	81.2
FA Ratio	1.00	0.7-1.3	1.10	1.10	1.11
Mixture Bulk SG (G_{mb})	2.412	-	2.472	2.481	2.481
Aggregate Effective SG (G_{se})	-	-	-	-	2.808
Aggregate Bulk SG (G_{sb})	-	-	-	-	2.800
Absorbed Asphalt Content (P_{ba}), %	-	-	-	-	0.10
Effective Asphalt Content (P_{be}), %	-	-	5.60	5.55	5.51
Effective Film Thicness (F_{be}), µm	-	-	-	-	9.4
Gradation, percent passing					
¾ in (19.0 mm)	100.0	±8.0	100.0	100.0	100.0
½ in (12.5 mm)	100.0	±8.0	100.0	100.0	100.0
3/8 in (9.5 mm)	97.0	±8.0	97.0	97.0	96.7
No. 4 (4.75 mm)	61.0	±8.0	58.0	60.0	56.6
No. 8 (2.36 mm)	38.0	±8.0	38.0	39.0	37.3
No. 16 (1.18 mm)	-	-	27.0	27.0	26.3
No. 30 (600 µm)	20.0	±6.0	20.0	20.0	19.7
No. 50 (300 µm)	-	-	15.0	15.0	14.7
No. 100 (150 µm)	-	-	10.0	10.0	10.2
No. 200 (75 µm)	6.0	±2.0	6.1	6.3	6.1

PG = performance grade; E = extremely heavy traffic; RAP = reclaimed asphal pavement; JMF = job-mix formula; VDOT = Virginia Department of Transportation; VTRC = Virginia Transportation Research Council; NMAS = nominal maximum aggregate size; SG = specific gravity; VTM voids in total mixture; VMA = voids in miniral aggregate; VFA = voids filled with asphalt; FA = fines to aggregate; - = not available.
[a] Process tolerance for one test from Table II-15 (VDOT, 2020).

Table B2. Volumetric Properties and Gradations for SM-9.5E HRMA

Mixture Type	SM-9.5E HRMA							
Virgin Binder Grade	PG 64E-22 HRMA							
RAP Content, %	15							
Additives	Warm mix additives							
Sample		**Process**	**Set 1**			**Set 2**		
	JMF	**Tolerance**	**Producer**	**VDOT**	**VTRC**	**Producer**	**VDOT**	**VTRC**
Property								
NMAS, mm	9.5	-	9.5	9.5	9.5	9.5	-	9.5
Asphalt Content, %	5.90	±0.43	5.65	5.61	5.80	5.62	-	5.99
Rice SG (G_{mm})	2.500	-	2.576	2.564	2.573	2.557	-	2.544
VTM, %	3.5	2.0-5.0	4.5	3.3	3.6	4.2	-	3.3
VMA, %	17.2	Min. 16.0	17.7	16.6	17.3	17.4	-	17.4
VFA, %	77.6	70.0 - 85.0	75.0	80.0	79.2	76.0	-	80.9
FA Ratio	1.0	0.7-1.3	1.0	1.0	1.04	1.10	-	1.06
Mixture Bulk SG (G_{mb})	2.412	-	2.461	2.479	2.481	2.449	-	2.460
Aggregate Effective SG (G_{se})	-	-	-	-	2.835	-	-	2.807
Aggregate Bulk SG (G_{sb})	-	-	-	-	2.827	-	-	2.799
Absorbed Asphalt Content (P_{ba}), %	-	-	-	-	0.10	-	-	0.10
Effective Asphalt Content (P_{be}), %	-	-	5.56	5.52	5.70	5.52	-	5.89
Effective Film Thicness (F_{be}), µm	-	-	-	-	10.1	-	-	9.8
Gradation, percent passing								
¾ in (19.0 mm)	100.0	±5.7	100.0	100.0	100.0	100.0	-	100.0
½ in (12.5 mm)	100.0	±5.7	100.0	100.0	99.9	100.0	-	100.0
3/8 in (9.5 mm)	97.0	±5.7	95.0	96.0	97.1	96.0	-	97.5
No. 4 (4.75 mm)	61.0	±5.7	56.0	56.0	58.2	60.0	-	61.3
No. 8 (2.36 mm)	38.0	±5.7	35.0	36.0	37.4	40.0	-	41.1
No. 16 (1.18 mm)	-	-	24.0	24.0	25.3	27.0	-	28.1
No. 30 (600 µm)	20.0	±4.3	18.0	18.0	18.5	20.0	-	20.4
No. 50 (300 µm)	-	-	13.0	13.0	13.7	15.0	-	14.8
No. 100 (150 µm)	-	-	9.0	9.0	9.5	10.0	-	10.1
No. 200 (75 µm)	6.0	±1.4	5.3	5.6	6.0	6.0	-	6.2

PG = performance grade; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; RAP = reclaimed asphal pavement; JMF = job-mix formula; VDOT = Virginia Department of Transportation; VTRC = Virginia Transportation Research Council; NMAS = nominal maximum aggregate size; SG = specific gravity; VTM voids in total mixture; VMA = voids in miniral aggregate; VFA = voids filled with asphalt; FA = fines to aggregate; - = not available.

[a] Process tolerance for two tests from Table II-15 (VDOT, 2020).

Table B3. Volumetric Properties and Gradations for SM-12.5E

Mixture Type	SM-12.5E							
Virgin Binder Grade	PG 64E-22							
RAP Content, %	15							
Additives	Warm mix additives							
Sample		**Process**	**Set 1**			**Set 2**		
	JMF	**Tolerance**	**Producer**	**VDOT**	**VTRC**	**Producer**	**VDOT**	**VTRC**
Property								
NMAS, mm	12.5	-	12.5	12.5	12.5	12.5	-	12.5
Asphalt Content, %	5.40	±0.43	5.34	5.28	5.45	5.49	-	5.20
Rice SG (G_{mm})	2.713	-	2.702	2.704	2.707	2.691	-	2.716
VTM, %	3.5	2.0 - 5.0	4.3	4.4	4.5	4.4	-	4.9
VMA, %	16.7	Min. 15.0	16.9	16.9	17.5	17.4	-	17.2
VFA, %	79.0	68.0 - 84.0	75.0	74.0	74.1	75.0	-	71.4
FA Ratio	1.2	0.7-1.3	1.10	1.10	1.08	1.00	-	1.03
Mixture Bulk SG (G_{mb})	2.617	-	2.587	2.585	2.584	2.573	-	2.582
Aggregate Effective SG (G_{se})	-	-	-	-	2.987	-	-	2.983
Aggregate Bulk SG (G_{sb})	-	-	-	-	2.961	-	-	2.957
Absorbed Asphalt Content (P_{ba}), %	-	-	-	-	0.30	-	-	0.30
Effective Asphalt Content (P_{be}), %	-	-	5.05	4.99	5.16	5.20	-	4.91
Effective Film Thicness (F_{be}), µm	-	-	-	-	10.1	-	-	10.3
Gradation, percent passing								
¾ in (19.0 mm)	100.0	±5.7	100.0	100.0	100.0	100.0	-	100.0
½ in (12.5 mm)	98.0	±5.7	97.0	97.0	97.7	96.0	-	96.3
3/8 in (9.5 mm)	87.0	±5.7	86.0	84.0	85.7	85.0	-	80.3
No. 4 (4.75 mm)	59.0	±5.7	52.0	51.0	51.6	52.0	-	47.1
No. 8 (2.36 mm)	40.0	±5.7	33.0	33.0	32.8	33.0	-	30.2
No. 16 (1.18 mm)	-	-	22.0	22.0	22.4	22.0	-	21.2
No. 30 (600 µm)	19.0	±4.3	16.0	16.0	16.1	16.0	-	15.2
No. 50 (300 µm)	-	-	12.0	12.0	11.8	12.0	-	11.0
No. 100 (150 µm)	-	-	8.0	8.0	8.3	8.0	-	7.7
No. 200 (75 µm)	5.9	±1.4	5.6	5.5	5.6	5.4	-	5.1

Red text indicates value that did not meet allowable tolerance. PG = performance grade; E = extremely heavy traffic; RAP = reclaimed asphal pavement; JMF = job-mix formula; VDOT = Virginia Department of Transportation; VTRC = Virginia Transportation Research Council; NMAS = nominal maximum aggregate size; SG = specific gravity; VTM voids in total mixture; VMA = voids in miniral aggregate; VFA = voids filled with asphalt; FA = fines to aggregate; - = not available.
[a] Process tolerance for two tests from Table II-15 (VDOT, 2020).

Table B4. Volumetric Properties and Gradations for SM-12.5E (HRMA)

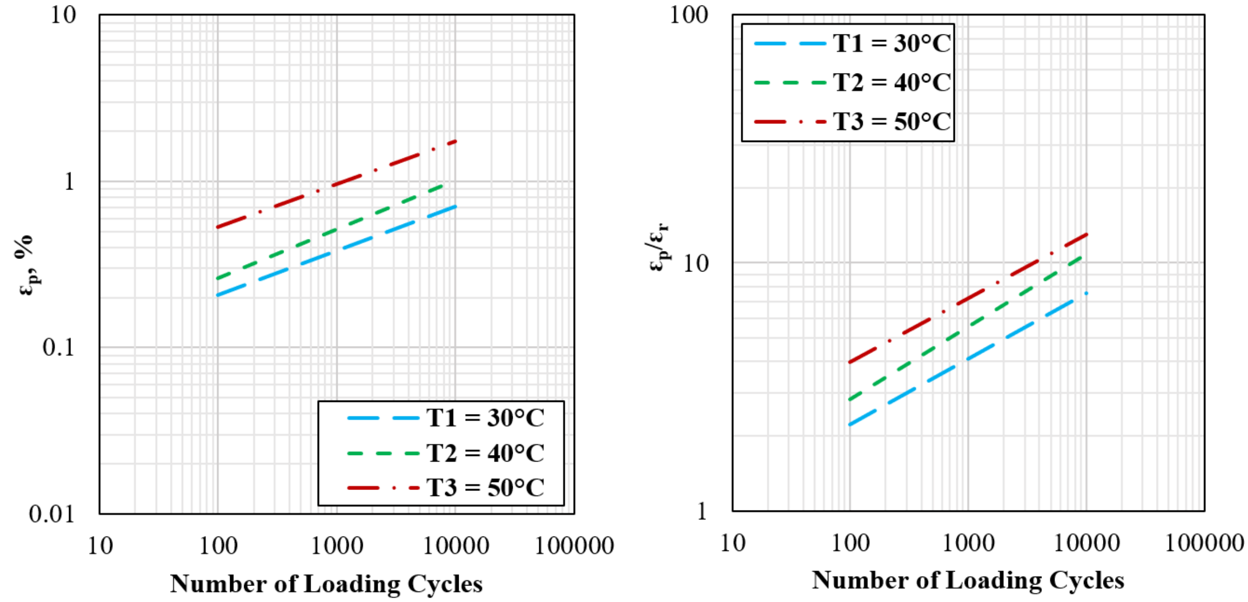
Mixture Type	SM-12.5E (HRMA)							
Virgin Binder Grade	PG 64E-22 (HRMA)							
RAP Content, %	15							
Additives	Warm mix additives							
Sample		**Process**	**Set 1**			**Set 2**		
	JMF	**Tolerance**	**Producer**	**VDOT**	**VTRC**	**Producer**	**VDOT**	**VTRC**
Property								
NMAS, mm	12.5	-	12.5	12.5	12.5	12.5	12.5	12.5
Asphalt Content, %	5.40	±0.43	5.65	5.57	5.37	5.65	5.45	5.38
Rice SG (G_{mm})	2.713	-	2.703	2.702	2.712	2.700	2.712	2.706
VTM, %	3.5	2.0-5.0	5.2	5.3	5.6	4.9	5.6	5.1
VMA, %	16.7	Min. 15.0	18.5	18.4	18.7	18.2	18.4	18.3
VFA, %	79.0	68.0-84.0	72.0	71.0	70.0	73.0	70.0	72.1
FA Ratio	1.2	0.7-1.3	0.80	0.80	0.74	0.80	0.80	0.82
Mixture Bulk SG (G_{mb})	2.617	-	2.563	2.560	2.559	2.569	2.560	2.568
Aggregate Effective SG (G_{se})	-	-	-	-	2.989	-	-	2.981
Aggregate Bulk SG (G_{sb})	-	-	-	-	2.981	-	-	2.973
Absorbed Asphalt Content (P_{ba}), %	-	-	-	-	0.09	-	-	0.09
Effective Asphalt Content (P_{be}), %	-	-	5.37	5.29	5.28	5.37	5.17	5.29
Effective Film Thicness (F_{be}), µm	-	-	-	-	12.1	-	-	11.3
Gradation, percent passing								
¾ in (19.0 mm)	100.0	±5.7	100.0	100.0	100.0	100.0	100.0	100.0
½ in (12.5 mm)	98.0	±5.7	97.0	96.0	96.1	97.0	95.0	96.3
3/8 in (9.5 mm)	87.0	±5.7	87.0	87.0	83.3	86.0	86.0	84.7
No. 4 (4.75 mm)	59.0	±5.7	58.0	57.0	52.6	58.0	56.0	55.5
No. 8 (2.36 mm)	40.0	±5.7	38.0	37.0	34.6	38.0	37.0	36.9
No. 16 (1.18 mm)	-	-	24.0	24.0	23.5	25.0	25.0	24.8
No. 30 (600 µm)	19.0	±4.3	17.0	17.0	16.1	17.0	17.0	17.0
No. 50 (300 µm)	-	-	11.0	11.0	10.6	12.0	12.0	11.5
No. 100 (150 µm)	-	-	7.0	7.0	6.6	8.0	7.0	7.3
No. 200 (75 µm)	5.9	±1.4	4.3	4.1	3.9	4.4	4.2	4.3

Red text indicates value that did not meet allowable tolerance. PG = performance grade; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; RAP = reclaimed asphal pavement; JMF = job-mix formula; VDOT = Virginia Department of Transportation; VTRC = Virginia Transportation Research Council; NMAS = nominal maximum aggregate size; SG = specific gravity; VTM = voids in total mixture; VMA = voids in miniral aggregate; VFA = voids filled with asphalt; FA = fines to aggregate; - = not available.

[a] Process tolerance for two tests from Table II-15 (VDOT, 2020).

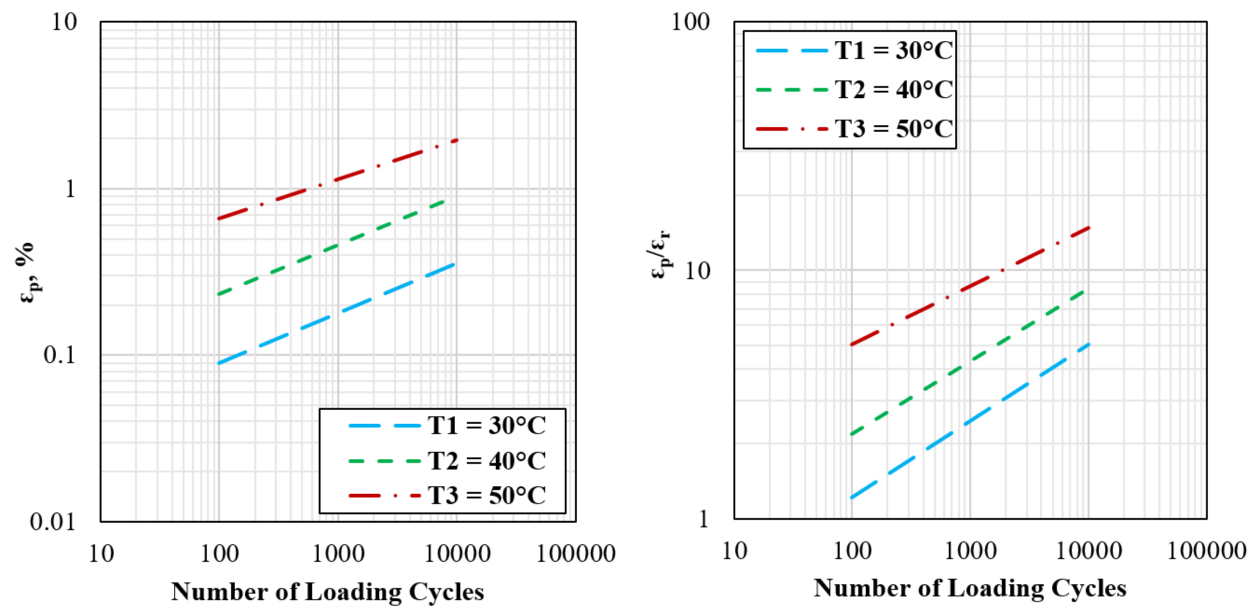
APPENDIX C

ADVANCED PERFORMANCE CHARACTERISTICS: RUTTING—REPEATED TRIAXIAL LOAD TEST



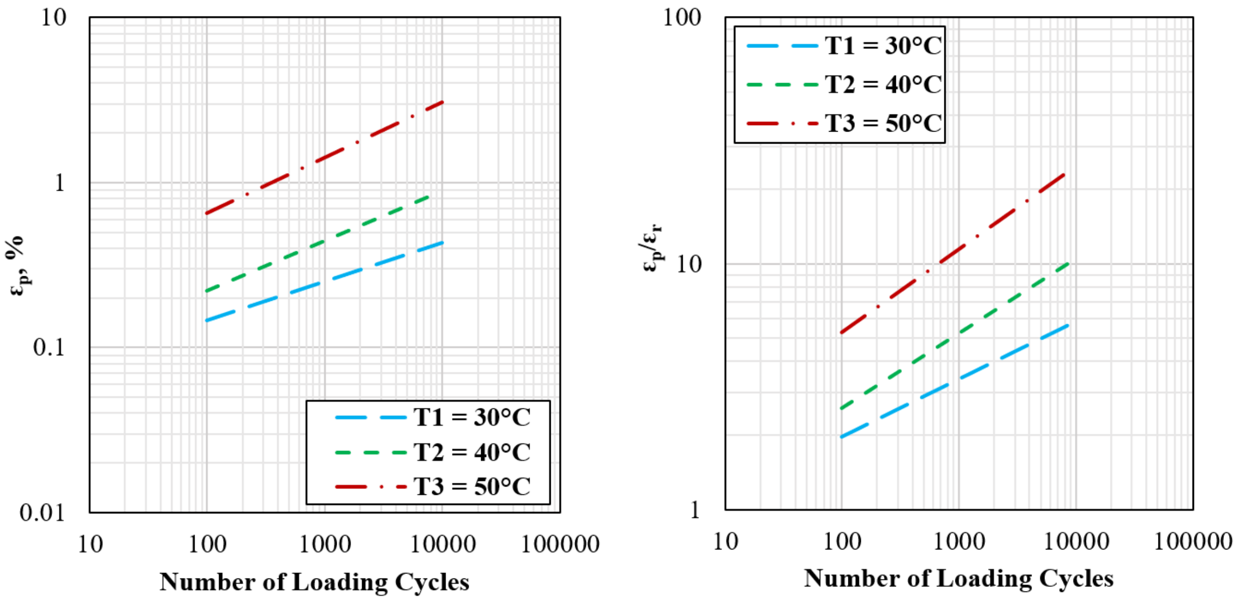
(a) (b)

Figure C1. Rutting Performance Characteristics of SM-9.5E at 30, 40, and 50°C: (a) ε_p; (b) $\varepsilon_p/\varepsilon_r$. SM = surface mixture; E = extremely heavy traffic; ε_p = permeant axial strain; ε_r = resilient axial strain; T = temperature.



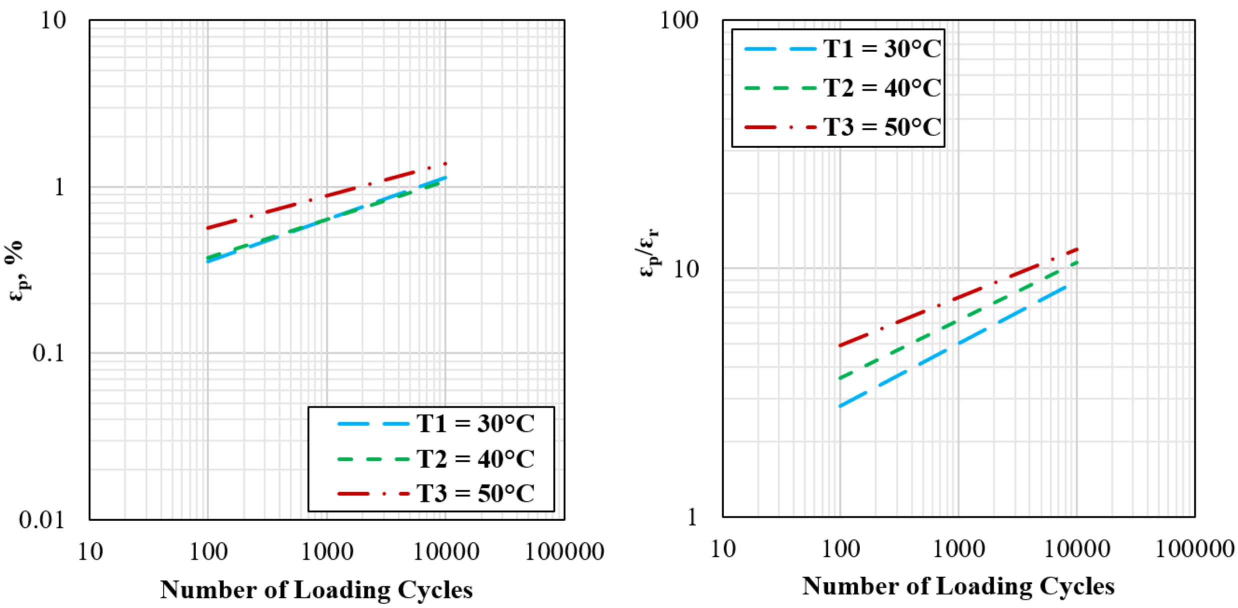
(a) (b)

Figure C2. Rutting Performance Characteristics of SM-9.5E HRMA at 30, 40, and 50°C: (a) ε_p; (b) $\varepsilon_p/\varepsilon_r$. SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; ε_p = permeant axial strain; ε_r = resilient axial strain; T = temperature.



(a) (b)

Figure C3. Rutting Performance Characteristics of SM-12.5E at 30, 40, and 50°C: (a) ε_p; (b) $\varepsilon_p/\varepsilon_r$. SM = surface mixture; E = extremely heavy traffic; ε_p = permeant axial strain; ε_r = resilient axial strain; T = temperature.



(a) (b)

Figure C4. Rutting Performance Characteristics of SM-12.5E HRMA at 30, 40, and 50°C: (a) ε_p;(b) $\varepsilon_p/\varepsilon_r$. SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; ε_p = permeant axial strain; ε_r = resilient axial strain; T = temperature.

The RLT test was conducted at three different temperatures: 86, 104, and 122°F (30, 40, and 50°C) for all evaluated E and HRMA mixtures. A rutting laboratory model for each mixture was developed following Equation C.1 based on the approach recommended in the MEPDG. Table C1 summarizes the regression coefficients of the rutting models

$$\frac{\varepsilon_p}{\varepsilon_r} = 10^{k_{r1}} * (T)^{k_{r2}} * (N)^{k_{r3}} \quad \text{[Eq. C1]}$$

where

ε_p = permanent axial strain, inch/inch (or mm/mm)
ε_r = resilient axial strain, inch/inch (or mm/mm)
N = number of loading cycles
T = temperature of the asphalt mixture in °F
k_{r1}, k_{r2}, and k_{r3} = experimentally determined coefficients.

Table C1. Summary of MEPDG Rutting Model Coefficients for E and HRMA Mixtures

Mixture ID	Rutting Model Coefficients		
	k_{r1}	k_{r2}	k_{r3}
SM-9.5E	-3.285848	1.595289	0.272614
SM-9.5E HRMA	-6.707707	3.252520	0.274721
SM-12.5E	-5.580451	2.747826	0.292435
SM-12.5E HRMA	-1.885402	0.999987	0.236201

MEPDG = mechanistic-empirical pavement design guide; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt; SM = surface mixture.

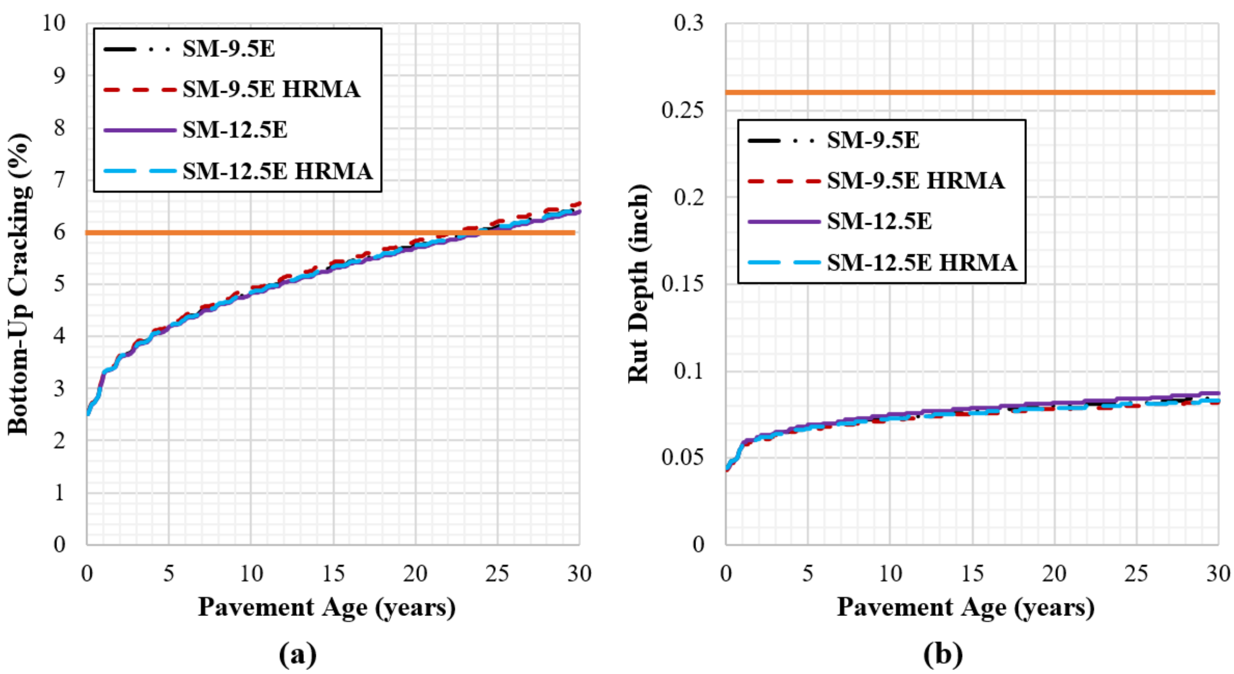


Figure C5. Pavement ME Design Simulation Results for Structure (i): (a) fatigue performance; (b) rutting performance. SM = surface mixture; E = extremely heavy traffic; HRMA = hybrid rubber modified asphalt. Orange solid lines indicates performance criteria.

APPENDIX D

NON-DESTRUCTIVE TESTING RESULTS

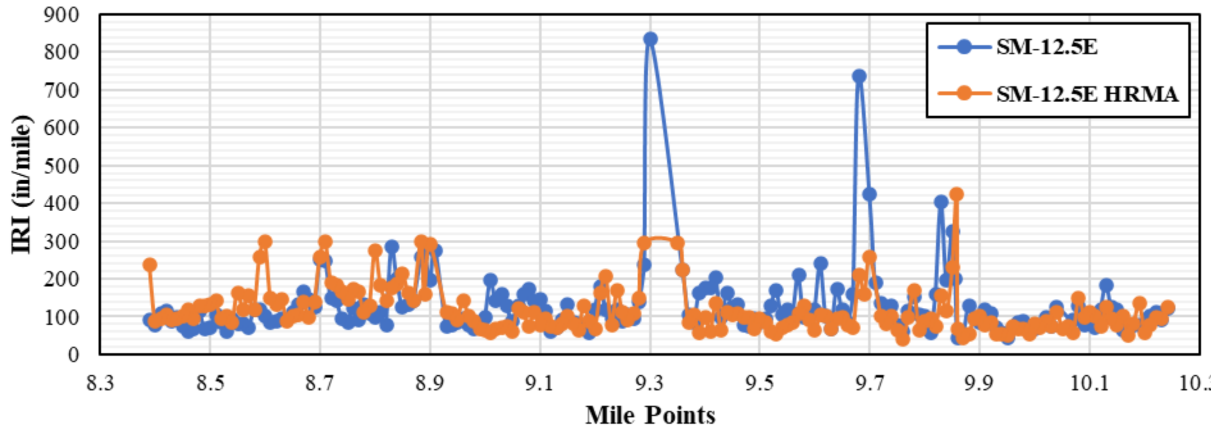


Figure D1. International Roughness Index (IRI) Results for Route 625 / Waxpool Road

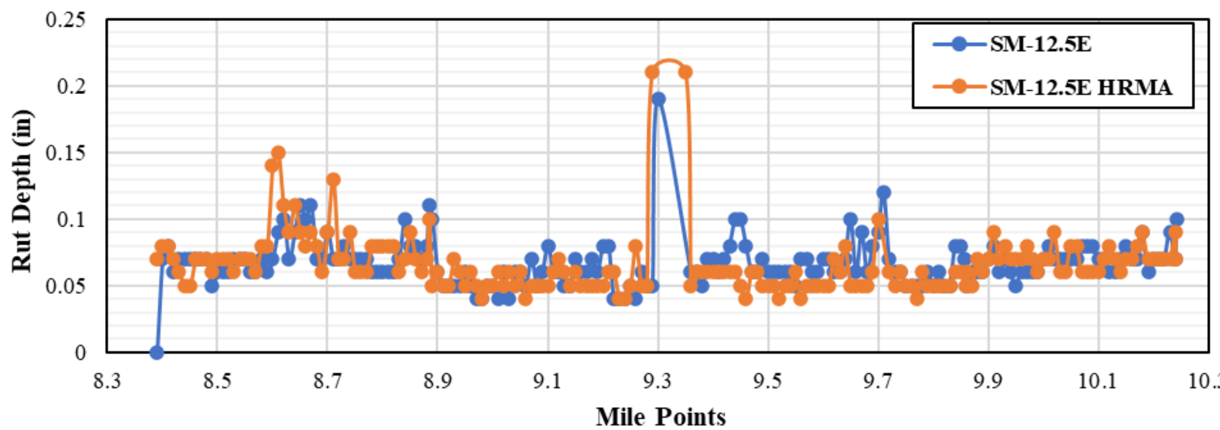


Figure D2. Rut Depth Results for Route 625 / Waxpool Road

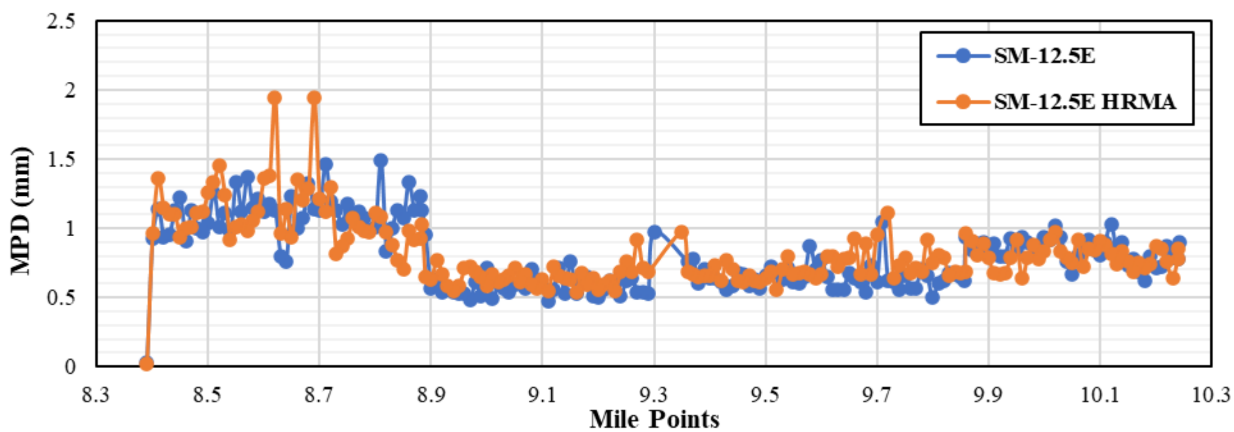


Figure D3. Mean Profile Depth (MPD) Results for Route 625 / Waxpool Road