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Laboratory Evaluation of a Warm Asphalt Technology for Use in Virginia

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16. Abstract:

Rising energy costs and increased environmental awareness have brought attention to the potential benefits of warm asphalt in the United States. Warm-mix asphalt (WMA) is produced by incorporating additives into asphalt mixtures to allow production and placement of the mix when heated to temperatures well below the 300ºF+ temperatures of conventional hot-mix asphalt (HMA). Potential benefits such as reduced plant emissions, workability at lower temperatures, extension of the paving season into colder weather, and reduced energy consumption at the plant may be realized with different applications.

Trial installations of WMA, including two sections using the Sasobit WMA additive, have been investigated in Virginia. This study presents the results of laboratory testing to evaluate the performance of the mixtures used in the two Sasobit trial sections. The evaluation included comparisons of compactibility, volumetric properties, moisture susceptibility, rutting resistance, and fatigue performance between the HMA and WMA mixtures used in each section. Mixtures produced in the laboratory under conditions of varying temperatures and aging periods were tested, and the effects of temperature and aging were evaluated. The long-term performance of the two test sections was also modeled using the *Mechanistic-Empirical Pavement Design Guide.*

Few differences were found between the HMA and WMA mixtures evaluated. The performance of WMA and HMA was similar when evaluated for moisture susceptibility, rutting potential, and fatigue resistance. The MEPDG-predicted distresses supported these conclusions; the predicted long-term performance of WMA and HMA was comparable. From these results, the recommendation was made that the Virginia Department of Transportation develop a special provision for the use of WMA.

Despite its benefits, direct cost savings from the use of WMA are unlikely to be seen by VDOT. Currently, one concern with the use of WMA is the initial cost, which varies depending on the technology used. The use of WMA technology requires either additives, a recurrent cost, or asphalt plant modifications, requiring capital investment. Over the long term, the use of WMA could save VDOT considerable dollars if the reduced aging of the mix translates into longer life; however, this has yet to be proven as WMA has not been employed for a sufficient time period to allow an evaluation of this benefit.

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FINAL REPORT

LABORATORY EVALUATION OF A WARM ASPHALT TECHNOLOGY FOR USE IN VIRGINIA

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Virginia Transportation Research Council
(A partnership of the Virginia Department of Transportation
and the University of Virginia since 1948)

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EXECUTIVE SUMMARY

Introduction

Rising energy costs and increased environmental awareness have brought attention to the potential benefits of warm asphalt in the United States. Warm-mix asphalt (WMA) is produced by incorporating additives into asphalt mixtures to allow production and placement of the mix when heated to temperatures well below the 300ºF+ temperatures of conventional hot-mix asphalt (HMA). Potential benefits such as reduced plant emissions, workability at lower temperatures, extension of the paving season into colder weather, and reduced energy consumption at the plant may be realized with different applications.

Purpose and Scope

Trial installations of WMA, including two sections using the Sasobit WMA additive, have been investigated in Virginia (Diefenderfer et al., 2007). This study presents the results of laboratory testing to evaluate the performance of the mixtures used in the two trial sections. The evaluation included comparisons of compactibility, volumetric properties, moisture susceptibility, rutting resistance, and fatigue performance between the HMA and WMA mixtures used in each section. Mixtures produced in the laboratory under conditions of varying temperatures and aging periods were tested, and the effects of temperature and aging were evaluated. The long-term performance of the two test sections was also modeled in accordance with the *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*, or the Mechanistic-Empirical Pavement Design Guide (MEPDG), developed by the National Cooperative Highway Research Program (NCHRP) Project 1-37A (NCHRP, 2004).

Methods

The two mixtures evaluated were designated Mixture A and Mixture B. Mixture A was a Superpave 9.5 mm nominal maximum aggregate size (NMAS) surface mix produced using Performance Graded (PG) 64-22 binder, designated as a Virginia Department of Transportation (VDOT) SM-9.5A mixture. Morelife 3300 antistrip was used at a dosage rate of 0.5% by weight of the binder. The aggregate was a mix of granite and siltstone. This was a typical SM-9.5A mixture used by the contractor; the only adjustment made to the mix design to produce WMA was the addition of Sasobit and the reduction of the production temperature. Sasobit was added at a dosage rate of 1.5% by weight of the binder. The binder content used for the WMA was the same as that used for the HMA.

Mixture B was a Superpave 12.5 mm NMAS surface mix produced using PG 64-22 binder, designated as a VDOT SM-12.5A mixture. Hydrated lime was used as an antistripping agent. The aggregate was a mix of limestone and gravel. Again, this was a typical SM-12.5A mixture used by the contractor. The mix design was the same as for the WMA and HMA mixtures, except for the addition of 1.5% Sasobit by weight of the binder and the reduction of production temperatures.

The study included mixtures produced at the plant and produced in the laboratory. During construction of the two trial sections in 2006, HMA and WMA were sampled at each plant. In addition, for Mixture A, HMA was produced in the laboratory using the plant production temperature and WMA was produced in the laboratory at temperatures of 230ºF, 265ºF, and 300ºF. In all cases, the mixing and compaction temperatures were the same. Finally, specimen sets of the Mixture A WMA were produced in the laboratory with entrapped moisture. For Mixture B, only plant-produced HMA and WMA were evaluated.

The mixtures were evaluated to determine moisture susceptibility, rutting potential, and expected fatigue performance. Moisture susceptibility was evaluated using the tensile strength ratio (TSR) and Hamburg wheel-track tests. Rutting susceptibility was determined using the Asphalt Pavement Analyzer (APA). Fatigue performance was assessed using the third point fatigue test. In addition, road cores were taken and the binder was extracted to evaluate any differences in aging between the HMA and WMA. Mixture and binder properties, along with characteristics of the trial installation sections, were used to predict long-term performance in accordance with the MEPDG.

Results and Discussion

Test results indicated that there were no significant differences in volumetrics between the HMA and the WMA mixtures examined in this study. The plant-produced and laboratory-produced mixtures had similar properties. In addition, this was found true for laboratory mixtures produced at varying temperatures. Initially, WMA appeared to compact more easily than HMA; however, the in-place compaction of all mixtures was the same. Therefore, laboratory and field compaction procedures can be treated the same for WMA as for HMA.

In general, moisture susceptibility should not be a problem with WMA that is properly produced and placed. TSR testing did not provide a complete understanding of moisture susceptibility as, overall, the results did not follow any trends. However, there did appear to be a positive effect from aging for the WMA. The TSR results for WMA produced with entrapped moisture showed that WMA was susceptible to moisture damage when the aggregates were not adequately dried during mixing; this susceptibility decreased when the mix was subjected to short-term oven aging before testing. The Hamburg wheel-track test results showed that the HMA and WMA were resistant to moisture susceptibility and indicated similar performance.

HMA and WMA should be expected to perform similarly in terms of rutting resistance. No significant differences were found for HMA and WMA for rutting susceptibility as indicated by the APA. The plant-produced WMA rutted slightly less than the HMA; however, these differences were not statistically significant. The laboratory-produced WMA was found to rut slightly less when produced at higher temperatures than when produced at low temperatures.

Expectations of fatigue resistance should be the same for HMA and WMA, as laboratory fatigue testing indicated similar performance. The HMA performed slightly better at lower strains than the WMA; however, the performance of the mixes appears nearly equal at higher strains.

Analysis using the MEPDG showed indistinguishable long-term predicted performance by the HMA and WMA for both mixtures.

Conclusions and Recommendations

In conclusion, based on the two mixtures and one WMA technology considered in this study, HMA and WMA should have equivalent performance when properly constructed. Thus, it is recommended that implementation of WMA proceed with a permissive specification to allow the use of reputable and reasonable technologies. Acceptance property requirements for WMA should not differ from those for HMA, with the exception of temperature and TSR values. The WMA technology manufacturer recommendations should be followed for temperature. Test results were not sufficiently conclusive to determine an acceptable value for the TSR. Continued monitoring of existing WMA installations and monitoring of future installations, particularly of different technologies, is suggested to continue to determine the suitability for various uses and to determine any limitations in recommended use. Finally, additional laboratory work to evaluate additional WMA technologies and use with different mixture types (such as stone matrix asphalt and mixtures with a high recycled asphalt pavement content) is recommended.

FINAL REPORT

LABORATORY EVALUATION OF A WARM ASPHALT TECHNOLOGY FOR USE IN VIRGINIA

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INTRODUCTION

The focus in construction has recently shifted toward sustainability and environmentally friendly practices. New technologies in asphalt production and placement have emerged that may save fuel and lower emissions as well as provide other benefits to contractors and transportation agencies. These technologies have been grouped together under the name "warm-mix asphalt" (WMA). Conventional hot-mix asphalt (HMA) is typically produced and compacted at temperatures between 285ºF to 340ºF; cold-mix asphalt is compacted at ambient temperatures (70ºF to 120ºF). WMA falls between the two and is generally defined as asphalt mixtures produced at temperatures between 212ºF and 275ºF. Several WMA technologies are available, including the following:

- *LEA,* Advanced Concepts Engineering Co.
- *CECABASE RT,* Arkema Group
- *Double Barrel Green System,* Astec Industries
- *Evotherm,* MeadWestvaco Asphalt Innovations
- *Advera WMA,* PQ Corporation
- *Terex,* Terex Roadbuilding
- *Sasobit*, Sasol Wax Americas, Inc. (National Asphalt Pavement Association, 2007).

When asphalt is produced at lower temperatures, there are many potential benefits such as reduced emissions and energy consumption and increased worker safety. WMA technologies also allow asphalt to be placed at cooler ambient temperatures and to be hauled farther without compromising workability. The lower production temperatures result in less binder oxidation during production and laydown, which may lead to greater fatigue resistance. Potential drawbacks of the technology include an increased susceptibility to moisture damage since the lower production temperatures may lead to the aggregate not being sufficiently dried before mixing. Additional concerns include an increased potential for rutting, possibly because of less aging (stiffening) of the binder, and compaction issues at the lower placement temperatures. The potential for increased curing times has also been reported, which could mean delays in opening roads to traffic (Newcomb, 2007). However, the answers regarding most of these issues, both positive and negative, have not yet been conclusively determined.

The Virginia Department of Transportation (VDOT) evaluated HMA and WMA mixtures used during two trial sections paved in Virginia in 2006 (Diefenderfer et al., 2007). The WMA was produced with the Sasobit technology. Sasobit is a wax byproduct of the Fischer-Tropsch process of natural gas and coal gasification. It comes in pellets or flakes and is combined with the binder to lower its viscosity (Sasol Wax, 2004). This report discusses the effects of lower temperature production on the compactibility, volumetrics, moisture susceptibility, rutting potential, and fatigue resistance of the two mixtures used during VDOT's field installations.

PURPOSE AND SCOPE

The purpose of this study was to evaluate the use of Sasobit WMA in the laboratory to determine if the technology appeared suitable for adoption by VDOT as an acceptable alternative to HMA.

Laboratory testing was performed to determine if there were significant differences between HMA and Sasobit WMA produced using the same mix designs. Two mix designs, correlating to the two field sections placed previously (Diefenderfer et al., 2007), were considered. Testing included volumetric analysis, moisture susceptibility evaluations, rutting and fatigue analysis, and an evaluation of the impact of wet aggregate during production on mixture performance. An analysis of extracted binder from cores taken from the field installations was also performed to determine if there were significant differences in the effects of aging on WMA as compared to HMA.

In addition, an analysis in accordance with the *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures,* or the Mechanistic-Empirical Pavement Design Guide (MEPDG), developed by the National Cooperative Highway Research Program (NCHRP) Project 1-37A (NCHRP, 2004), was conducted to predict performance measures to determine any differences between the expected long-term performance of the two mixtures.

METHODS

Two tasks were performed to achieve the study objectives:

1. *Laboratory testing was performed to determine if there were significant differences between HMA and Sasobit WMA produced using the same mix designs.* Testing was conducted to determine volumetric properties, moisture susceptibility, rutting potential, and fatigue resistance. In addition, mixtures were produced in the laboratory using different production temperatures and aging treatments to evaluate the effects of temperature and aging on moisture susceptibility. Mixtures were also produced with entrapped moisture to evaluate the effects of insufficient aggregate drying during production. Analysis was also performed on extracted binder from cores taken from the field installations to determine if aging had a significant effect on WMA as compared to HMA.

2. An analysis in accordance with the MEPDG was conducted to predict performance measures to determine any differences between the expected long-term performance of the two mixtures.

Description of the Two Field Mixtures

Mixture A was a Superpave 9.5 mm nominal maximum aggregate size (NMAS) surface mix produced using Performance-Graded (PG) 64-22 binder, designated as a VDOT SM-9.5A mixture. Morelife 3300 antistrip was used at a dosage rate of 0.5% by weight of the binder. The aggregate was a mix of granite and siltstone. A summary of the mix properties is provided in Table 1. This was a typical SM-9.5A mixture used by the contractor; the only adjustment made to the mix design to produce WMA was the addition of Sasobit and the reduction of the production temperature. Sasobit was added at a dosage rate of 1.5% by weight of the binder. The binder content used for the WMA was the same as that used for the HMA.

Mixture B was a Superpave 12.5 mm NMAS surface mix produced using PG 64-22 binder, designated as a VDOT SM-12.5A mixture. Hydrated lime was used as an antistripping agent. The aggregate was a mix of limestone and gravel. A summary of the properties of this mix is provided in Table 1. Again, this was a typical SM-12.5A mixture used by the contractor. The mix design was the same as for the WMA and HMA mixtures except for the addition of 1.5% Sasobit by weight of the binder and the reduction of production temperatures.

The study included mixtures produced at the plant and produced in the laboratory. During construction of the two trial sections in 2006, HMA and WMA were sampled at each plant. In addition, for Mixture A, HMA was produced in the laboratory using the plant production temperatures and WMA was produced in the laboratory at temperatures of 230°F, 265°F, and 300°F. In all cases, the mixing and compaction temperatures were the same. For Mixture B, only plant-produced HMA and WMA were tested.

Table 1. Properties of Mixtures A and B

Properties	Mixture A	Mixture B
Mixture type	SM-9.5A	SM-12.5A
Binder type	PG 64-22	PG 64-22
Design gyrations	65	65
Cumulative percent passing		
¾ in (19.0 mm)	100	100
½ in (12.5 mm)	100	96
$^3/_8$ in (9.5 mm)	92	86
No. 4 (4.75 mm)	60	-
No. 8 (2.36 mm)	43	34
No. 200 (75 μm)	5.7	6.0
Aggregate type	Granite and siltstone	Limestone and gravel
Binder content	5.50%	5.20%
Antistripping agent	Morelife 3300, 0.5% by weight of asphalt	Hydrated lime, 1.0% by weight of asphalt
Recycled asphalt pavement	20%	10%
Sasobit	1.5% by weight of asphalt	1.5% by weight of asphalt
Production temperature	HMA, 300ºF; WMA, 250ºF	HMA, 325-330ºF; WMA, 300ºF

HMA = hot-mix asphalt, WMA = warm-mix asphalt.

Laboratory Testing

The testing matrix for the mixtures is presented in Table 2.

Table 2. Laboratory Testing Matrix for Mixtures A and B

<table>
<tr><td colspan="5">Laboratory-Mixed Material: Mixture A</td></tr>
<tr><td>Temperature</td><td>Control HMA,
300ºF</td><td>WMA,
300ºF</td><td>WMA,
265ºF</td><td>WMA,
230ºF</td></tr>
<tr><td>Asphalt content</td><td colspan="4">%AC from job-mix formula</td></tr>
<tr><td colspan="5">Volumetrics</td></tr>
<tr><td>No. of gyrations</td><td colspan="4">Design, 65 gyrations</td></tr>
<tr><td colspan="5">Moisture susceptibility</td></tr>
<tr><td>TSR</td><td colspan="4">Aging states: none, short term, long term[a]</td></tr>
<tr><td>Hamburg</td><td colspan="4">7% air voids; 122°F; wheel load, 158 lb</td></tr>
<tr><td colspan="5">Rutting</td></tr>
<tr><td>APA</td><td colspan="4">7% air voids; 147°F; hose pressure, 120 psi; wheel load, 120 lb</td></tr>
<tr><td colspan="5">Fatigue</td></tr>
<tr><td>Flexural beams</td><td colspan="4">Test to 30% stiffness; 10 Hz haversine load; 68°F</td></tr>
<tr><td colspan="5">Plant-Mixed Material: Mixtures A and B</td></tr>
<tr><td>Temperature</td><td colspan="2">Control HMA
300ºF: Mixture A
330ºF: Mixture B</td><td colspan="2">WMA
250ºF: Mixture A
300ºF: Mixture B</td></tr>
<tr><td colspan="5">Volumetrics</td></tr>
<tr><td>No. of gyrations</td><td colspan="4">Design, 65 gyrations</td></tr>
<tr><td colspan="5">Moisture susceptibility</td></tr>
<tr><td>TSR</td><td colspan="4">Modified AASHTO T283</td></tr>
<tr><td>Hamburg</td><td colspan="4">7% air voids; 122°F; wheel load,158 lb</td></tr>
<tr><td colspan="5">Rutting</td></tr>
<tr><td>APA</td><td colspan="4">7% air voids; 147°F; hose pressure, 120 psi; wheel load, 120 lb</td></tr>
<tr><td colspan="5">Fatigue</td></tr>
<tr><td>Flexural beams</td><td colspan="4">Test to 30% stiffness; 10 Hz haversine load; 68°F</td></tr>
</table>

HMA = hot-mix asphalt, WMA = warm-mix asphalt, TSR = tensile strength ratio, APA = Asphalt Pavement Analyzer.

[a]Short-term aging: loose mix was aged for 4 days in a forced draft oven at 185ºF. Long-term aging: loose mix was aged for 8 days in a forced draft oven at 185ºF (Bell et al., 1994).

Volumetric Properties

The following volumetric properties were determined:

- Percent air voids, V_a
- Voids in mineral aggregate, VMA
- Voids filled with asphalt, VFA
- Percent absorbed binder, P_{ba}
- Percent effective binder, P_{be}
- Effective film thickness, F_{be}
- Percent density at the initial number of gyrations, N_{ini}.

Samples were compacted in the Superpave Gyratory Compactor in accordance with AASHTO T312, Preparing and Determining the Density of Hot-Mix Asphalt (HMA) Specimens by Means of the Superpave Gyratory Compactor. The specific gravity was determined in accordance with AASHTO T166, Bulk Specific Gravity of Compacted Hot-Mix Asphalt Using Saturated Surface Dry Specimens; and the binder content and gradation were determined in accordance with AASHTO T308, Determining the Asphalt Binder Content of Hot-Mix Asphalt by the Ignition Oven, and AASHTO T30, Mechanical Analysis of Extracted Aggregate (AASHTO, 2007). Plant-produced material was sampled from the truck. One set of specimens was compacted in the field (i.e., without reheating), and one set was reheated in the laboratory and compacted. The laboratory-produced specimens were heated, mixed, and compacted in the laboratory. The volumetric properties were measured and evaluated to determine the potential for deterioration.

Moisture Susceptibility

The ability of the mixes to resist moisture damage was measured using two tests: the tensile strength ratio (TSR) and the Hamburg wheel-track tests. Each test measures moisture susceptibility differently. The TSR test compares conditioned and unconditioned indirect tensile strengths of specimen sets. The Hamburg wheel-track test subjects submerged samples to a simulated traffic load based on the theory that moisture damage causes rutting. Both plant-produced and laboratory-produced mixtures were evaluated using these two tests.

In addition, mixtures were produced in the laboratory using different production temperatures and aging treatments to evaluate the effects of temperature and aging on moisture susceptibility. Mixtures were also produced with entrapped moisture to evaluate the effects of insufficient aggregate drying during production.

Tensile Strength Ratio Test

The TSR was measured in accordance with a modified version of AASHTO T283, Resistance of Compacted Hot Mix Asphalt (HMA) to Moisture-Induced Damage (AASHTO, 2007). The Virginia modification waives the 16-hour curing time and 24-hour storage time; the remainder of the procedure is followed; the specimens are subjected to vacuum saturation before undergoing one freeze-thaw cycle (VDOT, 2002). The minimum TSR value accepted in Virginia is 0.80. A TSR value less than 0.80 implies that the mix may be susceptible to moisture damage.

Hamburg Wheel-Track Test

The Hamburg wheel-track test, AASHTO T324, Hamburg Wheel-Track Testing of Compacted Hot-Mix Asphalt (HMA) (AASHTO, 2007), was performed in a modified Asphalt Pavement Analyzer (APA) using gyratory-compacted specimens, as shown in Figure 1. Measurements were taken at three points along a pair of gyratory pills and averaged to provide a specimen measurement; samples were composed of three specimen sets (at the left, center, and right locations shown in Figure 1). Samples were submerged in water at 122°F while a 158-lb load was applied with a 1.85-in-wide wheel. The test was considered to be complete at 20,000 passes or a displacement of 1.575 in whichever occurred first.

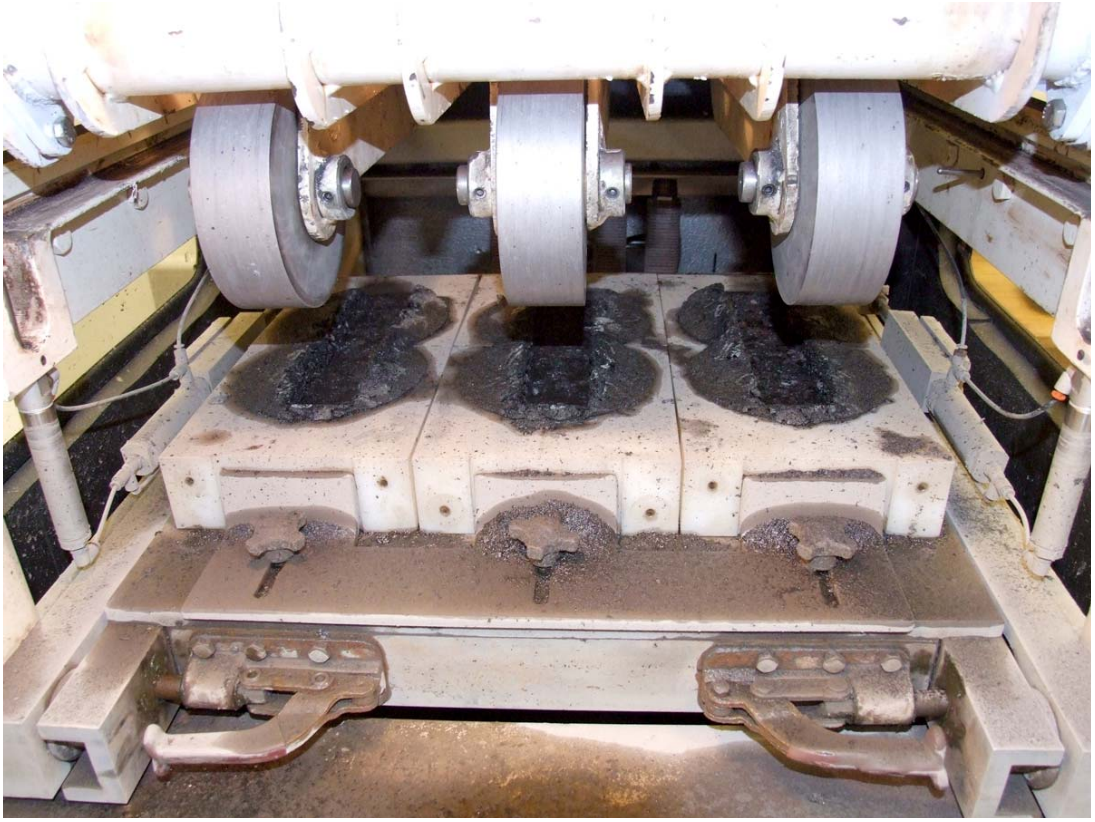

Figure 1. Setup for Hamburg Wheel-Track Test (After Draining)

Analysis of the data may be approached in two ways. The first is simply to determine the maximum rut depth. The Colorado Department of Transportation (DOT) specifies a maximum rut depth after 20,000 passes of 10 mm (Federal Highway Administration [FHWA], 2006b). A more rigorous analysis of the Hamburg wheel-track test data requires a plot of rut depth versus number of wheel passes, as illustrated in Figure 2. The displacement occurring after the first

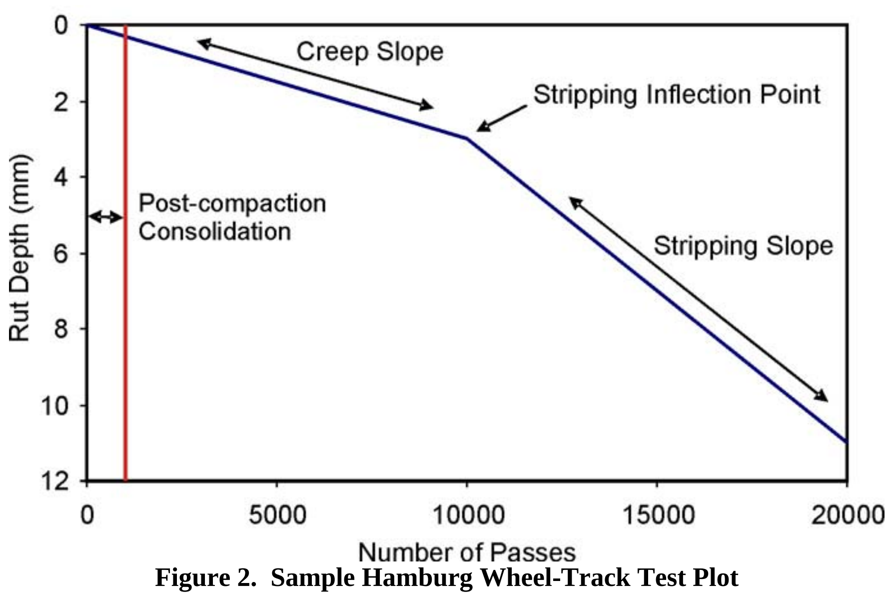

Figure 2. Sample Hamburg Wheel-Track Test Plot

1,000 cycles is called the post-compaction consolidation; most mixes will undergo densification during the first approximately 1,000 cycles. The next part of the curve is the creep slope; it primarily represents rutting attributable to plastic flow (Izzo and Tahmoressi, 1999). The final part of the curve is the stripping slope and is where rutting is occurring because of moisture damage. The intersection of the two slopes is where moisture damage begins to occur, and it is called the stripping inflection point. The Colorado DOT has found that mixes with a stripping inflection point of less than 10,000 passes are susceptible to moisture damage (FHWA, 2006b). This study followed the acceptance criterion of the Colorado DOT because VDOT has not established its own criterion.

Evaluation of Effects of Production Temperature and Aging Time

It was hypothesized by the researchers that WMA may continue to increase in tensile strength over time and that production temperature may have an influence on tensile strength. Thus, several additional sets of TSR specimens were produced in the laboratory and tested to investigate the influence of variables such as aging and temperature. To investigate aging, specimen sets were subjected to short-term and long-term aging before being tested. Short-term and long-term aging consisted of holding the compacted specimens in a forced draft oven at a temperature of 185ºF for 4 days and 8 days, respectively, prior to testing them in accordance with the modified AASHTO T283 procedure. To evaluate the effects of production temperature, WMA specimen sets were produced at temperatures of 230°F, 265°F, and 300°F and compared to HMA sets produced at 300°F. These specimen sets were produced with and without the addition of a liquid antistripping agent.

The TSR of the specimens was then determined using the TSR procedure previously discussed. The effects of production temperature and aging time were considered by comparing TSR values and analyzing Hamburg wheel-track test results.

Evaluation of Effects of Trapped Moisture

One concern regarding WMA is that the moisture from the stockpiles does not completely evaporate in the drum since mixing temperatures are lower than those used during typical HMA production. Typically, excess moisture evaporates as the aggregate travels through the drum in a drum plant or the aggregate dryer in a batch plant. If the plants run at lower temperatures, the moisture may not evaporate. The concern is that if the coated aggregates still contain moisture, stripping may occur prematurely. To test this theory in the laboratory, a "moist mixture" was produced. A procedure for moist mixing was developed based on a study by Huber et al. (2002).

The aggregate for batching was separated into the fraction retained on the No. 100 sieve (coarse material), the fraction passing the No. 100 sieve (fine material), and the recycled asphalt pavement (RAP). The coarse material was saturated with 20% moisture, covered to prevent evaporation, and allowed to soak for 24 hours at room temperature. While still covered, it was then placed in an oven at 300ºF for approximately 2 hours. This temperature was used because the ovens were already at that temperature for conventional HMA production. The coarse aggregate was then placed in a bucket mixer and heated while mixing until all free moisture in the mixer had evaporated. The fine portion and RAP were added, and the combined aggregate was heated until a small number of particles became surface-dry. The binder was then added, and mixing was continued. Heat was applied sparingly during mixing, only enough to facilitate coating. Temperature was not monitored during mixing; a visual inspection of the aggregate moisture and coating was used as a guide for the timing. As soon as the aggregate was coated, the mixing was stopped. The mix was usually at a temperature of around 190ºF when complete coating was achieved. The sample was then split into four portions: one portion was retained for moisture content determination, and three portions were compacted. The three compaction portions were compacted immediately in the gyratory compactor. The moisture sample was dried in a forced draft oven to constant mass, and the moisture loss was determined.

The TSR of the specimens was then determined using the TSR test previously discussed. The difference in the moisture susceptibility of mixtures with entrapped moisture was determined by comparing TSR values.

Rutting Potential

Testing was performed on gyratory samples compacted from plant-produced material using the APA to evaluate the rutting resistance of each mixture. This study employed the method typically used in Virginia to evaluate mixture rutting potential: Virginia Test Method 110, Method of Test for Determining Rutting Susceptibility Using the Asphalt Pavement Analyzer (VDOT, 2007). The test method allows the use of 6-in-diameter specimens that are compacted in a gyratory compactor to 8% air voids, although acceptance criteria for these specimens are not provided. A 120-lb load is applied at 120 psi. The test temperature is 122°F. The resulting rut depth is measured both automatically and by hand. Automatic average values are automatically detected and collected by the APA, and hand average values are calculated from measurements of rutting taken before testing and after the completion of 8,000 cycles of loading.

The acceptance criteria for rutting susceptibility using the APA in Virginia are presented in Table 3. It should be noted that the criteria are for beam specimens, not gyratory specimens. Gyratory specimens were used for this testing to allow specimens to be compacted at the contractors' laboratories. However, since specific criteria for gyratory specimens have not been developed, the existing requirements were used for comparative purposes. The maximum rut depth is an average of six specimens.

Table 3. Virginia Test Method 110 Acceptance Criteria for APA Beam Specimens

Traffic, ESALs	Mix Type	Maximum Rut Depth, mm
<3 million	Surface mix with PG 64-22	7
3-10 million	Surface mix with PG 70-22	5.5
>10 million	Surface mix with PG 76-22	3.5

Information in table from VDOT, 2007.
ESAL = equivalent single-axle load.

Fatigue Resistance

The fatigue resistance of Mixtures A and B was evaluated in accordance with AASHTO T321, Determining the Fatigue Life of Compacted Hot Mix Asphalt (HMA) Subjected to Repeated Flexural Bending, also known as the third point flexural fatigue test. Specimens were compacted in a vibratory compactor and then cut to the required dimensions. The target air voids were an average of the air void contents found between the control and Sasobit sections in each field trial; the tolerance was ±0.5%. A haversine load with a frequency of 10 Hz was used to apply constant strain conditions. Fatigue specimens were tested at strains of 300, 400, and 600 $\mu\varepsilon$. Some specimens were tested at 800 $\mu\varepsilon$. At least two beams were tested at each strain level.

Two methods were used to determine the predicted cycles to failure, N_f. The first follows the AASHTO test method and determines the cycles to failure, N_f, at 50% of the initial stiffness. The second uses the normalized modulus concept (Rowe and Bouldin, 2000). The normalized modulus is calculated as follows:

$$NM = \frac{S_i \times N_i}{S_o \times 50} \qquad \text{[Eq. 1]}$$

where

NM = normalized modulus × cycles, Pa/Pa
S_i = beam flexural stiffness at cycle i, Pa
N_i = cycle i
S_o = initial flexural stiffness estimated at approximately 50 cycles, Pa.

The normalized modulus is then plotted against the number of cycles to failure as illustrated in Figure 3. The cycles to failure, N_f, is taken at the maximum point of the curve.

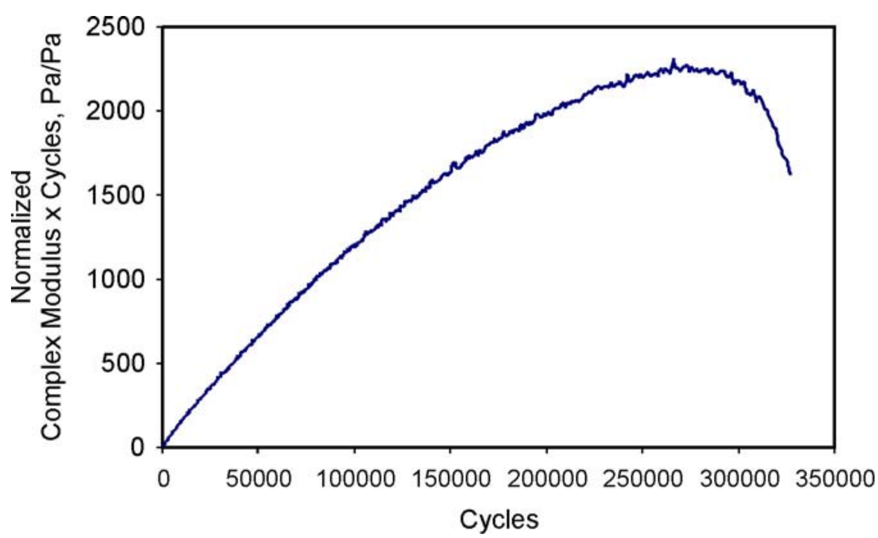

Figure 3. Example Plot of Normalized Complex Modulus Versus Cycles

Effects of Aging on the Binder

During the previous field study (Diefenderfer et al., 2007), cores were taken from the HMA and WMA sites at 3, 6, and 12 months after construction to evaluate the mixture performance over time. After other testing was performed on the cores, the binder was extracted in accordance with AASHTO T164, Standard Method of Test for Quantitative Extraction of Asphalt Binder from Hot-Mix Asphalt (HMA), Method A, and recovered using the Abson method of recovery, AASHTO T170, Standard Method of Test for Recovery of Asphalt from Solution by Abson Method (AASHTO, 2007). The recovered binder was then graded in accordance with AASHTO M320, Standard Specification for Performance-Graded Asphalt Binder (AASHTO, 2007). The effects of aging on the binder were evaluated using these results.

MEPDG Analysis to Provide Indication of Differences in Expected Long-Term Performance

To investigate the long-term differences between the performance of the HMA and the WMA, both trial sections were modeled in accordance with the *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*, or the Mechanistic-Empirical Design Guide (MEPDG) (NCHRP, 2004). The MEPDG takes into account the traffic loading, climatic effects, pavement structure, and material characteristics to evaluate a pavement for fatigue cracking, permanent deformation, and thermal cracking. First, a design life is determined along with serviceability and reliability limits. The traffic input includes average annual daily truck traffic (AADTT), number of lanes in the design direction, percent trucks in the design direction, and percent trucks in the design lane. Truck type distribution, seasonal traffic distribution, and traffic growths can all be specified. Climate files are available for weather stations around the United States; a groundwater depth must be specified. The structural inputs include layer thickness, layer material, and Poisson's ratio. For granular materials, a gradation must be specified. For asphalt mixtures, a gradation must also be identified in addition to effective binder content, air voids, and total unit weight.

There are three levels of input for material properties. Level 1 requires dynamic modulus data for the mix and dynamic shear rheometer (DSR) results for the binder. Level 2 uses gradation instead of dynamic modulus results for the mix but still requires DSR data. Level 3 employs gradation and binder grade. Where DSR results were available, this study used Level 2 input; otherwise Level 3 input was used. The underlying structure of the two test sections was determined and input into the MEPDG. Historic data provided information on the mixture types and traffic levels of the existing pavement. Finally, the binder data from the cores were input. The MEPDG uses mechanistic analysis and transfer functions to provide output that predicts field performance. Predicted cracking is reported in feet per mile and as a percentage of the pavement. Rutting is reported in inches, and the international roughness index (IRI) is reported in inches per mile. Finally, a reliability analysis was performed, and the results are presented as predicted reliability.

RESULTS AND DISCUSSION

Laboratory Testing

Volumetric Properties

Volumetric properties for Mixtures A and B are shown in Tables 4 and 5, respectively. *Plant* specimens refer to gyratory specimens that were compacted on site at the contractor's plant to eliminate differences in volumetrics or other properties that might be affected because of reheating. *Lab* specimens were made post-construction from loose mixture samples to evaluate the effects of reheating. Both plant and laboratory gyratory specimens were made for Mixture A HMA and WMA; however, for Mixture B, plant specimens were produced only for the WMA.

In the case of the plant-produced Mixture A WMA, there was a difference in total air voids between the plant-compacted and laboratory-compacted specimen. The air voids for the laboratory-produced Mixture A specimens (at 300°F) and plant-produced Mixture B specimens were similar.

Table 4. Mixture A Volumetric Results

	Plant-Produced Mix				Laboratory-Produced Mix			
	HMA		WMA		HMA	WMA		
Property	Plant[a]	Lab[b]	Plant	Lab	300°F	230°F	265°F	300°F
% AC	5.5	5.9	5.9	5.8	5.4	5.7	5.6	5.6
Rice SG (G_{mm})	2.504	2.501	2.498	2.502	2.508	2.507	2.508	2.509
% VTM	2.8	3.1	2.7	4.5	4.2	5.1	5.1	4.1
% VMA	14.7	15.7	15.3	16.8	15.9	17.3	17.1	16.2
% VFA	80.7	80.4	82.2	73.3	73.6	70.4	70.2	74.5
Bulk SG (G_{mb})	2.433	2.424	2.430	2.390	2.402	2.379	2.380	2.406
% Density @ N_{ini}	89.5	89.5	89.6	88.1	88.3	87.2	87.2	88.3
Sieve	**Percent Passing**							
¾ in (19.0 mm)	100	100	100	100	100	100	100	100
½ in (12.5 mm)	99.6	100	100	99.8	99.6	99.7	99.4	99.5
$^3/_8$ in (9.5 mm)	94.3	93.5	94.6	93.5	94.1	93.3	91.5	92.2
No. 4 (4.75 mm)	62.0	62.9	61.1	62.0	62.6	65.9	60.5	62.1
No. 8 (2.36 mm)	43.9	44.0	42.9	44.0	45.0	48.2	44.1	45.1
No. 200 (75 µm)	6.1	6.3	5.5	6.1	6.5	6.5	6.8	6.8

HMA = hot-mix asphalt, WMA = warm-mix asphalt, AC = asphalt content, SG = specific gravity, VTM = air void content, VMA = voids in mineral aggregate, VFA = voids filled with asphalt.
[a]*Plant* indicates gyratory specimens compacted at the plant during production.
[b]*Lab* indicates gyratory specimens compacted after construction from loose mixture samples.

Table 5. Mixture B Volumetric Results

Property	Plant-Produced Mix			
	HMA		WMA	
	Plant[a]	Lab[b]	Plant	Lab
% AC	No specimens compacted at plant	5.4	5.6	5.8
Rice SG (G_{mm})		2.604	2.571	2.597
% VTM		3.3	2.3	2.9
% VMA		15.8	15.3	16.5
% VFA		79.1	85.2	82.5
Bulk SG (G_{mb})		2.518	2.513	2.522
% Density @ N_{ini}		86.7	87.8	87.0
Sieve	**Percent Passing**			
¾ in (19.0 mm)	No specimens compacted at plant	100	100	100
½ in (12.5 mm)		95.8	97.3	97.0
$^3/_8$ in (9.5 mm)		84.1	84.1	85.2
No. 4 (4.75 mm)		48.3	49.9	51.0
No. 8 (2.36 mm)		32.7	33.0	33.4
No. 200 (75 µm)		6.5	5.9	6.3

HMA = hot-mix asphalt, WMA = warm-mix asphalt, AC = asphalt content, SG = specific gravity, VTM = air void content, VMA = voids in mineral aggregate, VFA = voids filled with asphalt.
[a]*Plant* indicates gyratory specimens compacted at the plant during production.
[b]*Lab* indicates gyratory specimens compacted after construction from loose mixture samples.

Moisture Susceptibility

Tensile Strength Ratio Test Results

The plant-compacted mixes generated mixed TSR results, as shown in Table 6. For Mixture A, the HMA complied with the 0.80 TSR specification, but the plant-compacted WMA did not. To investigate the failure further, a loose mix sample was taken back to the laboratory and reheated and compacted for additional testing. There was rain the day before production, so the stockpiles for Mixture A were wet. This may explain the low TSR and the improvement of the WMA after reheating, although it still did not comply with the 0.80 specification. Both materials for Mixture B performed well, having TSR values greater than 0.80.

In addition to the comparison of TSR values, tensile strengths (shown in Table 7) were considered for each mixture, with varied results. A set of *F*-tests and *t*-tests were conducted to evaluate the differences; these results are presented in Table 8. The *F*-tests were evaluated using a level of significance of $\alpha = 0.10$, and the *t*-tests were conducted using the appropriate variance assumptions and $\alpha = 0.05$. The Mixture A conditioned and unconditioned HMA strengths were significantly higher than the strengths of the WMA compacted in the field. However, despite the

Table 6. Tensile Strength Ratio for Plant-Produced Mixes

Mix	Mixture A	Mixture B
HMA, plant compacted	0.82	0.85
WMA, plant compacted	0.69	0.90
WMA, lab compacted[a]	0.75	-

HMA = hot-mix asphalt, WMA = warm-mix asphalt.
[a]Sample collected during production and reheated for compaction.

Table 7. Tensile Strengths for Plant-Produced Mix

Test Result	Mixture A	Mixture B
HMA, Plant Compacted		
Average conditioned strength, psi	124	186
Standard deviation	4	37
Average unconditioned strength, psi	151	219
Standard deviation	4	35
Average air voids, %	7.0	6.9
Standard deviation	0.06	2.45
WMA, Plant Compacted		
Average conditioned strength, psi	92	156
Standard deviation	2	6
Average unconditioned strength, psi	133	173
Standard deviation	4	16
Average air voids, %	6.8	4.1
Standard deviation	0.11	0.94
WMA, Lab Compacted [a]		
Average conditioned strength, psi	182	
Standard deviation	9	
Average unconditioned strength, psi	241	
Standard deviation	6	
Average air voids, %	6.5	
Standard deviation	0.16	
HMA = hot-mix asphalt, WMA = warm-mix asphalt.		

[a] Sample collected during production and reheated for compaction.

Table 8. Tensile Strength Comparisons of Plant-Produced Mixtures

Mix	Equivalent Variance (α = 0.10)	Equivalent Means (α = 0.05)
Mixture A		
Unconditioned, plant-compacted WMA vs. HMA	yes	no
Conditioned, plant-compacted WMA vs. HMA	yes	no
Unconditioned, lab-compacted WMA vs. HMA	yes	no
Conditioned, lab-compacted WMA vs. HMA	yes	no
Mixture B		
Unconditioned, WMA vs. HMA	yes	yes
Conditioned, WMA vs. HMA	yes	yes

WMA = warm-mix asphalt, HMA = hot-mix asphalt.

failing TSR value, the lab-compacted WMA strengths were significantly higher than the HMA strengths. Although the WMA in Mixture B had a higher TSR, the conditioned and unconditioned WMA strengths were not significantly different from the HMA strengths at the α = 0.05 level of significance.

Effects of Production Temperature and Aging Time

As discussed in the “Methods” section, it was hypothesized by the researchers that WMA may continue to increase in tensile strength over time and that production temperature may have an influence on tensile strength. Thus, several additional sets of TSR specimens were produced in the laboratory and tested to investigate the influence of aging and temperature.

TSR results were mixed, as evident in Table 9. The results for the HMA specimens were not improved after short-term aging or by the addition of an antistripping agent; however, the results were improved after long-term aging. The addition of an antistripping agent improved the results for all of the WMA samples. The long-term aging improved the results for the samples produced at 265ºF and 300ºF.

An analysis of variance (ANOVA) of the strength data (shown in Table 10) showed that both temperature and aging had an effect on the WMA unconditioned and conditioned strengths. The results of the ANOVA performed on the WMA are presented in Table 11. The strengths increased with increased aging as well as with increased temperature; in addition, increased aging and increased temperature reinforced each other in terms of increasing strength.

Table 9. Tensile Strength Ratio for Laboratory-Produced Mixtures, Mixture A

Specimen	HMA	WMA		
	300ºF	**300ºF**	**265ºF**	**230ºF**
Unaged specimens, no antistrip	0.78	0.82	0.72	0.48
4-day oven-aged specimens, no antistrip	0.66	0.83	0.90	0.84
8-day oven-aged specimens, no antistrip	0.80	0.93	0.92	0.74
Unaged specimens, with antistrip	0.76	0.86	0.92	0.72

HMA = hot-mix asphalt, WMA = warm-mix asphalt.

Table 10. Tensile Strengths for Laboratory-Produced Mixtures, Mixture A

Specimen	HMA	WMA		
	300ºF	**300ºF**	**265ºF**	**230ºF**
Unaged specimens, no antistrip				
Average conditioned strength, psi	148	161	134	82
Standard deviation	21	2	12	5
Average unconditioned strength, psi	190	196	187	169
Standard deviation	9	8	9	9
Average air voids, %	7.4	6.8	7.0	6.9
Standard deviation	0.12	0.12	0.09	0.05
4-day oven-aged specimens, no antistrip				
Average conditioned strength, psi	182	193	155	149
Standard deviation	16	13	3	3
Average unconditioned strength, psi	277	232	172	178
Standard deviation	37	12	7	3
Average air voids, %	7.0	7.1	7.0	6.6
Standard deviation	0.11	0.16	0.14	0.07
8-day oven-aged specimens, no antistrip				
Average conditioned strength, psi	223	244	185	159
Standard deviation	7	4	6	11
Average unconditioned strength, psi	280	263	202	215
Standard deviation	18	21	11	8
Average air voids, %	6.9	6.8	6.9	7.0
Standard deviation	0.07	0.17	0.11	0.14
Unaged specimens, with antistrip				
Average conditioned strength, psi	158	168	188	141
Standard deviation	12	7	20	7
Average unconditioned strength, psi	207	195	204	196
Standard deviation	8	5	23	7
Average air voids, %	7.0	6.8	7.1	6.7
Standard deviation	0.08	0.08	0.15	0.08

HMA = hot-mix asphalt, WMA = warm-mix asphalt.

Table 11. Summary of Two-Way Fixed ANOVA Performed on Laboratory-Mixed Tensile Strengths

Unconditioned Strengths					
Source	df	SS	MS	*F*	*p*
Temperature	2	15022.9	7511.4	65.00	0.000
Aging	2	12177.7	6088.9	52.69	0.000
Interaction	4	3479.9	870.0	7.53	0.000
Error	27	3120.0	115.6		
Total	35	33800.6			
Conditioned Strengths					
Source	df	SS	MS	*F*	*p*
Temperature	2	29450.1	14725.0	250.56	0.000
Aging	2	29867.6	14933.8	254.11	0.000
Interaction	4	3534.6	883.7	15.04	0.000
Error	27	1586.7	58.8		
Total	35	64439.0			

df = degrees of freedom, SS = sums of squares, MS = mean square, *F* = *F*-ratio, *p* = *p*-value.

A *t*-test with $\alpha = 0.05$ was used to examine further the conditioned and unconditioned strengths of the materials at 300ºF both with and without an antistripping agent. An *F*-test with a level of significance of $\alpha = 0.1$ was used to compare the variances; these results are presented in Table 12. There was no significant difference in any of the conditioned strength comparisons at the $\alpha = 0.05$ level of significance. There was a difference between the unconditioned strengths of the HMA with and without an antistripping agent. There was also a difference between the unconditioned strengths of the 300ºF WMA with an antistripping agent and the HMA with an antistripping agent. In both cases, the unconditioned strength of the HMA with an antistripping agent was greater.

The effects of aging on the samples provided a clearer trend. Figure 4 illustrates the tensile strengths of the WMA produced at 300ºF and the HMA. The graph shows the conditioned and unconditioned strengths of the material at initial, short-term, and long-term aging. All but the unconditioned HMA increased at similar rates over time.

Figures 5 and 6 illustrate the indirect tensile strengths of the laboratory-produced WMA. The strengths generally increased with aging. This was likely due to the stiffening effect that

Table 12. Additional Comparisons of Strengths of Lab-Produced Mix

Strength	**Equivalent Variance (α = 0.10)**	**Equivalent Means (α = 0.05)**
Conditioned Strengths		
300ºF WMA vs. HMA	no	yes
HMA vs. HMA with antistrip	yes	yes
300ºF WMA vs. 300ºF WMA with antistrip	no	yes
300ºF WMA with antistrip vs. HMA with antistrip	yes	yes
Unconditioned Strengths		
300ºF WMA vs. HMA	yes	yes
HMA vs. HMA with antistrip	yes	no
300ºF WMA vs. 300ºF WMA with antistrip	yes	yes
300ºF WMA with antistrip vs. HMA with antistrip	yes	no

HMA = hot-mix asphalt, WMA = warm-mix asphalt.

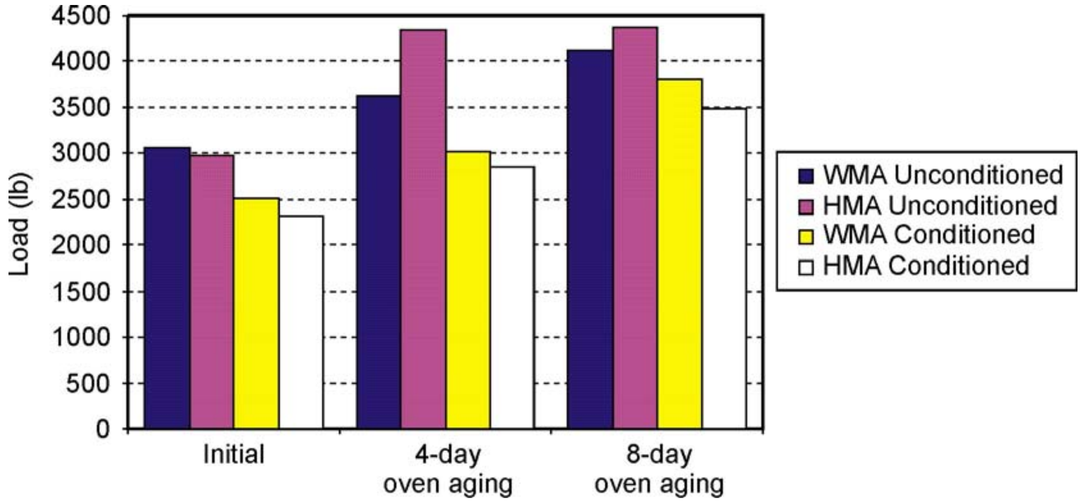

Figure 4. Indirect Tensile Strength of Mixture A at 300ºF. WMA = warm-mix asphalt, HMA = hot-mix asphalt.

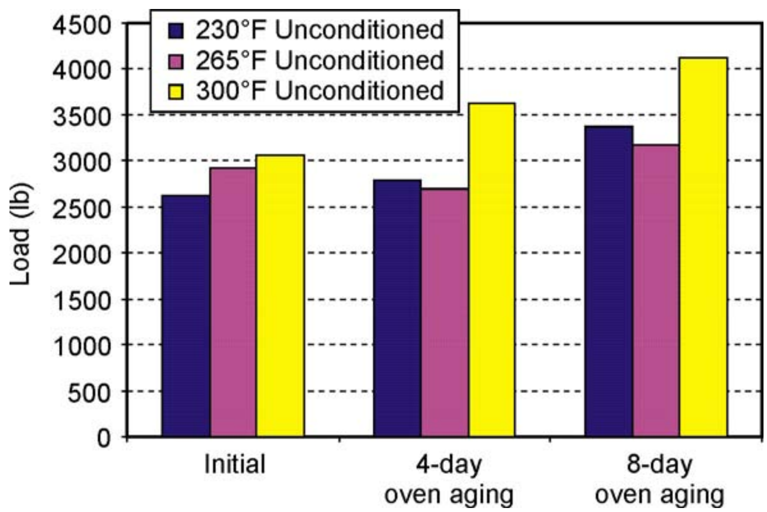

Figure 5. Indirect Tensile Strengths of Laboratory-Mixed Unconditioned Samples

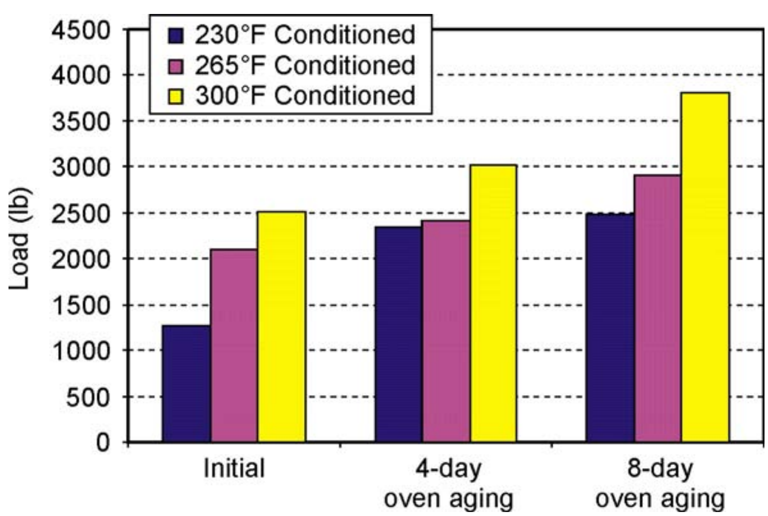

Figure 6. Indirect Tensile Strengths of Laboratory-Mixed Conditioned Samples

aging has on binder. The 300ºF samples generally had the highest unconditioned and conditioned strengths. Again, this was likely due to the fact that when binder is heated to greater temperatures it will undergo more aging, which will cause it to stiffen.

The performance of WMA, when compared to HMA, was similar except in the case of Mixture A where wet stockpiles may have been a factor. Overall, the WMA moisture susceptibility performance improved when the WMA was produced at higher temperatures and when aged.

Hamburg Wheel-Track Test Results

The results of the tests performed on the plant-produced mixtures are presented in Table 13. The maximum allowed deformation at 20,000 cycles is not specified in the AASHTO procedure, but a maximum of 10 mm after 20,000 cycles is specified by the Colorado DOT (FHWA, 2006b). The measured rut depths of the specimen sets for both trials were well below the 10 mm criterion. It can be seen in Figures 7 and 8 that the specimens underwent only plastic deformation and had not yet reached the stripping inflection point. The Hamburg wheel tracking device malfunctioned with the Mixture A HMA sample, and data for the first 9,000 passes were not recorded. Based on the depth of rutting after 20,000 passes, it can be assumed that this sample also underwent only plastic deformation. From these observations, it can be concluded that all mixes should be resistant to stripping.

Table 13. Summary of Hamburg Wheel-Track Test Results for Plant-Produced Mixtures

Mixture	Average Air Voids (%)	Standard Deviation (Air Voids)	Rut Depth at 20,000 Passes (mm)
Mixture A HMA	7.6	0.5	2.11
Mixture A WMA	7.8	0.2	2.13
Mixture B HMA	7.4	0.5	2.44
Mixture B WMA	7.1	0.3	2.07

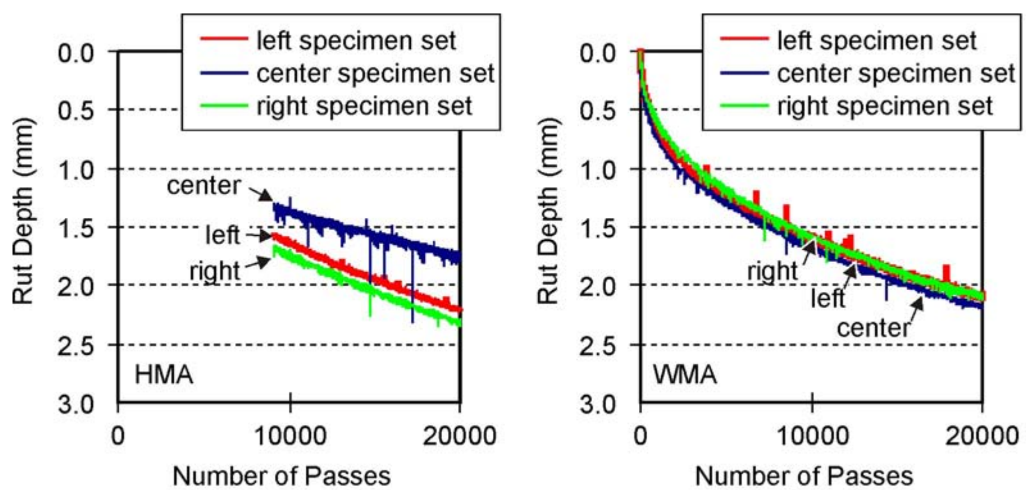

Figure 7. Hamburg Rut Depths for Mixture A

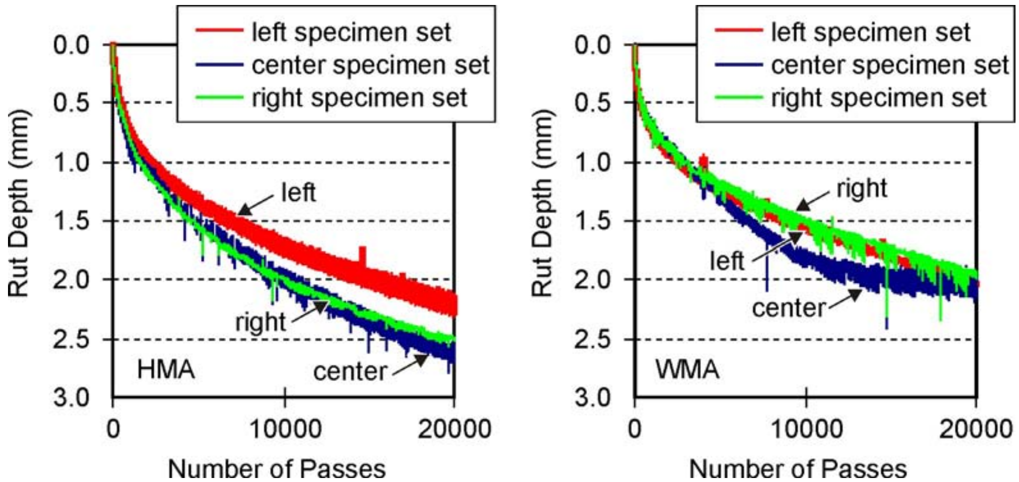

Figure 8. Hamburg Rut Depths for Mixture B

The laboratory-produced samples were also tested in the Hamburg wheel tracking device. A summary of the results is presented in Table 14. Figures 9 and 10 illustrate the development of rutting in the samples. The warm mix produced at 230ºF was the only sample to fail, and it failed only one criterion. The limit for maximum rut depth after 20,000 passes is 10 mm; the sample had a rut depth of 11.8 mm at approximately 15,000 cycles. However, the recommended criterion for the stripping inflection point is a minimum of 10,000 passes and the sample had an inflection point of 11,000 passes, as can be seen in Figure 9. The other sample that began to exhibit moisture damage was the unaged HMA, although the rut depth and stripping inflection point indicated that it was still resistant to moisture damage. The rest of the samples measured well below the 10 mm criterion, and all were still undergoing plastic deformation after 20,000 passes. As shown in Figure 10, the long-term oven aged HMA and WMA samples performed the best based on rut depths, followed by the WMA produced at 300ºF.

Table 14. Summary of Hamburg Wheel-Track Test Results for Laboratory Samples

Sample	Average Air Voids (%)	Standard Deviation	Rut Depth at 20,000 Passes (mm)	Stripping Inflection Point (passes)
WMA 230ºF	7.3	0.1	11.8[c]	11,000
WMA 265ºF	7.1	0.2	6.2	--[b]
WMA 300ºF	7.1	0.1	3.0	--[b]
HMA 300ºF	7.0	0.1	6.2	17,000
WMA 300ºF, long-term aging[a]	6.7	0.1	1.5	--[b]
HMA 300ºF, long-term aging[a]	7.1	0.1	2.2	--[b]

WMA =warm-mix asphalt, HMA = hot-mix asphalt.
[a] Loose mix was aged for 8 days at 185°F.
[b] Failed in plastic flow.
[c] Test halted after approximately 15,000 passes.

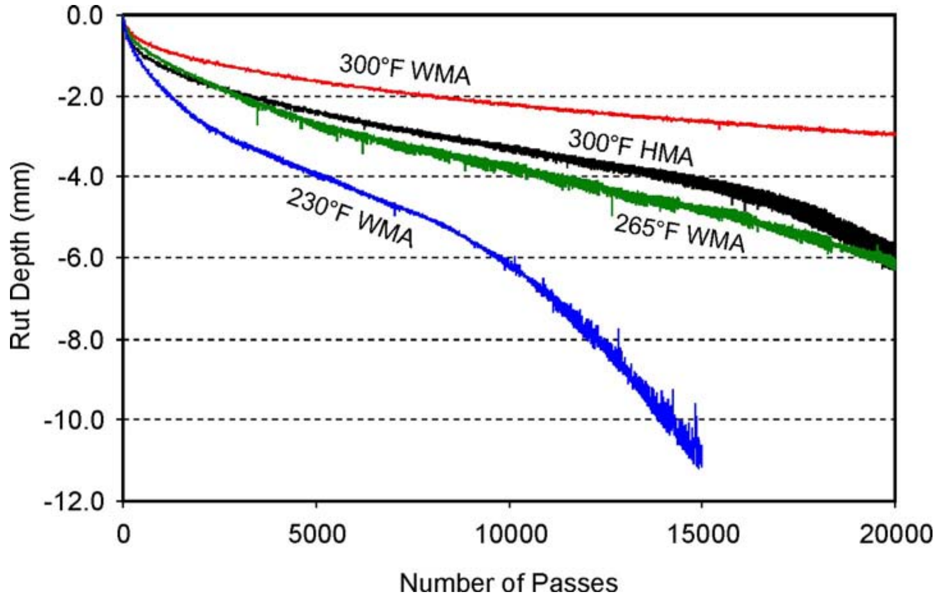

Figure 9. Hamburg Rut Depths for Laboratory-Produced Mixtures. WMA = warm-mix asphalt, HMA = hot-mix asphalt.

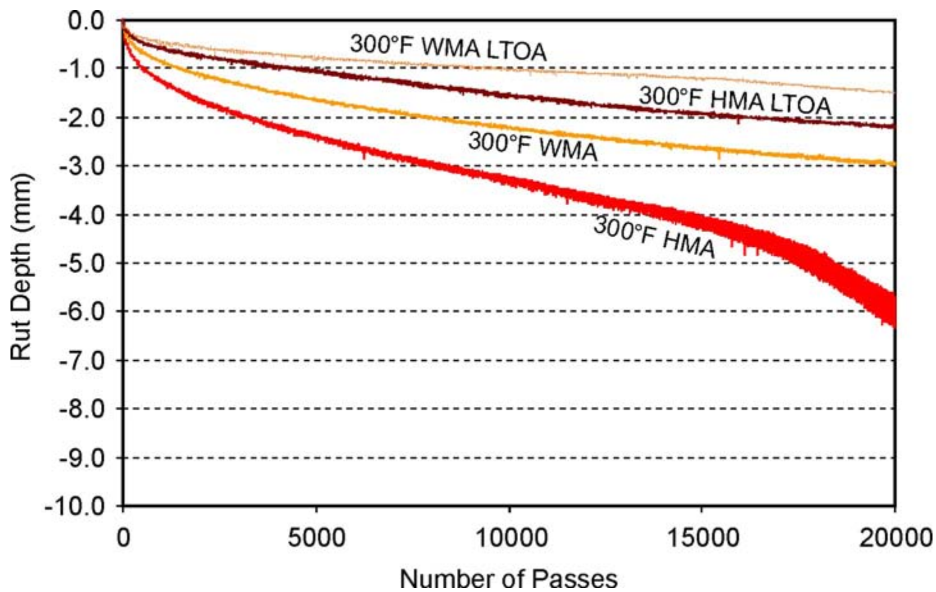

Figure 10. Effect of Long-Term Oven Aging (LTOA) on Hamburg Rut Depths. WMA = warm-mix asphalt, HMA = hot-mix asphalt.

Effects of Entrapped Moisture

The effect of wet stockpiles on mixture TSRs was simulated in the laboratory for Mixture A. The moist mixing procedure achieved an average mixture moisture content (determined immediately after mixing) of 0.5% for the unaged samples and 0.9% for the 4-day oven-aged samples. Table 15 presents the TSR results for the WMA moist mix.

The unconditioned strengths of the unaged material were slightly lower than those of the plant-produced mix; but the TSR value were much lower because of the difference in

Table 15. Mixture A Warm-Mix Asphalt (WMA): Wet Mix Tensile Strength Ratio

Specimen	Average Unconditioned Strength (psi)	Average Conditioned Strength (psi)	Tensile Strength Ratio	Average Moisture (%)
Unaged samples	114	57	0.50	0.5%
4 day oven aging	167	118	0.71	0.9%
Plant-produced WMA	133	92	0.69	Not measured

conditioned strengths. After 4 days of oven aging at 185°F, the strengths increased considerably. The TSR value also increased, but still did not comply with the 0.80 specification.

Neither of the mixtures that simulated moist stockpiles complied with the TSR specification, again indicating that the use of moist aggregate has a considerable influence on the potential for stripping.

Rutting Potential

Table 16 shows the average values for each sample. Both HMA and WMA specimen sets for Mixtures A and B were found to have acceptable rutting resistance; the specification limit for these mixtures when tested as beam specimens is 7.0 mm of rutting (VDOT, 2007). For Mixture A, the WMA specimen set was shown to have an average of more than 0.5 mm less rutting than the control mixture. This could be due to the stiffening influence of Sasobit at temperatures below the additive's melting point, which has been promoted as a benefit of the technology (Butz et al., 2001). However, this was not found statistically significant using the *t*-test. The results of the *t*-test are presented in Table 17. Again, an *F*-test was performed at the α = 0.10 level to determine whether the variances were equal. The *t*-test was then performed with

Table 16. Asphalt Pavement Analyzer Rut Measurements for Plant-Produced Mixtures A and B

Mix	HMA Laboratory Specimens			WMA Laboratory Specimens		
	Hand Average (mm)	Automated Average (mm)	Average Voids (%)	Hand Average (mm)	Automated Average (mm)	Average Voids (%)
Mixture A						
Average	5.23	4.39	7.0	4.32	3.81	7.0
Standard deviation	0.59	0.43	0.2	0.35	0.48	0.1
Mixture B						
Average	3.24	2.74	8.5	2.83	2.72	8.5
Standard deviation	0.04	0.11	0.1	0.53	0.25	0.3

HMA = hot-mix asphalt, WMA = warm-mix asphalt.

Table 17. *F*-test and *t*-test Results for Trial A and Trial B Rutting Measurements

Measurement	Equivalent Variance (α = 0.10)	Equivalent Means (α = 0.05)
Trial A		
Hand HMA vs. hand WMA	yes	yes
Automated HMA vs. automated WMA	yes	yes
Trial B		
Hand HMA vs. hand WMA	no	yes
Automated HMA vs. automated WMA	yes	yes

HMA = hot-mix asphalt, WMA = warm-mix asphalt.

the appropriate variance assumption at a significance level of $\alpha = 0.05$. Similarly, the Mixture B WMA specimens experienced less rutting than the HMA samples, but the *t*-test indicated that the rutting resistance values were not significantly different.

In addition to the plant-produced testing, WMA was produced in the laboratory at different temperatures and tested in the APA along with HMA produced at 300ºF. The average hand measurement, average automated measurement, and average air voids are presented in Table 18. Again, all samples complied with the 7 mm requirement. A series of *t*-tests was performed to assess the difference in mixes, and these results are presented in Table 19. A level of significance of $\alpha = 0.05$ was used, and equal variances were assumed after *F*-tests were performed on all data sets using a $\alpha = 0.1$. The difference in rutting between the 230°F WMA and 300ºF WMA was significant; all other WMA comparisons showed the rutting to be the same. The WMA produced at 300ºF rutted significantly less than the HMA. The rut depths for the 230°F WMA and HMA were equal.

Based on this analysis, it can be concluded that the rutting potential of WMA decreases with increasing production temperatures. The rutting potential of WMA is equal to or, in some cases, less than that of HMA.

Table 18. Asphalt Pavement Analyzer Rut Measurements for Mixture A Laboratory-Produced Samples

Mix	Hand Average (mm)	Standard Deviation	Automated Average (mm)	Standard Deviation	Air Voids (%)	Standard Deviation
WMA 230°F	2.39	0.70	2.38	0.29	7.9	0.1
WMA 265°F	1.37	0.37	1.77	0.25	7.4	0.3
WMA 300°F	1.11	0.47	1.38	0.22	7.2	0.4
HMA 300°F	2.12	0.48	2.27	0.28	7.6	0.2

WMA = warm-mix asphalt, HMA = hot-mix asphalt.

Table 19. *F*-tests and *t*-tests of Asphalt Pavement Analyzer Results for Laboratory-Produced Mix

Measurement	Equivalent Variance ($\alpha = 0.10$)	Equivalent Means ($\alpha = 0.05$)
Automated		
230°F WMA vs. 265°F WMA	Yes	Yes
230°F WMA vs. 300°F WMA	Yes	No
265°F WMA vs. 300°F WMA	Yes	Yes
300°F WMA vs. 300°F HMA	Yes	No
265°F WMA vs. 300°F HMA	Yes	Yes
230°F WMA vs. 300°F HMA	Yes	Yes
Hand		
230°F WMA vs. 265°F WMA	Yes	No
230°F WMA vs. 300°F WMA	Yes	No
265°F WMA vs. 300°F WMA	No	Yes
300°F WMA vs. 300°F HMA	Yes	No
265°F WMA vs. 300°F HMA	Yes	Yes
230°F WMA vs. 300°F HMA	Yes	No

WMA = warm-mix asphalt, HMA = hot-mix asphalt.

Fatigue Resistance

Figure 11 shows the fatigue analysis of Trial A using the 50% initial stiffness method. The WMA had a slightly lower fatigue resistance at the lower strain levels, but as the strain increased, the performance of the WMA matched that of the HMA. When plotted using the normalized modulus method, as shown in Figure 12, however, the WMA appeared to perform

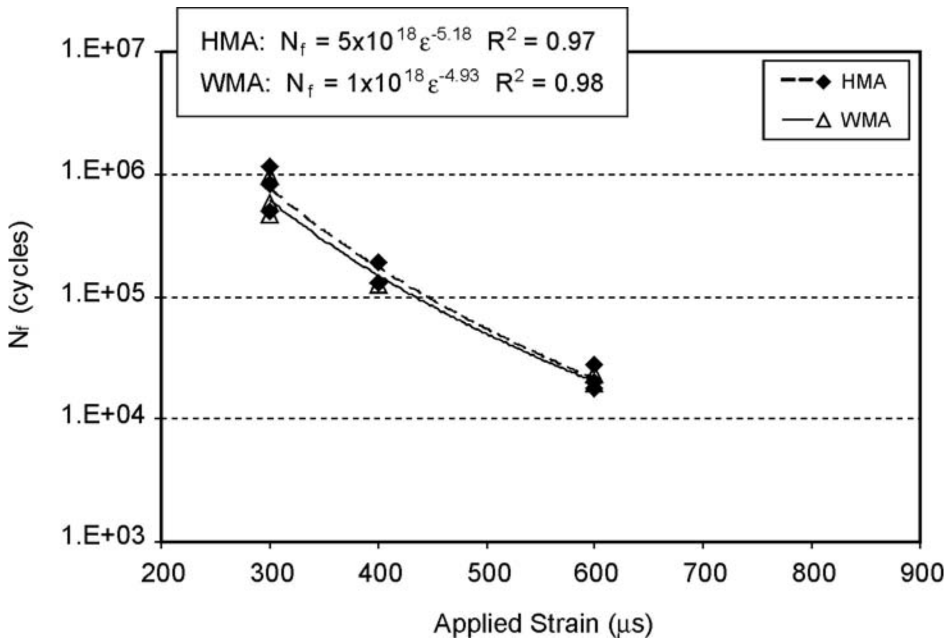

Figure 11. Fatigue Results for Mixture A Using 50% Initial Flexural Stiffness. HMA = hot-mix asphalt, WMA = warm-mix asphalt.

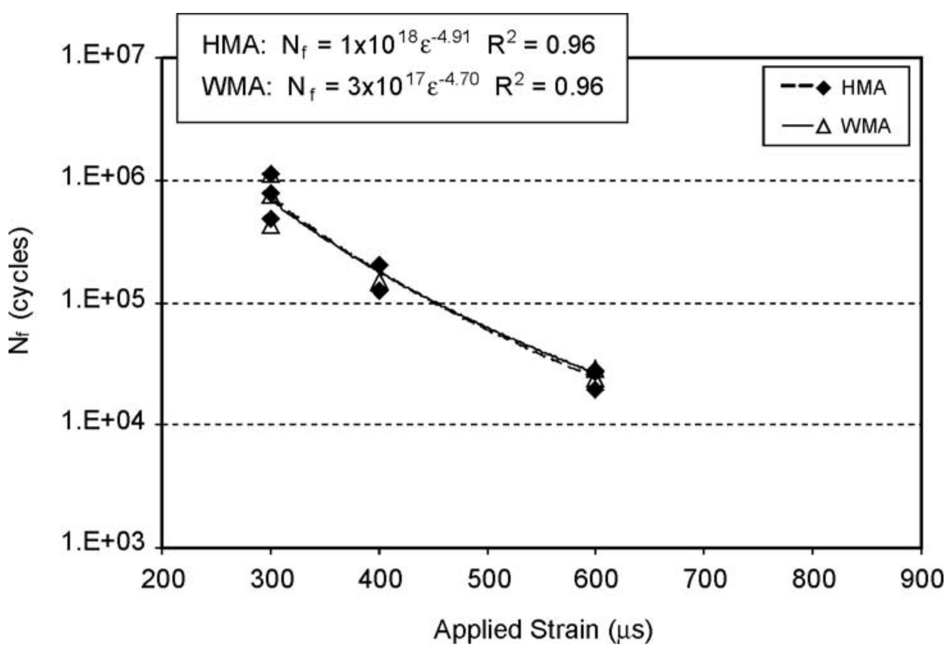

Figure 12. Fatigue Results for Mixture A Using Normalized Modulus. HMA = hot-mix asphalt, WMA = warm-mix asphalt.

slightly better than the HMA at all strain levels. The difference in performance in both cases was minimal. Another distinction to note is the lower R^2 value for the normalized modulus method, indicating that this method of analysis does not fit the measured fatigue values as well as the 50% initial stiffness method.

In the case of Trial B, the WMA and HMA performed similarly when plotted using the 50% initial stiffness method, as seen in Figure 13. When plotted using the normalized modulus method, shown in Figure 14, the HMA performed better at lower strains and the WMA

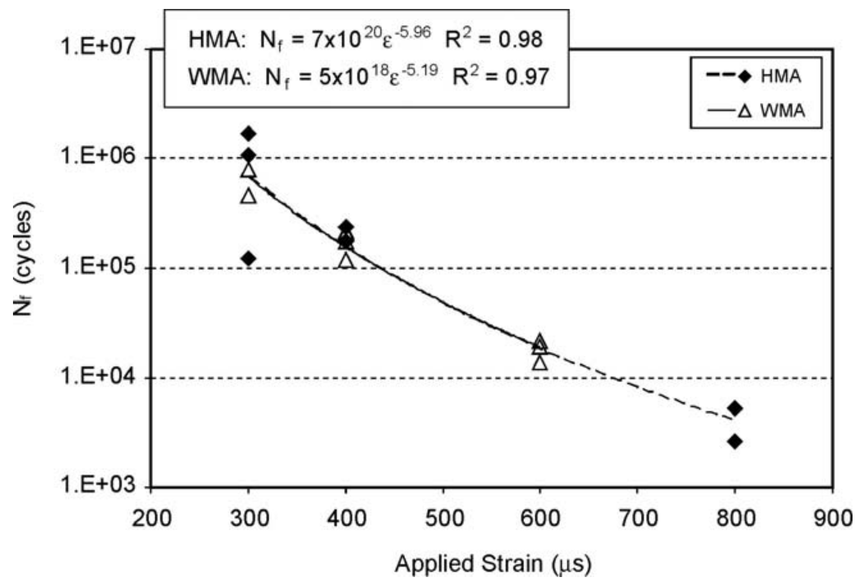

Figure 13. Fatigue Results for Mixture B Using 50% Initial Flexural Stiffness. HMA = hot-mix asphalt, WMA = warm-mix asphalt.

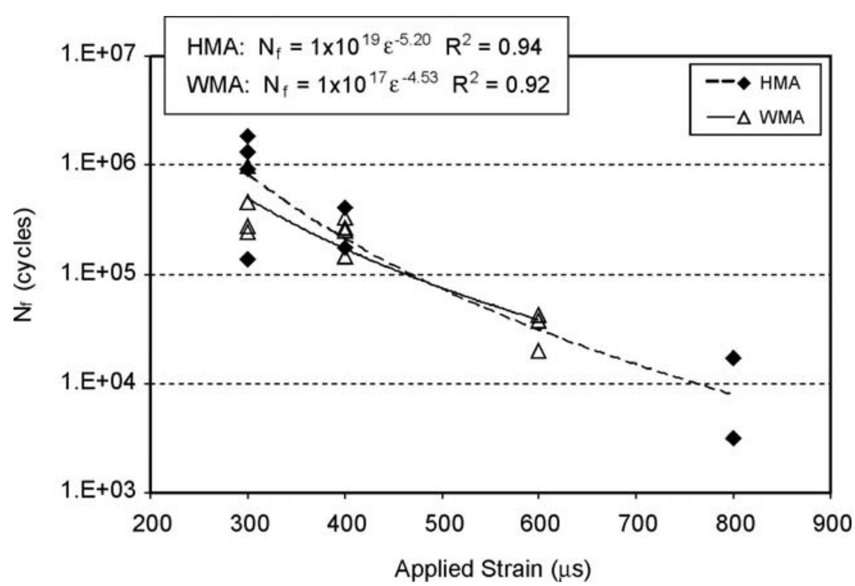

Figure 14. Fatigue Results for Mixture B Using Normalized Modulus. HMA = hot-mix asphalt, WMA = warm-mix asphalt.

performed better at higher strains. Again, these differences were not considerable, and the difference in results between the two methods of determining N_f support the conclusion that the plant-produced WMA and HMA performed similarly.

A similar response was seen with the laboratory-produced mix. The fatigue plots for the laboratory-produced mix are presented in Figures 15 and 16. Using the 50% initial flexural stiffness method, presented in Figure 15, the results show that the HMA performed slightly better than the WMA. In addition, the WMA produced at 300°F performed better than the WMA

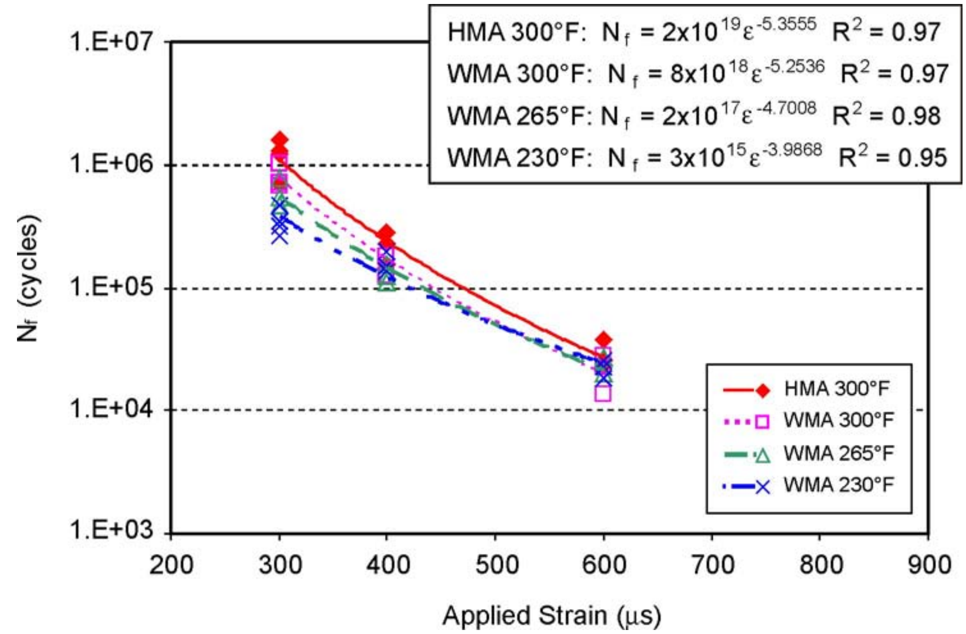

Figure 15. Fatigue Results for Lab-Produced Mixtures Using 50% Initial Flexural Stiffness. HMA = hot-mix asphalt, WMA = warm-mix asphalt.

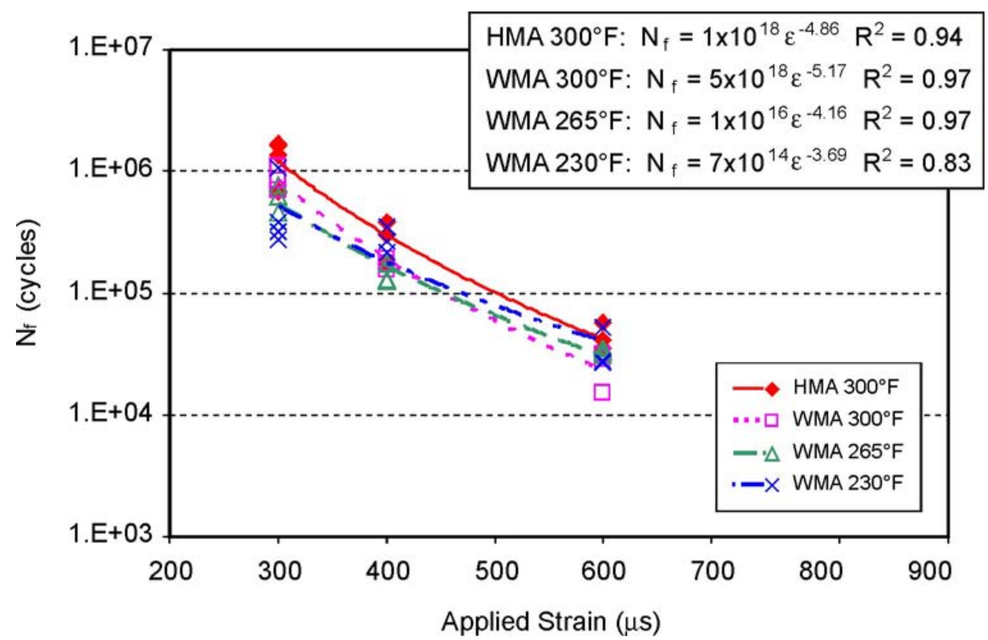

Figure 16. Fatigue Results for Laboratory-Produced Mix Using Normalized Modulus. HMA = hot-mix asphalt, WMA = warm-mix asphalt.

produced at 265°F and the WMA produced at 230°F performed the worst at low strain levels. As the strain levels increased, however, the difference in fatigue resistance became less for all samples.

A similar trend was seen with the normalized modulus plot presented in Figure 16. The HMA performed the best at all strain levels, although the difference became less as the applied strain increased. The 300°F WMA performed the best at a low strain level and the worst at a high strain level. The difference between the 265°F WMA and 230°F WMA remained constant with strain; the 230°F WMA outperformed the 265°F WMA. Again, the normalized modulus method yielded lower R^2 values than the 50% initial stiffness method.

WMA appeared to undergo more fatigue damage than HMA at lower strain levels, but as the strain increased, the performance of WMA and HMA became more similar. This trend was seen for both the plant-produced and lab-produced mixtures. There is not a straightforward application of flexural fatigue results to in-situ pavement life prediction; however, the general implications of these results may be that the preferred locations for WMA use are those that have a higher traffic loading.

Effects of Aging on the Binder

Data from the cores taken at 3 months are not presented as those cores were mistakenly used for other testing and destroyed.

The results for Mixture A are presented in Table 20. Comparing the HMA results to those of the virgin binder, it can be seen that the inclusion of RAP in the mix increased the HMA

Table 20. Performance-Graded Binder Data for Mixture A Cores

Test	Virgin Binder	HMA			WMA		
		Initial	**6 Mo**	**1 Yr**	**Initial**	**6 Mo**	**1 Yr**
Dynamic Shear on RTFO-aged binder							
G*/sinδ, 64°C	4.817				12.88		
G*/sinδ, 70°C	2.226	3.98	4.925		5.886	4.834	6.657
G*/sinδ, 76°C		1.914	2.376	5.461	2.778	2.293	3.115
G*/sinδ, 82°C				2.769			1.524
G*/sinδ, 88°C				1.240			
Dynamic Shear on PAV-aged binder							
G*sinδ, 22°C	4520						8260
G*sinδ, 25°C	3028	4952	4409	5011	4893	4255	5796
G*sinδ, 28°C		3428	3032	3616	3445	2968	4128
G*sinδ, 31°C					2379		2819
Creep Stiffness, 60 sec							
S, -6°C							150
M, -6°C							0.347
S, -12°C	210	235	235	252	244	229	308
M, -12°C	0.339	0.332	0.320	0.315	0.310	0.302	0.299
S, -18°C				494			
M, -18°C				0.277			
Binder Grade	PG 64-22	PG 70-22	PG 76-22	PG 82-22	PG 76-22	PG 76-22	PG 76-16

HMA = hot-mix asphalt, WMA = warm-mix asphalt, RTFO = rolling thin film oven, PAV = pressure aging vessel.

binder grade by one high temperature grade, from PG 64-22 to PG 70-22. The inclusion of Sasobit and RAP in the WMA increased the binder grade one high temperature grade over the HMA and two high temperature grades over the virgin binder. Over time, the HMA binder continued to stiffen, reaching a grade of PG 82-22 after 1 year of service. The WMA maintained a constant high temperature grade during the first year of service; however, the low temperature grade increased by one grade between 6 months and 1 year.

The Mixture B results are shown in Table 21. Both the addition of the RAP and the addition of Sasobit and RAP resulted in an increase in the binder grade at both the high and low temperature grades. The HMA maintained a performance grade of PG 70-16 through 6 months, although the high-temperature grade increased during the period from 6 months to 1 year. The WMA binder grade remained constant through 1 year.

Results from the recovered binder analysis indicated only one high temperature grade difference between the WMA and HMA after 1 year of service for both mixtures. One low temperature grade difference was seen for Mixture A but was not evident for Mixture B. Comparisons with the virgin binder grade indicated that the inclusion of RAP or of RAP and Sasobit should be expected to increase the binder high temperature grade; experimentation was not performed to determine the isolated influence of Sasobit.

Table 21. Performance-Graded Binder Data for Mixture B Cores

Test	Virgin Binder	HMA			WMA		
		Initial	6 Mo	1 Yr	Initial	6 Mo	1 Yr
Dynamic Shear on RTFO-aged Binder							
G*/sinδ, 64°C	3.560						
G*/sinδ, 70°C	1.652	3.257	3.594	5.627	3.405	4.032	4.58
G*/sinδ, 76°C		1.567	1.71	2.668	1.569	1.951	2.121
G*/sinδ, 82°C				1.304			
Dynamic Shear on PAV-aged Binder							
G*sinδ, 22°C	4325	7097			6051		
G*sinδ, 25°C	2959	4909	3871	7015	4256	5292	5862
G*sinδ, 28°C			2652	4876		3659	4103
Creep Stiffness, 60 sec							
S, -6°C				153		136	132
M, -6°C				0.343		0.343	0.321
S, -12°C	178	225	187	304	246	269	281
M, -12°C	0.304	0.285	0.281	0.293	0.268	0.29	0.28
Binder grade	PG 64-22	PG 70-16	PG 70-16	PG 76-16	PG 70-16	PG 70-16	PG 70-16

HMA = hot-mix asphalt, WMA = warm-mix asphalt, RTFO = rolling thin film oven, PAV = pressure aging vessel.

MEPDG Analysis to Provide Indication of Differences in Expected Long-Term Performance

As discussed in the “Methods” section, the MEPDG provides a tool to analyze each test section for long-term performance. The MEPDG uses traffic, climate, pavement structure, and pavement material input; calculated distress, and used transfer functions to predict performance. The field sections containing Mixtures A and B were both modeled using the MEPDG.

Recovered binder data from the initial, 6-month, and 1-year cores were used to investigate the impact of aging. To exaggerate the performance results, each section was modeled in an extremely hot climate, an extremely cold climate, and under extreme traffic loads. A matrix of the MEPDG evaluations performed is presented in Table 22.

Table 22. Testing Matrix for MEPDG Evaluation

Factors	Mixture A		Mixture B	
	HMA	WMA	HMA	WMA
Initial binder data	x	x	x	x
6 mo binder data	x	x	x	x
1 yr binder data		x	x	x
Hot climate, initial binder data	x	x	x	x
Cold climate, initial binder data	x	x	x	x
High traffic load, initial binder data	x	x	x	x

MEPDG = Mechanistic Empirical Pavement Design Guide, HMA = hot-mix asphalt, WMA = warm-mix asphalt.

MEPDG Input

As discussed in the "Methods" section, the MEPDG used a comprehensive set of input values to calculate response. This study used Level 3 input in all cases except the surface layer where recovered binder data were available. The sections for Mixtures A and B each used unique models since the pavement structure, location, and traffic loads were different. The input values were held constant for each MEPDG run, except for the variable noted in the testing matrix presented in Table 22. Tables 23 and 24 present the traffic and climate input values for the sections using Mixtures A and B, respectively.

The design life and reliability were taken from VDOT guidelines (VDOT, 2003). The initial IRI, terminal IRI, and growth factor were default values. The AADTT, lane configuration, truck percentages, operational speed, and climate were determined for each section. The depth of water table was approximated using engineering judgment.

Tables 25 and 26 present the structural and material input for the Mixture A section. The input stayed the same for every Mixture A MEPDG run. The layer thicknesses were initially determined using a combination of historic records, core information, and ground penetrating

Table 23. Traffic and Climate Input for Mixture A Section

Design life	20 yr
Initial IRI, in/mi	63
Terminal IRI, in/mi	172
Reliability	85
Initial two-way AADTT	22
Number of lanes in design direction	1
Percent of trucks in design direction, %	50
Percent of trucks in design lane, %	100
Operational speed, mph	55
Growth factor	4%
Climate	Charlottesville, Virginia
Depth of water table	15 ft

IRI = International Roughness Index, AADTT = average annual daily truck traffic.

Table 24. Traffic and Climate Input for Mixture B Section

Design life	20 yr
Initial IRI, in/mi	63
Terminal IRI, in/mi	172
Reliability	85
Initial two-way AADTT	70
Number of lanes in design direction	1
Percent of trucks in design direction, %	50
Percent of trucks in design lane, %	100
Operational speed, mph	55
Growth factor	4%
Climate	Roanoke, Virginia
Depth of water table	15 ft

IRI = International Roughness Index, AADTT = average annual daily truck traffic.

radar data. Details of this analysis may be found in Hearon (2008). The MEPDG is limited in the number of layers that can be input, so layers of similar material were combined. In Layer 1, the effective binder content, air voids, total unit weight, and gradation were based on an average of the values measured from the plant-produced mix. The default values for Poisson's ratio, thermal conductivity, and heat capacity were all used. For Layers 2, 3, 4, 5, and 6, a combination of default values and approximations were used. Since the purpose of this exercise was to evaluate the difference in performance of the surface layer, an approximation of the underlying structure was sufficient.

The recovered binder data used for Mixture A are presented in Table 27; this was one of the variables that differentiated the MEPDG runs. Table 28 presents the climatic and traffic loading input that varied between runs for Mixture A; these input values were intended to exaggerate the results of the analysis. The AADTT chosen was greater than the current AADTT by a factor of 10.

The same procedure was followed for Mixture B. Tables 29 and 30 present the material and structural input values for the Mixture B section. The recovered binder data for Mixture B are presented in Table 31; as with Mixture A, this was one of the variables that differentiated the MEPDG runs. Table 32 presents the climatic and traffic loading input values that varied between runs for the Mixture B section; again, these input values were intended to exaggerate the results of the analysis. The AADTT chosen was greater than the current AADTT by a factor of 10.

MEPDG Output

Table 33 presents a summary of the MEPDG analysis results for the Mixture A pavement section. All predicted distress levels met the distress targets. This implies that the predicted long-term performance of both the HMA and WMA met the design life goals. The other important finding was the similar performance of the HMA and WMA; they were not different. Even with an exaggerated climate or traffic load, the change in predicted distresses was practically insignificant.

Table 25. MEPDG Input: Mixture A Pavement Section Layers 1 Through 3

Layer 1	
Material type	Asphalt
Layer thickness, in	1.5
Input level	2
Effective binder content, %	5.25
Air voids, %	7.7
Total unit weight, pcf	151
Poisson’s ratio	0.35
Thermal conductivity of asphalt, BTU/hr-ft-F^o	0.67
Heat capacity of asphalt, BTU/lb-F^o	0.23
Cumulative % retained 3/4 inch sieve	0
Cumulative % retained 3/8 inch sieve	5
Cumulative % retained No. 4 sieve	38
% passing No. 200 sieve	6.1
Layer 2	
Material type	Asphalt
Layer thickness, in	2.8
Input level	3
Binder type	AC-20
Effective binder content, %	4.6
Air voids, %	4
Total unit weight, pcf	148
Poisson's ratio	0.35
Thermal conductivity of asphalt, BTU/hr-ft-F^o	0.67
Heat capacity of asphalt, BTU/lb-F^o	0.23
Cumulative % retained 3/4 inch sieve	0
Cumulative % retained 3/8 inch sieve	12
Cumulative % retained No. 4 sieve	45
% Passing No. 200 sieve	6
Layer 3	
Material type	Asphalt
Layer thickness, in	2.9
Input level	3
Binder type	AC-20
Effective binder content, %	4.6
Air voids, %	4
Total unit weight, pcf	148
Poisson’s ratio	0.35
Thermal conductivity of asphalt, BTU/hr-ft-F^o	0.67
Heat capacity of asphalt, BTU/lb-F^o	0.23
Cumulative % retained 3/4 inch sieve	0
Cumulative % retained 3/8 inch sieve	10
Cumulative % retained No. 4 sieve	40
% Passing No. 200 sieve	6

MEPDG = Mechanistic-Empirical Pavement Design Guide.

Table 26. MEPDG Input for Mixture A Pavement Section Layers 4 Through 6

Layer 4	
Material type	Asphalt
Layer thickness, in	4
Input level	3
Binder type	AC-20
Effective binder content, %	4.6
Air voids, %	4
Total unit weight, pcf	148
Poisson's ratio	0.35
Thermal conductivity of asphalt, BTU/hr-ft-F^{o}	0.67
Heat capacity of asphalt, BTU/lb-F^{o}	0.23
Cumulative % retained 3/4 inch sieve	20
Cumulative % retained 3/8 inch sieve	45
Cumulative % retained No. 4 sieve	57
% passing No. 200 sieve	4
Layer 5	
Material type	Permeable aggregate
Layer thickness, in	6
Input level	3
Poisson's ratio	0.35
Coefficient of lateral pressure	0.5
Modulus, psi	15000
% passing No. 200 sieve	5.5
% passing No. 40 sieve	10
% passing No. 4 sieve	45.8
Maximum dry unit weight, pcf	126.9
Specific gravity of solids	2.7
Saturated gravimetric water content, %	7.5
Calculated degree of saturation, %	61.9
Layer 6	
Material type	Soil subgrade, A-7-6
Layer thickness, in	Semi-infinite
Input level	3
Poisson's ratio	0.35
Coefficient of lateral pressure	0.5
Modulus, psi	11500
Plasticity index	30
Liquid limit	51
% passing No. 200 sieve	79.1
% passing No. 40 sieve	88.8
% passing No. 4 sieve	94.9
Maximum dry unit weight, pcf	97.7
Specific gravity of solids	2.7
Saturated hydraulic conductivity, ft/hr	8.95E-06
Saturated gravimetric water content, %	22.2
Calculated degree of saturation, %	82.7

MEPDG = Mechanistic-Empirical Pavement Design Guide.

Table 27. MEPDG Binder Input for Mixture A Surface Layer

Dynamic Shear on RTFO-Aged Binder	HMA		WMA		
	Initial	6 Mo	Initial	6 Mo	1 Yr
G*, 64°C	5907	7360	12540	7265	
G*, 70°C	3942	4876	5792	4777	6566
G*, 76°C	1904	2363	2752	2277	3090
G*, 82°C					1517
δ, 64°C	79.9	79.75	76.86	79.05	
δ, 70°C	82.04	81.88	79.7	81.21	80.53
δ, 76°C	84.12	83.97	82.14	83.35	82.73
δ, 82°C					84.48

MEPDG = Mechanistic-Empirical Pavement Design Guide, HMA = hot-mix asphalt, WMA = warm-mix asphalt.

Table 28. Mixture A Section Variable MEPDG Input

Input	HMA	WMA
Hot climate, initial binder data	Miami, Florida	Miami, Florida
Cold climate, initial binder data	Buffalo, New York	Buffalo, New York
High traffic, initial binder data	AADTT = 220	AADTT = 220

MEPDG = Mechanistic-Empirical Pavement Design Guide, HMA = hot-mix asphalt, WMA = warm-mix asphalt, AADTT = average annual daily truck traffic.

Table 34 presents the output for the Mixture B pavement section. All distress predictions remained the same between runs except for the asphalt top-down cracking. This could have been due to binder properties but could also have been due to an insufficient structure. The approximately threefold increase in predicted asphalt top-down cracking under a higher traffic load leads to the conclusion that these values were high because the structure of the pavement was insufficient for the applied traffic load. Again, the predicted long-term performance of HMA and WMA was similar for this structure.

The results of the MEPDG analysis support what the results of the field and laboratory evaluations suggested: the long-term performance of both the WMA and HMA will likely be similar.

SUMMARY OF FINDINGS

The findings for the mixtures and Sasobit WMA technology used in this study can be summarized as follows:

- There were no significant differences in volumetrics between the HMA and the WMA. The plant-produced and laboratory-produced mixtures had similar properties. This was also found true for laboratory mixtures produced at varying temperatures.

- Initially, WMA appeared to compact more easily than HMA; however, the in-place compaction of all mixtures was the same. Therefore, laboratory and field compaction procedures for WMA and HMA can be treated the same.

Table 29. MEPDG Input for Mixture B Pavement Section Layers 1 Through 3

Layer 1	
Material type	Asphalt
Layer thickness, in	2
Input level	2
Effective binder content, %	5.33
Air voids, %	6
Total unit weight, pcf	151
Poisson’s ratio	0.35
Thermal conductivity of asphalt, BTU/hr-ft-F°	0.67
Heat capacity of asphalt, BTU/lb-F°	0.23
Cumulative % retained 3/4 inch sieve	0
Cumulative % retained 3/8 inch sieve	16
Cumulative % retained No. 4 sieve	50
% Passing No. 200 sieve	6.2
Layer 2	
Material type	Asphalt
Layer thickness, in	1
Input level	3
Binder type	PG 64-22
Effective binder content, %	5.33
Air voids, %	2
Total unit weight, pcf	150
Poisson’s ratio	0.35
Thermal conductivity of asphalt, BTU/hr-ft-F°	0.67
Heat capacity of asphalt, BTU/lb-F°	0.23
Cumulative % retained 3/4 inch sieve	0
Cumulative % retained 3/8 inch sieve	0
Cumulative % retained No. 4 sieve	5
% Passing No. 200 sieve	10
Layer 3	
Material type	Asphalt
Layer thickness, in	1.3
Input level	2
Binder type	PG 64-22
Effective binder content, %	5.33
Air voids, %	4
Total unit weight, pcf	148
Poisson’s ratio	0.35
Thermal conductivity of asphalt, BTU/hr-ft-F°	0.67
Heat capacity of asphalt, BTU/lb-F°	0.23
Cumulative % retained 3/4 inch sieve	0
Cumulative % retained 3/8 inch sieve	10
Cumulative % retained No. 4 sieve	40
% passing No. 200 sieve	6

MEPDG = Mechanistic-Empirical Pavement Design Guide.

Table 30. MEPDG Input for Mixture B Pavement Section Layers 4 and 5

Layer 4	
Material type	Crushed Stone
Layer thickness, in	6
Input level	3
Poisson's ratio	0.35
Coefficient of lateral pressure	0.5
Modulus, psi	30000
Passing No. 200 sieve, %	8.7
Passing No. 40 sieve, %	20
Passing No. 4 sieve, %	44.7
Maximum dry unit weight, pcf	127.2
Specific gravity of solids	2.7
Saturated gravimetric water content, %	7.4
Calculated degree of saturation, %	61.2
Layer 5	
Material type	Soil Subgrade, A-7-6
Layer thickness, in	Semi-infinite
Input level	3
Poisson's ratio	0.35
Coefficient of lateral pressure	0.5
Modulus, psi	11500
Plasticity index	30
Liquid limit	51
% passing No. 200 sieve	79.1
% passing No. 40 sieve	88.8
% passing No. 4 sieve	94.9
Maximum dry unit weight, pcf	97.7
Specific gravity of solids	2.7
Saturated hydraulic conductivity, ft/hr	8.95E-06
Saturated gravimetric water content, %	22.2
Calculated degree of saturation, %	82.7

MEPDG = Mechanistic-Empirical Pavement Design Guide.

Table 31. MEPDG Binder Input for Mixture B Surface Layer

Dynamic Shear on RTFO-Aged Binder	**HMA**			**WMA**		
	Initial	**6 Mo**	**1 Yr**	**Initial**	**6 Mo**	**1 Yr**
G*, 64°C	4901	5426		5123	6019	6971
G*, 70°C	3238	3564	5566	3370	3999	4537
G*, 76°C	1563	1701	2653	1559	1943	2110
G*, 82°C			1301			
δ, 64°C	81.9	80.62		80.15	80.63	79.92
δ, 70°C	83.77	82.48	81.56	81.79	82.68	82.1
δ, 76°C	85.62	84.33	83.79	83.53	84.7	84.23
δ, 82°C			85.65			

MEPDG = Mechanistic-Empirical Pavement Design Guide, HMA = hot-mix asphalt, WMA = warm-mix asphalt.

Table 32. MEPDG Input: Mixture B Section Variable

Input	HMA	WMA
Hot climate, initial binder data	Miami, Florida	Miami, Florida
Cold climate, initial binder data	Buffalo, New York	Buffalo, New York
High traffic, initial binder data	AADTT = 70	AADTT = 70

MEPDG = Mechanistic-Empirical Pavement Design Guide, HMA = hot-mix asphalt, WMA = warm-mix asphalt, AADTT = average annual daily truck traffic.

- TSR results did not provide a complete understanding of moisture susceptibility. There did, however, appear to be a positive effect from aging for the WMA. In addition, as the production temperature increased, WMA tensile strengths increased.

- The Hamburg wheel-track test results showed that both the HMA and WMA were resistant to moisture susceptibility. The results indicated little difference between the performance of HMA and WMA. The results also suggested that the performance OF WMA could be improved when produced at higher temperatures.

- The TSR results for WMA produced with entrapped moisture showed that WMA was susceptible to moisture damage when the aggregates were not adequately dried during mixing. This susceptibility decreased when the mix was subjected to short-term oven aging.

- The HMA and WMA performed similarly in the APA. The plant-produced WMA rutted slightly less than the HMA; this may have been due to the stiffening properties of Sasobit at temperatures below its melting point. However, these differences were not statistically significant. The laboratory-produced WMA rutted slightly less when produced at higher temperatures than when produced at low temperatures.

- Fatigue results indicated that the HMA performed slightly better at lower strains than the WMA; however, the performances of the mixes appeared nearly equal at higher strains. Again, the WMA appeared to perform slightly better when produced at higher temperatures than when produced at low temperatures.

- Analysis using the MEPDG showed indistinguishable long-term predicted performances by the HMA and WMA for both mixtures.

Table 33. Mixture A Pavement Section MEPDG Output

Factors	Asphalt Surface Down Cracking, ft/mile	Asphalt Bottom Up Cracking, %	Asphalt Thermal Fracture, ft/mi	Permanent Deformation (Asphalt Only), in	Permanent Deformation (Total Pavement), in
Distress target	2000	25	1000	0.25	0.75
HMA initial	0	0	1	0.02	0.16
HMA 6-mo	0	0	1	0.02	0.16
WMA initial	0	0	1	0.02	0.16
WMA 6-mo	0	0	1	0.02	0.16
WMA 1-yr	1	0	0	0.02	0.16
HMA high traffic	0.1	0	1	0.07	0.28
WMA high traffic	0.1	0	1	0.06	0.27
HMA cold weather	0	0	1	0.01	0.14
WMA cold weather	0	0	1	0.01	0.14
HMA warm weather	0	0	1	0.02	0.16
WMA warm weather	0	0	1	0.02	0.16

MEPDG = Mechanistic-Empirical Pavement Design Guide.

Table 34. Mixture B Pavement Section MEPDG Output

Factors	Asphalt Surface Down Cracking, ft/mile	Asphalt Bottom Up Cracking, %	Asphalt Thermal Fracture, ft/mi	Permanent Deformation (Asphalt Only), in	Permanent Deformation (Total Pavement), in
Distress target	2000	25	1000	0.25	0.75
HMA initial	3710	0	1	0.09	0.41
HMA 6-mo	3600	0	1	0.09	0.41
WMA initial	3610	0	1	0.09	0.41
WMA 6-mo	3550	0	1	0.09	0.41
WMA 1-yr	3610	0	1	0.09	0.41
HMA high traffic	3390	0	1	0.09	0.41
WMA high traffic	10000	0	1	0.27	0.74
HMA cold weather	9970	0.1	1	0.27	0.74
WMA cold weather	3150	0	1	0.06	0.37
HMA warm weather	2990	0	1	0.06	0.37
WMA warm weather	4530	0	1	0.13	0.46

MEPDG = Mechanistic-Empirical Pavement Design Guide.

CONCLUSIONS

- No significant differences in volumetric properties are expected between HMA and WMA produced using the same mix design.

- Laboratory and field compaction procedures for HMA and WMA can be treated similarly.

- The failure of one WMA mixture to meet the HMA TSR criterion for moisture susceptibility does not necessarily mean that WMA mixtures are susceptible to moisture damage. Therefore, the significance of any factors that may have influenced the low TSR value should be considered (i.e., moist stockpiles).

- Moisture susceptibility and rutting recommendations for the Hamburg wheel-track test and APA can be treated the same for WMA as for HMA. When evaluated for moisture susceptibility and rutting potential, the WMA showed equal performance to the HMA.

- WMA has a similar fatigue resistance to HMA.

- Based on analysis using the MEPDG, the long-term performance of WMA should be similar to that of HMA.

- Based on the two mixtures and one WMA technology considered in this study, HMA and WMA should have equivalent performance when properly constructed.

- Further research is needed to validate the research findings for mixtures with different binders and aggregate structures and also with different WMA technologies. The performance of WMA in base or intermediate pavement layers should also be investigated.

RECOMMENDATIONS

1. *VDOT's Materials Division should proceed with implementation of a permissive specification allowing the use of WMA produced with reputable and reasonable technologies.* Acceptance property requirements for WMA should not differ from those of HMA with the exception of temperature and TSR values. The recommendations of the WMA technology manufacturer should be followed for temperature. TSR test results were not sufficiently conclusive to determine an acceptable value for the TSR.

2. *The Virginia Transportation Research Council should continue to monitor the existing WMA field sections to validate the laboratory performance predictions.* New installations of different technologies should also be evaluated and monitored to evaluate the suitability for various uses and to determine any limitations on usage.

3. *The Virginia Transportation Research Council should continue to investigate additional WMA technologies, stone matrix asphalt produced using WMA technology, and WMA*

produced with high percentages of RAP. Each of these categories has the potential to provide future benefit to VDOT.

COSTS AND BENEFITS ASSESSMENT

Inclusion of warm asphalt technology as an option for paving operations shows promise in providing benefits to both VDOT and the contracting community. Warm asphalt has the potential to be beneficial to both VDOT and contractors because the asphalt paving season could be extended into cooler weather, thus allowing the use of traditional material temperatures while also allowing lower production temperatures, resulting in reduced cooling time before the pavement could be opened to traffic. The lower production temperatures are also thought to increase mixture durability by reducing the aging of the mix during production, providing the potential for a longer life. Benefits to contractors include the ability to increase hauling distances between plant and project, reduced plant emissions resulting in improved air quality, and reduced energy demand.

Despite its benefits, direct cost savings from the use of WMA are unlikely to be seen by VDOT. Currently, one primary concern with the use of WMA is the initial cost, which varies depending on the technology used. The use of WMA technology requires either additives, a recurrent cost, or asphalt plant modifications, requiring capital investment. Over the long term, the use of WMA could save VDOT considerable dollars if the reduced aging of the mix translates into longer life; however, this has yet to be proven as WMA has not been employed for a sufficient time period to allow an evaluation of this benefit.

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