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ENGINEERING PROPERTIES AND FIELD PERFORMANCE OF WARM MIX ASPHALT TECHNOLOGIES

FINAL REPORT

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DISCLAIMER

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CHAPTER 1

BACKGROUND

Recent surveys show that the use of warm-mix asphalt (WMA) continues to expand in the United States due to its environmental benefits, energy savings, and construction advantages. In at least eight states, more than half of all asphalt paving mixtures are now produced using WMA technologies (*1*). However, as WMA moves into mainstream use, one of the obstacles to implementation is uncertainty about how WMA may affect short and long-term field performance. Since asphalt binders may harden less at the lower production temperatures used with WMA, there has been some concern that WMA pavements may have a greater potential for rutting. There has also been concern about WMA pavements being more susceptible to moisture damage. Furthermore, a better understanding of how WMA affects engineering properties of asphalt mixtures and how those properties relate to field performance is needed to facilitate the implementation of this technology.

INTRODUCTION

Attention to the impact of mankind's activities on the environment has increased around the world. An outgrowth of this interest was the Kyoto Protocol that challenged nations to reduce their collective emissions of six greenhouse gases by 5.2 percent of 1990 levels, with the majority of this decrease expected to come from manufacturing. In many parts of the world, the asphalt paving industry has begun to use WMA in lieu of hot-mix asphalt (HMA) to reduce greenhouse gases emitted during asphalt paving operations. The primary difference between WMA and HMA is the temperature at which it is produced. The production temperature of WMA is typically 25 to 90°F (15 to 50°C) below that of HMA. The actual temperature reduction depends upon the warm mix technology used.

Development of the first WMA technologies began in Europe where its use has remained limited for the past decade. In 2002, representatives from the United States asphalt paving industry traveled to Europe to learn about Europeans' advancements in the area of WMA. The first documented WMA pavement in the United States was constructed in 2004, and since then, several hundred field trials have been constructed.

WMA technologies allow the complete coating of aggregates, placement, and compaction at lower temperatures than conventional HMA. Although the reduction in temperature varies by technology, WMA is generally produced at temperatures ranging from 25°F lower than HMA to the boiling point of water (212°F). Simply put, WMA technologies are aids to workability and compaction.

Currently, there are three categories of WMA technologies: asphalt foaming technologies, organic additives, and chemical additives. A fourth category, referred to as hybrids, utilizes combinations of the other categories. The asphalt foaming technologies include a variety of processes to foam asphalt including water-injecting systems, damp aggregate, or the addition of a hydrophilic material such as a zeolite. In the asphalt plant, the water turns to steam, disperses throughout the asphalt, and expands the binder, providing a corresponding temporary increase in volume and fluids content, similar in effect to increasing the binder content. Available chemical additives often include surfactants that aid in coating and lubrication of the asphalt binder in the mixture. The organic additives are typically special types of waxes that cause a decrease in binder viscosity above the melting point of the wax. Therefore, wax properties are carefully selected based on the planned in-service temperatures. Approximately 30 WMA technologies are currently marketed in the United States.

Benefits of WMA may include reduced emissions, reduced fuel usage, reduced binder oxidation, and paving benefits such as the potential for increased densities, less binder aging, cool-weather paving, longer haul distances and improved working conditions for the paving crew. These purported benefits need to be better documented. Although most aspects of designing and constructing WMA are similar to those of HMA, lower production temperatures and changes in binder characteristics associated with WMA could result in differences in pavement performance relative to HMA. Reduced oxidation of the binder may improve the cracking resistance of a pavement but may reduce its moisture and rutting resistance. Reduced oxidation and better compactability of WMA may allow for higher percentages of reclaimed asphalt pavement (RAP); however, the lower mixing temperatures may not facilitate the initial extent of blending of the aged and virgin binder typically seen with HMA.

The two primary concerns associated with WMA are the potential for rutting and moisture damage. Since the mixing and compaction temperatures are lower than those of HMA, the binder experiences less aging and can be less stiff and potentially more prone to rutting. Moisture susceptibility is a concern with WMA because the aggregates are not exposed to the higher mixing temperatures associated with HMA and, therefore, may not be dried completely. In addition, binders are less oxidized during the mix production process, and softer binders can be more susceptible to moisture damage susceptibility (*2*).

Evidence of the environmental benefits of WMA also needs to be better documented. If WMA is demonstrated to reduce fuel consumption and stack emissions while facilitating higher RAP and reclaimed asphalt shingle contents (RAS), then the use of WMA would be a significant step towards sustainable development for highway agencies and industry. Reduction of emissions other than carbon dioxide (CO_2) may also assist in compliance in non-attainment areas. Additionally, the use of WMA could further reduce the exposure of workers to asphalt fumes.

PROJECT OBJECTIVES

The four primary objectives of this study were to (1) establish relationships between laboratory measured engineering properties of WMA mixes and the field performance of pavements constructed with WMA technologies, (2) compare the relative measures of performance between

WMA and conventional HMA pavements, (3) compare production and placement practices, and if possible, costs between WMA and HMA pavements, and (4) provide relative energy usage, emissions measurements, and fume exposure of WMA compared to conventional HMA.

SCOPE

This research was divided into two phases. The first phase involved literature reviews on engineering properties of WMA mixtures, WMA mix design, production, environmental and emissions assessments, and field performance of WMA. From these reviews, a state-of-knowledge report on WMA was prepared. Phase I also included the development of experimental plans to accomplish the research objectives. A three-volume Interim Report that included the literature reviews, the current state-of-knowledge report, and the amplified experimental plan was submitted to NCHRP in December 2009.

The second phase of the project involved executing the approved experimental plans to gather materials from WMA field projects, evaluate the engineering properties of WMA and HMA, compare the early-life field performance of WMA and HMA, quantify energy, emissions, and health benefits associated with WMA, and validate the WMA mix design recommendations from NCHRP Project 9-43. This two volume report details all the activities and analyses to accomplish these Phase II objectives.

REPORT ORGANIZATION

The final report for this study includes two volumes. This report is Volume I which includes the experiments related to the analysis of engineering properties of WMA compared to HMA and the early field performance of WMA and companion HMA test sections built across the United States. Chapter 1 presents the report introduction, objectives of the project, scope of work, and a summary of accelerated pavement testing of WMA pavement test sections. The experimental plans for laboratory and field testing are presented in Chapter 2. This chapter also contains the plans for performance monitoring and mix design verifications. The results and analyses of laboratory test results and the field performance for each project are presented in Chapter 3, 4, and 5. Chapter 5 also discusses proposed revisions to the Draft Appendix to AASHTO R 35: Special Mixture Design Considerations and Methods for Warm Mix Asphalt (WMA) developed in NCHRP Project 9-43. Chapter 6 provides a brief economic analysis of WMA, and Chapter 7 summarizes the project findings and presents suggestions for modifying current practice.

SUMMARY OF ENERGY USAGE, EMISSIONS MEASUREMENTS, AND FUME EXPOSURE OF WMA COMPARED TO CONVENTIONAL HMA

Volume II of this report details the testing, analysis, and findings associated with the experiments to assess energy savings, plant emissions, and health impacts to paving crews. For convenience, the main findings from Volume II are summarized as follows.

One of the research objectives of this study was to compare plant emissions during WMA production to those during HMA production. The experiments included the following:

1. Monitoring fuel usage for six projects consisting of the production of six HMA control mixtures and eleven WMA mixtures.
2. Measuring plant stack emissions of duplicate production runs at three projects consisting of three HMA control and eight WMA mixtures (twenty-two total measurements).
3. Collecting worker exposures to respirable fumes over complete production days during two multi-technology projects consisting of two HMA controls and six WMA mixes.
4. Developing revised recommendations for monitoring fuel usage based on stack emission data to evaluate energy consumption during mix production.
5. Reviewing and refining procedures for collecting and analyzing worker exposure to fumes during paving. The revised protocol is based on total organic matter instead of benzene soluble matter.

Fuel Usage

Analysis of fuel usage data revealed the importance of comparing the energy consumption of different technologies, such as WMA to HMA, over similar, steady-state, time frames. Historical fuel usage data typically available for HMA production includes fuel used for warm up, plant waste, and end of run cleanout. The data collected in the project experiments showed that an average reduction in mix temperature of 48°F resulted in average fuel savings of 22.1 percent. This was higher than predictions based on thermodynamic material properties. The increased fuel savings appear to be related to the fact that heat radiated through the plant's dryer shell and ductwork into the surrounding environment instead of being transferred to the mix are actually larger than expected. Potential errors were identified for direct measures of fuel usage such as tank sticks and gas meter readings by comparing measured fuel usage to fuel usage calculated from stoichiometric plant stack emissions. Gas meters were found to update usage only after large time intervals, on the order of 30 minutes for some meters, inducing error. Recommended best practices for mix production include reducing aggregate moisture contents by sloping stockpile areas away from the plant, feeding the plant using dryer materials obtained from the high side of the stockpiles, and covering stockpiles with high fines contents. Significant fuel savings were demonstrated for one project with low stockpile moisture contents.

Stack Emissions

Emissions of greenhouse gases such as carbon dioxide (CO_2) decreased with reduced fuel usage. Carbon monoxide (CO) and volatile organic compound (VOC) measurements appear to be more related to burner maintenance and tuning and less related to reductions in fuel usage and consequently the use of WMA. One project with a parallel-flow dryer, using reclaimed oil as fuel, indicated a reduction in VOCs when producing WMA. Significant reductions in sulfur dioxide (SO_2) were observed for the same project. The two other projects used natural gas as fuel, which has a lower sulfur content. Emissions of nitrous oxide (NO_x), a precursor to the formation of ground level ozone, are higher for fuel oils compared to natural gas. With one exception, small reductions in NO_x were noted for WMA. For the exception, the burner was set at 26 percent of its firing rate for the WMA compared to 75 percent for the corresponding HMA at the same production rate. This low firing rate may have resulted in more excess air than necessary for complete combustion, contributing to NO_x formation. Formaldehyde, classified as a hazardous air pollutant, is a byproduct of the combustion of carbon-based fuels. The distribution of formaldehyde measurements was lower for WMA than for HMA and comparable to state-of-the-art plant performance observed in the mid-Atlantic United States.

Worker Exposure

Worker exposure to asphalt fumes has typically been assessed by measuring the benzene soluble fraction of the fumes. In most studies comparing worker exposures between HMA and WMA, benzene soluble fractions were below detectable limits. Thus quantitative comparisons could not be made. Heritage Research Group developed a new measure based on total organic matter (TOM) for this study. Worker exposure was measured at two multi-technology sites. At one site, HMA temperatures behind the screed were cooler than normal for HMA and were actually within the expected temperature range for WMA. This resulted in a low temperature differential between the HMA and WMA; on average only 12°C different. At the other site, mat temperatures immediately behind the screed were on average 50°C cooler. With one exception, the WMA mixtures at both sites resulted in a minimum of 33 percent reduction in TOM; the one exception being an 8.4 percent increase at the site where the HMA was placed near WMA temperatures. The TOM reduction was statistically significant at the 95 percent confidence level for five of six mixes. The asphalt binder at one site showed higher overall emissions in the temperature range typically associated with HMA production. The sample with the highest overall TOM from each mix/site combination was tested for polycyclic aromatic compounds (PAC). Naphthalene was detected in the highest concentrations. Only one non-carcinogenic 4-6 ring PAC, pyrene, was detected and it was from an HMA sample. All of the nine PACs for asphalt reviewed by IARC (International Agency for Research on Cancer) were below detectable limits.

Findings and Suggested Revisions to Practice

The use of WMA reduces fuel usage during mixture production. These reductions can help offset the cost of WMA technologies or equipment. Reductions in stack emissions of greenhouse gases are consistent with reductions in fuel usage. Use of WMA should receive credit for reductions in greenhouse gases in life-cycle assessments. WMA also resulted in reductions in SO_2 when using high sulfur fuels, such as reclaimed oil.

Recommended revisions to the Test Framework for Documenting Emissions and Energy Reductions of WMA and Conventional HMA (*3*) are:

- Corresponding WMA and HMA measurements should be made over similar time periods of steady-state production to compare fuel usage and stack emissions of WMA and HMA,
- Direct fuel measurements— tank sticks, fuel meter, or gas meter readings— should be supplemented with stoichiometric fuel measurements in accordance with EPA Method 19.
- Total organic matter should replace benzene soluble fraction for quantitative comparison of WMA and HMA worker exposure.

PERFORMANCE OF WMA EXPERIMENTAL SECTIONS AT ACCELERATED PAVEMENT TEST FACILITIES

WMA has been evaluated at three noteworthy accelerated pavement test facilities in the United States: the NCAT Test Track, the University of California Pavement Research Center (UCPRC), and MnROAD. This section provides a summary of the performance of the WMA experimental sections tested at these facilities.

NCAT Test Track

Since 2005, several different WMA technologies have been evaluated at the NCAT Test Track. Experimental objectives have varied with the different evaluations. Test sections at the NCAT Test Track are 200 feet in length and are trafficked 16 hours per day in two-year periods by five heavily loaded truck-trailer rigs. Axle loads on the trailers are set at 20,000 pounds, the maximum legal limit permitted on United State interstate highways. Performance of test sections is closely monitored for distress. Some sections are also instrumented to measure the pavement's response to loading and climatic changes. Further details of the NCAT Test Track are reported elsewhere *(4)*.

The first evaluation of a WMA technology on the Test Track occurred in the fall of 2005 when three temporary test sections were constructed to evaluate the rutting performance of MeadWestvaco's early Evotherm ET technology *(5)*. The test sections were built late in the second cycle of the Test Track when previously constructed test sections from another experiment failed and repairs were necessary to safely and efficiently complete the track's operations. Two of the temporary test sections contained Evotherm ET in the intermediate pavement layers. The surface layers were 9.5 mm Superpave mixes and the intermediate layers were 19.0 mm Superpave mixes. One of the three sections was a control section with an HMA surface layer (section N2). The control section contained a PG 67-22 binder. Another section contained Evotherm ET in the surface layer (section E9). The Evotherm ET technology was an emulsion based system that is no longer marketed in the United States. The third section (section N1) contained Evotherm ET and three percent SBR latex by weight of binder in the surface layer.

The same mix design was used for each of the three surface mixes. The surface layers were constructed at a one inch thickness.

The mixes were produced with an Astec Double Barrel plant. The mixing temperature of the WMA mixes was 239°F (115°C), and the target compaction temperature was 225°F (107°C). However, equipment problems were encountered during paving the surface of section N1, so the WMA was kept in a silo for 17 hours. By the time it was placed, the mix had cooled to 205°F (96°C). Once paving was completed, images from an infrared camera showed that there was much less thermal segregation with the WMA sections than with the HMA sections. Cores were used to determine in-place densities. Results showed that each of the surface layers had average densities between 92.1 and 93.4 percent of theoretical maximum specific gravity (G_{mm}), which indicated that Evotherm ET provided good compactability at significantly lower production and placement temperatures than conventional hot mix.

The WMA placed in section N1 was opened to traffic 1.75 hours after paving. After 43 days in service (to the end of the test cycle), the maximum rutting measured in any section was 1.1 mm. During the 43-day time span, 515,333 ESALs were applied to the sections. The Evotherm test sections remained in service throughout the next cycle with no cracking and excellent rutting performance. Section E9 ultimately endured more than 16 million ESALs with only 4 mm of rutting before the test section was removed for a different experiment.

In 2009, another group of WMA and control test sections were constructed as part of the Test Track's 4th research cycle *(4).* These WMA sections were built using the WMA technologies in each lift of a 7-in. asphalt pavement structure. The objective of this experiment was to evaluate the pavement structural responses and short-term performance of WMA under full-scale accelerated pavement testing. State Department of Transportation (DOT) sponsors of the experiment selected two WMA technologies to use in the test sections: Evotherm-DAT and Astec Double Barrel Green, referred to herein as WMA-A (i.e., Additive) and WMA-F (i.e., Foam), respectively.

The test sections were built on a stiff subgrade and a graded-aggregate base commonly used at the Test Track. The cross-sections for each of the test sections consisted of a 3-in. asphalt base course, a 2.75-in. intermediate layer, and a 1.25-in. surface layer. The mix designs for each layer were the same for the control and both WMA sections. The Superpave mixtures were designed using 80 gyrations. Table 1 shows a summary of as-built properties of the test sections. Gradations, asphalt contents, and volumetric properties were reasonably consistent among the three test sections. The asphalt binders from the plant-produced mixtures were extracted, recovered, and graded using AASHTO T 164, ASTM D5404, and AASHTO R39, respectively. The critical high temperatures for the binders recovered from WMA-A mixtures were a few degrees lower than for WMA-F which was possibly due to less plant aging of the binder due to the lower plant mixing temperatures used for WMA-A.

Table 1 As-Built Data for Virgin WMA and Control Mixes

Property	Surface Layer			Intermediate Layer			Base Layer		
	HMA Control	WMA -F	WMA -A	HMA Control	WMA -F	WMA -A	HMA Control	WMA -F	WMA -A
% Passing 25.0 mm	100	100	100	99	99	98	99	99	99
% Passing 2.36 mm	59	60	61	47	48	48	46	47	50
% Passing 0.075 mm	6.0	6.7	6.1	5.3	5.3	4.9	5.1	5.7	5.3
AC (%)	6.1	6.1	6.4	4.4	4.7	4.6	4.7	4.7	5.0
Air Voids (%)	4.0	3.3	3.4	4.4	4.3	4.9	4.0	4.1	3.0
Plant Discharge Temp. (°F)	335	275	250	335	275	250	325	275	250
In-Place Density (% of G_{mm})	93.1	92.3	93.7	92.8	92.9	92.9	92.6	92.3	93.9
Recovered True Grade	81.7 -24.7	82.0 -25.7	80.3 -25.7	85.1 -25.1	86.6 -23.9	82.5 -25.1	77.1 -24.1	75.6 -25.1	73.7 -25.4

The control HMA and WMA sections performed very well through the cycle. No cracking was evident, IRI data were steady, texture changes were very small, and rut depths were satisfactory by most agency standards. Figure 1 shows the rutting progression through the 10 million Equivalent Single Axles Load (ESAL) applications over the two-year trafficking period. Although the rut depths for the WMA sections were slightly higher than those for the control section, likely as a result of the softer binders in the WMA sections, the differences are considered acceptable.

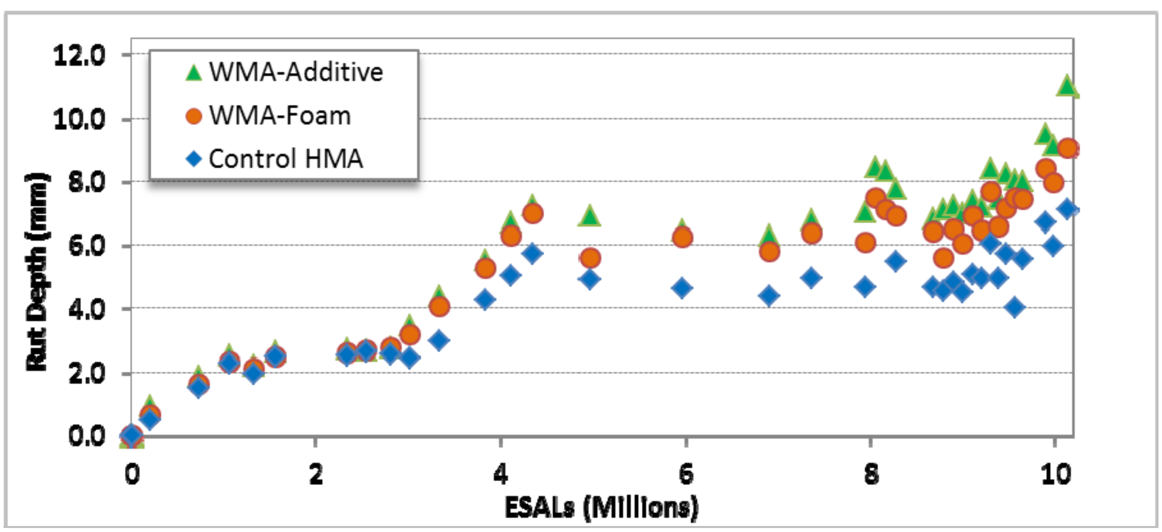


Figure 1 Rutting of the Control HMA and WMA Test Sections in the 4th Cycle of the NCAT Test Track

Falling-weight deflectometer (FWD) testing was performed to compare the seasonal behavior of pavement layer moduli for WMA and HMA test sections. The data presented below are based on FWD tests conducted in the right wheelpath with the 9 kip load. The pavement layer moduli were backcalculated from deflection data using EVERCALC 5.0 for a three-layer cross-section (asphalt-concrete, aggregate base, and subgrade soil). Temperatures of the pavement were recorded near the asphalt pavement surface, mid-depth in the asphalt cross-section, and near the bottom of the asphalt cross-section. Previous studies using Test Track data have shown the effectiveness of using the mid-depth pavement temperature to capture the effect of environmental changes on composite pavement moduli (*6,7)*. Figure 2 shows the plot of moduli versus mid-depth temperature and the regression parameters for these relationships. Statistical analysis of temperature-moduli regression constants k_1 (intercept) and k_2 (slope) indicated that the WMA sections had similar slopes, but lower intercepts than the control HMA section. This indicated that the WMA sections had lower moduli at all temperatures, likely due to the reduced plant aging of the binders for these sections. Further analysis found that the WMA moduli were statistically lower by 7 to 10 percent at the three reference temperatures.

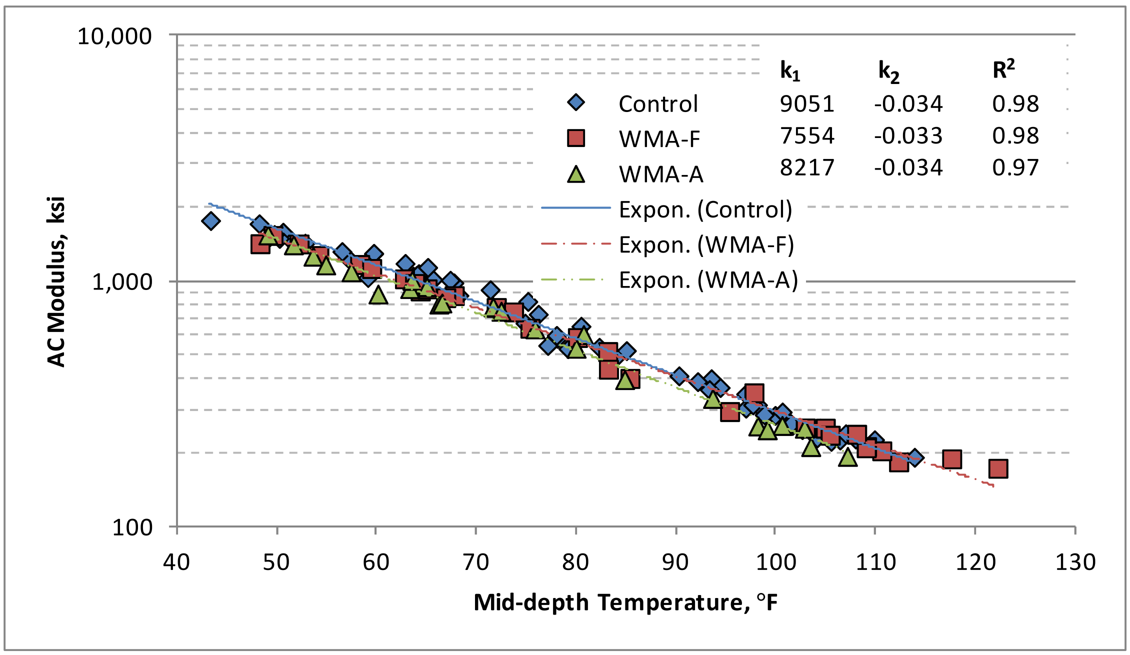


Figure 2 Backcalculated Asphalt Concrete Modulus versus Temperature

These test sections were also instrumented with strain gauges and pressure plates to measure the response of the pavements under live traffic. The strain gauges were installed at the bottom of the asphalt base layer. Longitudinal strain results are reported here since previous studies at the Test Track have shown that longitudinal strains were about 36 percent higher than transverse strain measurements (*6,7*). Figure 3 shows the correlation of longitudinal strain to mid-depth temperature for these three test sections. These relationships follow an exponential function; the regression constants and correlation coefficients are shown in the figure. A statistical analysis found that the regression coefficients of the WMA sections were not statistically different from the control. This indicated that despite the small differences in moduli for WMA and HMA, the pavements did not respond differently under traffic for critical strains.

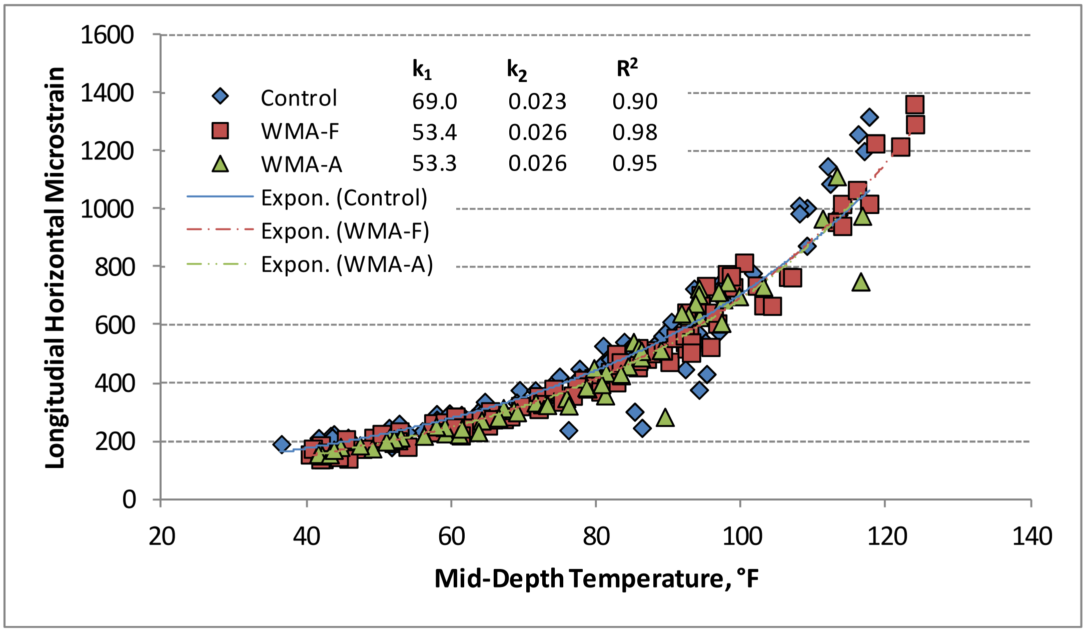


Figure 3 Longitudinal Strain versus Temperature

Another pair of test sections in the 2009 cycle of the Test Track featured WMA combined with 50 percent RAP mixtures. As with the above experiment, the test sections had a 7-in. total asphalt concrete thickness. Both sections contained 50 percent RAP in the each of the three layers. The 50 percent RAP-WMA mixes were produced using the Astec Double-Barrel Green asphalt foaming system. The Superpave mix designs used a PG 67-22 as the virgin binder and an N_{design} of 80 gyrations. No changes were made in the mix designs for the WMA. A summary of the as-produced mix data is shown in Table 2. The virgin control HMA from the previous experiment is also shown for reference. As can be seen, the production temperature for the mixes was reduced by 50°F when the foamed binder WMA was used. True grades of the recovered binders show that the lower production temperatures resulted in a decrease in the high and low critical temperatures for the WMA binders.

Table 2 As-Produced Data for the 50% RAP and Control Mixes

Property	Surface Layer			Intermediate Layer			Base Layer		
	Virgin HMA Control	50% RAP HMA	50% RAP WMA	Virgin HMA Control	50% RAP HMA	50% RAP WMA	Virgin HMA Control	50% RAP HMA	50% RAP WMA
% Passing 25.0 mm	100	100	100	99	98	99	99	99	97
% Passing 2.36 mm	59	48	51	47	46	47	46	47	44
% Passing 0.075 mm	6.0	4.7	4.8	5.3	5.6	5.7	5.1	5.8	5.3
AC (%)	6.1	6.0	6.1	4.4	4.4	4.7	4.7	4.7	4.6
Air Voids (%)	4.0	3.8	3.2	4.4	4.5	3.7	4.0	4.2	4.1
Plant Discharge Temp. (°F)	335	325	275	335	325	275	325	325	275
In-Place Density (% of G_{mm})	93.1	92.6	92.1	92.8	92.9	93.1	92.6	95.0	94.2
Recovered True Grade	81.7 -24.7	87.8 -15.4	83.8 -17.7	85.1 -25.1	N.T.	N.T.	77.1 -24.1	95.0 -12.8	88.7 -14.1

N.T. - not tested. The intermediate and base layers for the 50% RAP HMA and 50% RAP-WMA were produced with the same mix design and at the same temperature. Their recovered binder properties can be presumed to be the same.

Field performance of the 50 percent RAP HMA, 50 percent RAP WMA, and the control section was excellent through the entire two-year trafficking period. Plots of rutting performance are shown in Figure 4. None of the sections had any cracking, IRI was steady, and texture changes were typical for the first two-years of dense-graded surface mixes.

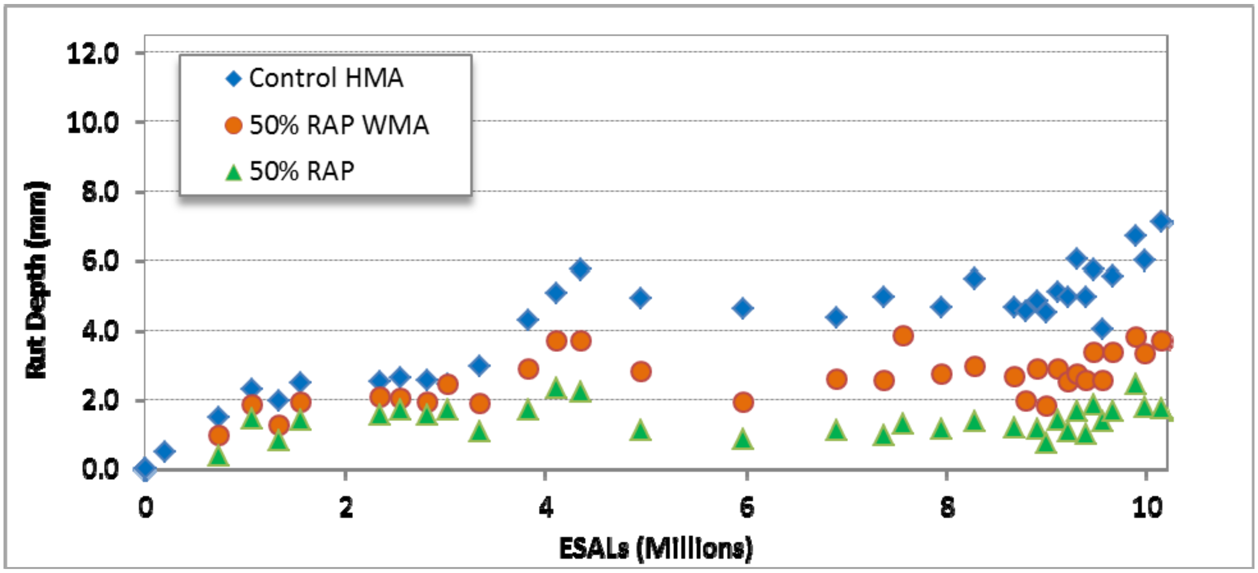


Figure 4 Rutting for Control, 50% RAP HMA and 50% RAP WMA Sections

Pavement moduli back-calculated from FWD testing throughout the research period are shown in Figure 5. Regression parameters for the temperature-moduli relationships are shown in the figure. Statistical analysis indicated that there were significant differences in the moduli among the sections with the 50 percent RAP sections having moduli 16 to 43 percent higher than the virgin control HMA. The largest differences were observed at higher temperatures.

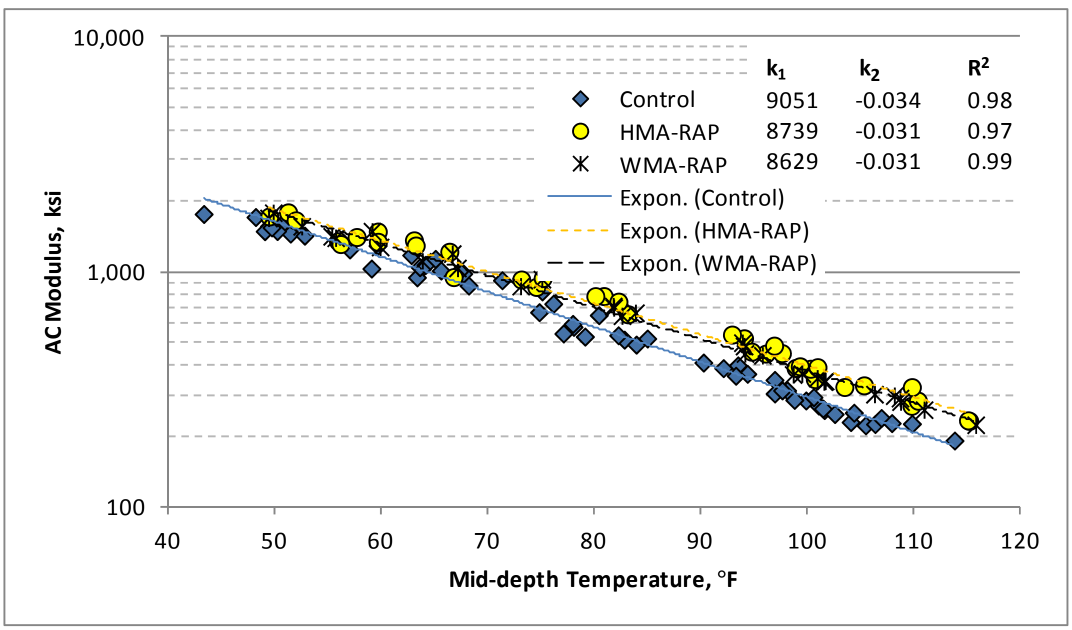


Figure 5 Backcalculated AC Modulus versus Temperature

Longitudinal strain measurements under live traffic were obtained from strain gauges at the bottom of the asphalt base layers. The relationships between this critical strain and mid-depth pavement temperature are shown in Figure 6. A statistical analysis indicated that the measured strain responses of the 50 percent RAP sections were significantly lower than those of the control section by 7 to 31 percent, with the largest differences observed at higher temperatures.

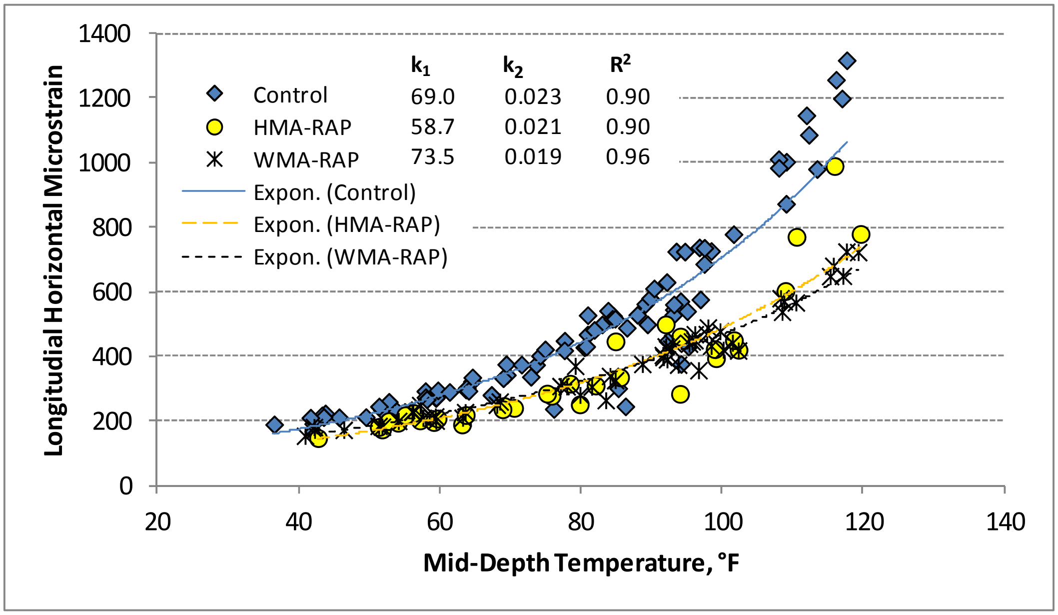


Figure 6 Longitudinal Strain versus Temperature

University of California Pavement Research Center

Heavy Vehicle Simulator (HVS) testing at the University of California's Pavement Research Center (UCPRS) has included two experiments to assess rutting performance of WMA mixes compared to HMA control mixes. In the first HVS rutting experiment, referred to as Phase 1, Advera, Evotherm DAT, and Sasobit were used in a dense-graded mix *(8)*. A standard Hveem mix design was used and no adjustments were made to accommodate the WMA additives. Each section included two lifts of approximately 60 mm of the test mixture.

The WMA technology vendors provided on-site guidance regarding modifications to the asphalt plant to accommodate the WMA additives. Advera and Evotherm DAT were introduced to the mix through pipes installed below and into the asphalt binder supply line, respectively, while the Sasobit was pre-blended with the asphalt binder in a tank before mix production. The target production temperature for the control mix was set at 310°F (155°C) and 250°F (120°C) for the WMAs. Table 3 summarizes the asphalt contents measured using AASHTO T 308 from samples taken during production of the mixes. The binder contents of the HMA control and Advera and Evotherm mixes were similar and close to the target. The binder content of the

Sasobit mix was 0.72 percent below the target. The problem was attributed to a binder feed-rate problem from the tanker during mix production. The low asphalt content for the Sasobit section impacted its performance results as noted below.

Table 3 Asphalt Contents of UCPRC WMA Phase 1 Sections

	Target	Control	Advera	Evotherm	Sasobit
Binder Content (%)	5.2	5.29	5.14	5.23	4.48

The test sections were constructed using conventional equipment and operations. Although some emissions were visually evident from the HMA during transfer of the mix from the truck to the paver, none was observed for the WMA mixes. Some tenderness was noted in the Evotherm DAT and Sasobit sections, resulting in shearing under the rollers, indicating that the compaction temperatures may have been higher than optimal. The Advera mix showed no evidence of tenderness, and acceptable compaction was achieved. In-place densities for the control and Advera mix sections were 94.4 percent and 94.6 percent, respectively. In-place densities for the Evotherm and Sasobit sections were approximately 93.0 percent.

HVS operations followed standard UCPRC protocols. The temperature of the sections was maintained at 122±7°F (50±4°C) at 2-in. (50 mm) below the surface using infrared heaters inside a temperature control chamber. The sections were tested predominantly during the wet season (October through March); however, the sections received no direct rainfall due to cover from the temperature-control chamber.

The HVS loading sequence for each section is summarized in Table 4. Loading was applied with a dual-wheel configuration, using radial truck tires inflated to 104 psi (720 kPa), in a channelized, unidirectional loading mode. An average maximum rut of 0.5-in. (12.5 mm) over the entire section was used as the failure criterion.

Table 4 Summary of Phase I HVS Loading Sequences

Phase	Section	Wheel Load[1] (kN)	Load Repetitions	Total ESALs
1	Control	40 60	185,000 10,000	239,900
	Advera	40	170,000	170,000
	Evotherm	40	185,000	185,000
	Sasobit[2]	40 60	185,000 100,000	734,014

[1] 40 kN = 9,000 lb. 60 kN = 13,500 lb
[2] Testing terminated before failure criteria was reached

Rutting performance for the four sections is shown in Figure 7. The densification during the initial part of the loading was slightly greater (~1 mm) for the Advera (Additive B) and Evotherm (Additive C) sections compared to the control. Beyond the initial densification phase, the rutting rate of these WMA sections was similar to that of the control. The performance of the

Sasobit section was not directly compared to the control section due to the lower asphalt content of the Sasobit section. The UCPRC research team concluded that the three WMA technologies tested in this experiment would not significantly influence rutting performance of asphalt mixes.

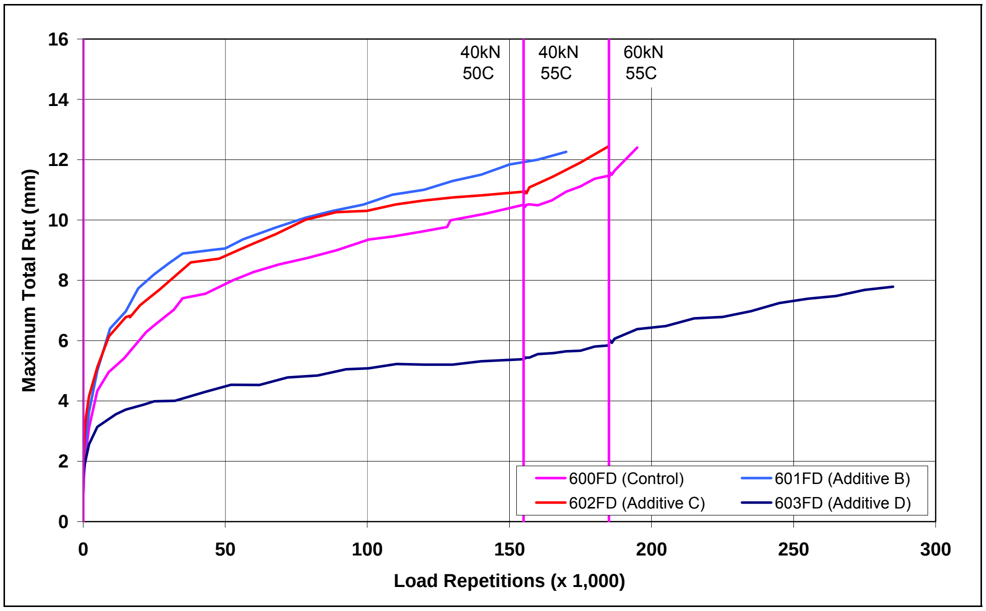


Figure 7 Comparison of Measured Rutting in Phase 1 HVS Testing

Phase 2 of the UCPRC research focused on accelerated testing for moisture damage *(8)*. Prior to testing, each section was presoaked with water for 14 days. A 6-in. (150 mm) high dam was constructed around each test section, and a row of 1-in. (25 mm) diameter holes was drilled 10 inches apart to the bottom of the upper lift of asphalt, well away from the wheel-path. During testing, a constant flow of preheated water (122°F [50°C]) was maintained across the section at a rate of 15 L/hour to try to induce moisture damage. As in Phase 1, the pavement temperature was maintained at 122°F (50°C) at a depth of two inches (50 mm) below the surface. Phase 2 testing began in summer 2008 and ended in spring 2009. The Phase 2 loading sequence is summarized in Table 5.

Table 5 Summary of Phase 2 HVS Loading Sequences

Phase	Section	Wheel Load(kN)	Repetitions	ESALs
2	Control	40	185,000	185,000
		60	80,000	439,200
		90	106,000	3,195,000
	Advera	40	157,000	157,000
		60	32,000	175,700
		90	431,500	13,006,100
	Evotherm	40	166,000	166,000
		60	118,000	647,800
		90	68,000	2,049,600
	Sasobit	40	152,000	152,000
		60	137,000	752,000
		90	175,500	5,289,900

Measured rutting for the four sections during Phase 2 are compared in Figure 8. In this phase, the densification part of rutting for all WMA sections was less than for the control section, opposite of the behavior in the Phase 1. This indicates that the reduced plant-aging of the WMA binders at lower production temperatures may only influence performance in the first few months after construction. As evident in Figure 8, the Evotherm and control HMA sections rutted at a higher rate than the other two sections. This was attributed to the Evotherm and control sections being shaded for much of the day, while the Advera and Sasobit sections had sun most of the day. The shading is believed to have reduced the rate of aging of the Evotherm and control HMA sections. Trafficking was terminated on the Advera and Sasobit sections before the failure criterion was met in the interest of completing the study. None of the sections showed any indication of moisture damage on completion of testing.

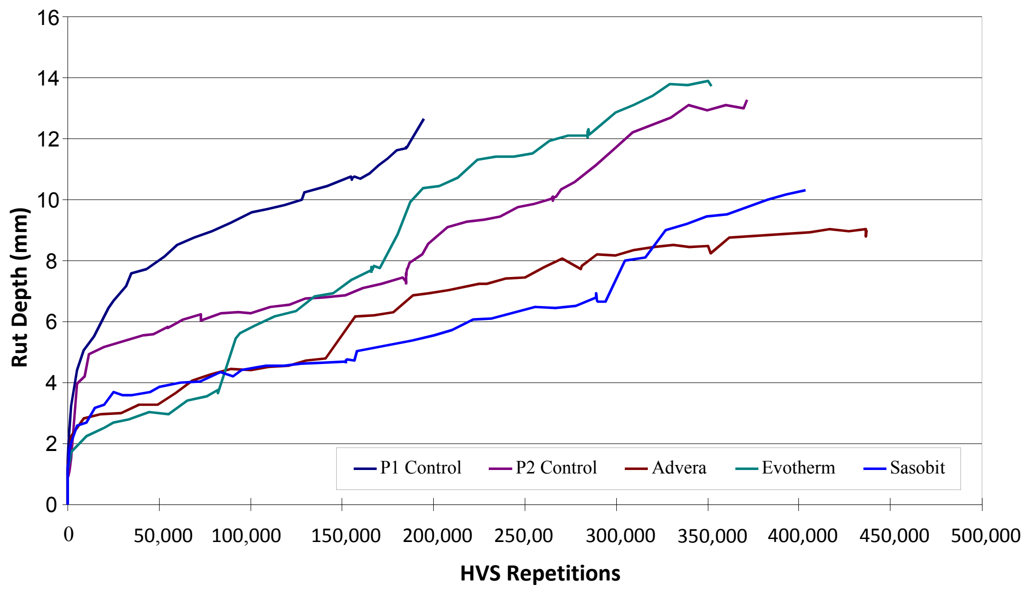


Figure 8 Comparison of Measured Rutting for Phase 2 HVS Testing

Top-down cracking was observed in all four sections. However, the crack patterns, crack lengths, and crack density were similar among the sections. The cracks did not appear to penetrate below the top lift on any section. A forensic investigation found no evidence of moisture damage in any of the sections. Forensic analysis also revealed that rutting was confined to the top lift of asphalt in all four test sections. De-bonding of the top and bottom lifts of asphalt was observed in the control section only. A tack coat was used between lifts.

Although the lower asphalt content of the Sasobit section confounded its comparison to the control HMA, this phase of testing further reinforced findings from the first phase that the three WMA additives do not negatively influence the rutting performance of the mix. The results also indicate that the three WMA additives did not increase the moisture sensitivity of the mixes compared to the control. Binder aging in the WMA and HMA and its effect on performance over time deserves further investigation.

Phase 3 of HVS testing at UCPRC involved the construction and testing of seven WMA technologies with rubber-modified gap-graded mix designs *(9)*. Two groups of test sections were evaluated, each group being produced at a different plant. The first group included a control mix, and WMA sections using Gencor Ultrafoam-GX, Evotherm, and Cecabase. The target binder content for this group was 7.3 percent. The binder contained 18 percent rubber. The mix design was a standard Caltrans rubberized gap-graded mix. No changes were made to the mix design for the WMA technologies. The second group included a new rubberized gap-graded control mix, and WMA sections using Sasobit, Advera, Astec Double-Barrel Green (DBG), and Rediset. The target binder content for this group was 8.3 percent and the binder contained 19 percent rubber. As before, no changes were made to the mix design to accommodate the WMA technologies. Quality control results for the mixes are shown in Table 6. The test results for the first group were consistent. All sections had total binder contents above the target of 7.3 percent and in-place density results were low. Test results for the second group were more variable with binder contents ranging from 7.7 percent for the control mix to 10.0 percent for the Rediset section. In-place density results in the second group were even lower.

Table 6 Quality Control Test Results for the Phase 3 Test Sections

Group 1				
Parameter	Control	Gencor	Evotherm	Cecabase
Binder Content (%)	7.7	7.9	7.7	7.7
Prod. Temp. °F (°C)	320 (160)	284 (140)	248 (125)	266 (130)
Paving Temp. °F (°C)	309 (154)	262 (128)	237 (120)	262 (128)
Lab Air Voids (%)	4.9	6.3	6.2	6.4
In-Place Density (%G_{mm})	90.5	88.8	88.3	89.1

Group 2					
Parameter	Control	Sasobit	Advera	Astec DBG	Rediset
Binder Content (%)	7.7	8.0	7.6	8.4	10.0
Prod. Temp. °F (°C)	335 (166)	300 (149)	295 (145)	295 (145)	285 (140)
Paving Temp. °F (°C)	279 (137)	279 (137)	266 (130)	257 (125)	258 (126)
Lab Air Voids (%)	11.6	8.5	10.7	9.1	8.4
In-Place Density (%G_{mm})	85.8	86.9	85.6	86.0	86.8

The test sections were constructed in one lift at approximately 65 mm thick on top of a nominal 70 mm thick HMA bottom layer. Below the HMA was an aggregate base approximately 40 cm thick.

Results of the HVS testing are shown Figure 9 and Figure 10 for the two groups. In the first group, the Evotherm section performed equivalent to the control section. The Gencor Ultrafoam and Cecabase sections had better rutting performance. The primary difference in the performance of the test sections appeared to occur in the initial densification period. In the second group, the Sasobit section had slightly less rutting (~0.5 mm) than the control section, and Rediset and Astec DBG sections had slightly more rutting (~1 mm) than the control mix until 160,000 load repetitions, when the load magnitude was increased. From that point, the Astec DBG section had an increased rate of rutting. However, this section also had 0.7 percent higher asphalt content compared to the control mix. Interestingly, the Rediset section continued to perform similarly to the control section despite the very high binder content for the Rediset section.

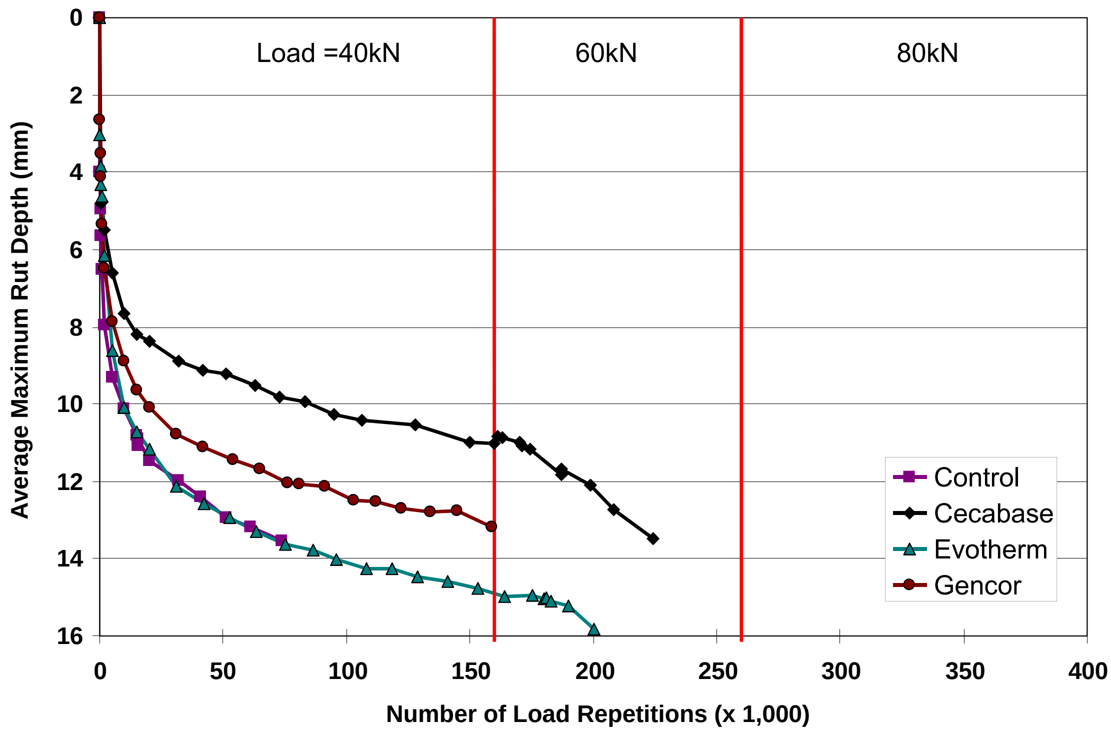


Figure 9 Phase 3 HVS Group 1 Rutting Performance

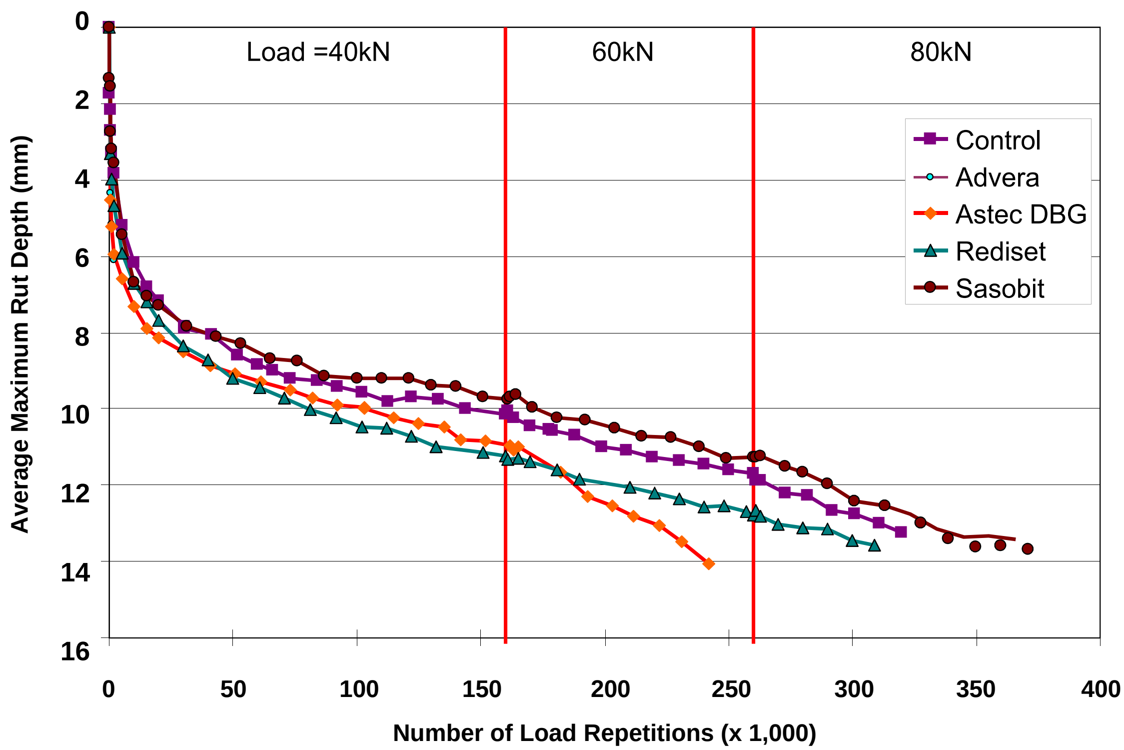


Figure 10 Phase 3 HVS Group 2 Rutting Performance

MnROAD

In 2008, WMA was used in six cells built in on the main line of the MnROAD pavement testing facility. The main line of the facility carries almost 1 million ESAL's per year. A 12.5 mm nominal maximum aggregate size (NMAS), 3-10 million ESAL category mix design was used for the surface and non-surface layers. The mix contained PG 58-34, 20 percent RAP (from MnROAD millings) and Evotherm 3G. The WMA was produced approximately 50°F cooler than normal HMA production temperatures. Five cells were constructed with a 3-in. surface layer and a 2-in. underlying layer over a 12-in. aggregate base, a 7-in. select granular, and a clay subgrade. The five cells varied by the aggregate base which included 100 percent recycled concrete, 50-50 blend of concrete and Class 5 aggregate, 100 percent RAP, taconite railroad ballast, and a control cell using Class 5 aggregate. The sixth cell was a 3-in. WMA overlay of an existing HMA pavement, representing a typical Minnesota rehabilitation strategy. A total of 2100 tons of WMA were used in the six cells. Figure 11 shows an illustration of the WMA related cells. A control HMA section with the same pavement structure and traffic was not constructed.

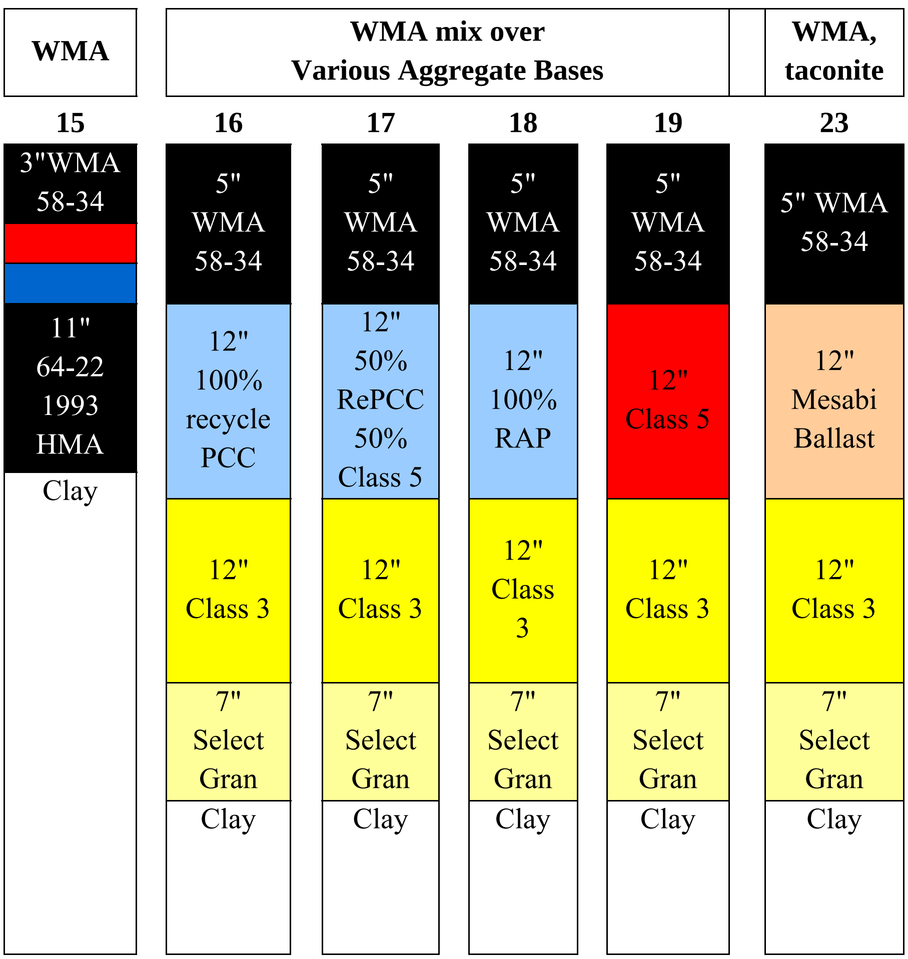


Figure 11 WMA Test Cells at MnROAD

Compaction was measured with a nuclear density gauge and showed equal density to HMA with less effort. The paving crew found the WMA easy to work and appreciated the lower temperatures and lack of fumes behind the paver. The morning after paving, the WMA was still slightly tender, but it stiffened with time. Tensile Strength Ratio (TSR) results on the surface and non-surface layers were 86 percent and 83 percent respectively, indicating that the mixes had good resistance to moisture susceptibility.

As of January 2014, with approximately 4.7 million ESALs and four winter cycles, the WMA sections were performing very well. Table 7 shows the 2013 fall performance survey results for the main driving lane only. Manual distress surveys from the fall of 2013 show that Cell 15, which was built over a previously constructed cracked HMA pavement that had reflective cracking noted as 68.9 m of low-severity transverse cracking and 7.3 m of moderate-severity transverse cracking. Cell 16 had a small amount of transverse cracking; all of the other sections have very little transverse cracking. Some raveling is showing up on cells 18 and 23 mostly along side of the outside paving construction joint near the outside HMA shoulder. None of the sections had any wheelpath (fatigue) cracking. Roughness measurements for all of the WMA cells were considered good and rut depths were mostly around a 7 mm. Both the rutting and ride numbers increase over the last year.

Table 7 Performance of MnROAD WMA Test Cells after 4.5 Years (Driving Lane)

Cell	Transverse Cracking, m		Longitudinal Cracking, m	Raveling, m^2	IRI : Right Wheelpath m/km	Average Rut Depth mm
	Low Severity	Moderate Severity	Low Severity			
15	61.3	7.3	0	0	1.39	5.3
16	1.8	3.7	0	0	1.15	8.1
17	0.6	0.3	1.2	0	1.35	6.9
18	0	0	1.2	48.9	1.11	9.4
19	0	0	36.6	0	1.32	6.9
23	0	0	43.9	11.0	1.25	6.9

Summary of WMA Evaluations at Accelerated Pavement Testing Facilities

A variety of WMA technologies have been tested under heavy loading conditions in APT facilities primarily to evaluate rutting performance. Most of the WMA test sections performed similarly to companion HMA sections. Each of the facilities has reported that compaction of the test sections was aided by the WMA technologies considering the much lower placement temperatures used in the construction of the WMA sections. The NCAT Test Track experiments also demonstrated that WMA mixes provide similar structural response to HMA under traffic and seasonal climate changes. The UCPRS HVS testing also demonstrated that the WMA mixes were not susceptible to moisture damage under saturated conditions. Trafficking continues on the NCAT test sections and MnROAD cells to further evaluate fatigue cracking and wear.

Performance of the WMA cells at MnROAD will also continue to be evaluated for thermal cracking.

CHAPTER 2

EXPERIMENTAL PLAN

INTRODUCTION

Plans for field and laboratory experiments were developed to meet the objectives of this study. The field experiment was developed to gather information to assess short-term pavement performance of new and existing WMA pavements. Field performance assessments were limited to short-term performance since the oldest documented WMA pavement was less than 10-years old at the completion of this study. The field experimental plan also included the collection of energy usage data, plant emissions data, and industrial hygiene testing. That experiment and its data, analyses, and findings are described in Volume II. The laboratory testing determined material properties, compared those properties for WMA and HMA, used the properties in models to predict long-term pavement performance, and validated current recommendations for mix design and testing of WMA in the laboratory.

FIELD PROJECTS: PRODUCTION AND CONSTRUCTION DOCUMENTATION

EXISTING AND NEW PROJECTS

Production and construction information was collected from six WMA projects built prior to the start of this study and eight new WMA projects that were constructed and monitored during the course of this study. The projects built prior to the start of this study are referred to as the "existing projects"; the eight projects built and evaluated during this study are referred to as the "new projects". The existing and new projects are listed in Table 8 and Table 9, respectively. For each project (existing and new), a control HMA section was constructed to provide a direct comparison for field performance and materials properties. The materials properties were also used to examine relationships between engineering properties and field performance.

Table 8 Existing WMA Sites Documented and Sampled

Location	Roadway	WMA Technologies	Date Constructed
St. Louis, MO	Hall Street	Evotherm ET, Sasobit, Aspha-min	Sep-2006
Iron Mountain, MI	M95	Sasobit	Sep-2006
Silverthorne, CO	I-70	Advera, Sasobit, and Evotherm DAT	Aug-2007
Franklin, TN	SR45	Astec DBG, Advera, Evotherm DAT, and Sasobit	Oct-2007
Graham, TX	US 380	Astec DBG	Jun-2008
George, WA	I-90	Sasobit	Jun-2008

Table 9 New WMA Sites Documented and Sampled

Location	Roadway	WMA Technologies	Date Constructed
Walla Walla, WA	US-12	Aquablack	Apr-2010
Centreville, VA	I-66	Astec DBG	Jun-2010
Rapid River, MI	CR-513	Evotherm 3G and Advera	Jun-2010
Baker, MT	CR-322	Evotherm DAT	Aug-2010
Munster, IN	Calumet Ave.	Evotherm 3G, Gencor foam, Heritage wax	Sep-2010
Jefferson Co., FL	SR 30	Terex foaming system	Oct-2010
New York, NY	Little Neck Pkwy.	Cecabase RT, SonneWarmix, BituTech PER	Oct-2010
Casa Grande, AZ	SR-84	Sasobit	Dec-2011

Description of WMA Technologies Evaluated

As previously noted, WMA technologies can be classified in three categories of WMA technologies: chemical additives, asphalt foaming processes and organic additives. In the following paragraphs, a short description of the different technologies that were evaluated as part of this project is presented.

Chemical Additives

Cecabase RT. Cecabase RT was developed by CECA, a division of the Arkema Group. Initially developed in France in 2003, Cecabase RT is a patented, water-free, chemical additive (made up of 50 percent renewable raw materials) that imparts increased workability to asphalt mixtures at lower temperatures. The blend of surfactants in Cecabase RT is designed to reduce the surface tension of the binder, improving coating at low temperatures, and to act as a lubricant at the binder/aggregate interface, facilitating compaction. It is a liquid additive, and can be injected directly into the asphalt line. Recommended addition rates are typically 0.3 to 0.5 percent by weight of asphalt binder *(10)*.

Evotherm. Evotherm is a chemical package used to enhance coating, adhesion, and workability at reduced temperatures. It was developed by Mead Westvaco in the United States. It was originally introduced in 2004 as Evotherm Emulsion Technology (ET). In 2005 Evotherm Dispersed Asphalt Technology (DAT) was introduced, using the same chemical additive as Evotherm ET. The Evotherm DAT is diluted with a small amount of water that will affect the degree of temperature reduction. The chemical solution is injected into the asphalt line before mixing for drum plants, or into the pug mill for batch plants. Evotherm 3G (Third Generation) was later introduced with the difference that the additive does not contain water and can be added at the binder terminal or mix plant. Evotherm DAT allows a slightly higher reduction in temperature than Evotherm 3G *(10)*.

Asphalt Foaming Processes

Advera. Advera is a synthetic zeolite composed of aluminosilicates and alkalimetals that contains approximately 20 percent water of crystallization that is released by increasing the temperature above the boiling point of water. The zeolite releases a small amount of water, creating a controlled, prolonged foaming effect, leading to a slight increase in binder volume and improved mix workability. The product is typically added at 0.20-0.25 percent by total weight of the mix *(10)*.

Aquablack WMA Systems. The Aquablack system uses a stainless-steel foaming gun in conjunction with a center convergence nozzle to produce foaming. The technology produces microbubbles with water pressure up to 1000 psi to atomize the water and create expansion of the foam with microbubbles that are retained through mixing, storage and placement *(10)*.

Aspha-min. This zeolite product is added at a rate of 0.3 percent by total weight of the mixture and is usually added to the mixture at the same time as the liquid asphalt. Similar to Advera, this is a synthetic zeolite composed of aluminosilicates and alkali metals that contains approximately 20 percent of water of crystallization that is released at temperatures above the boiling point of water. A controlled foaming effect is created by the release of water from the zeolite. This effect leads to a slight increase in binder volume. It is reported that this action provides a 6-7 hour period of improved workability which lasts until the temperature drops below approximately 212°F (100°C) *(10)*.

Astec Double-Barrel Green Systems. This water-injection asphalt foaming system uses a multi-nozzle device to microscopically foam the asphalt binder and cause it to expand. Each nozzle injects water into a separate mixing/foaming chamber. The nozzles open and close at the same time. The water is regulated by a positive displacement pump and water flow meter controlled by feedback from the asphalt flow. The rate of water added is approximately 1 pound per ton of mix; a small percentage of this water is encapsulated in the binder as steam, increasing the binder volume *(10)*.

Terex WMA Systems. Using a patented, foamed-asphalt technology developed in 1998, the Terex® WMA System uses a single expansion chamber to provide consistent asphalt binder/water mixture at any desired production rate. The Terex® WMA System is manufactured to fit any unitized counter-flow mixing drum. The only requirement is a jacketed asphalt binder line and water feed pipes that have to be provided by the contractor. The system foams asphalt outside of the rotating drum and then injects the foamed asphalt into the drum's mixing chamber *(10)*.

Organic Additives

Bitutech PER. This additive is intended for use with high RAP or RAS mixes and is reported to improve the mixing of aged and virgin binders. The product is also marketed under the name

"Hydrogreen." The product is added at 0.5-0.75 percent of the total weight of RAP plus RAS. It is designed to supplement the maltene phase of the asphalt binder in mixes with high RAP contents. It also helps in dispersion of asphaltenes and provides viscosity reduction which translates to a better coating of the aggregates and improved compaction at reduced temperatures *(10)*.

Sasobit. Sasobit is described as an "asphalt flow improver" during the asphalt mixing and laydown operations due to its ability to lower the viscosity of the asphalt binder *(6)*. This decrease in viscosity allows working temperatures to be decreased by 32-97°F (18-54°C). Sasobit has a melting temperature of about 216°F (102°C) and is completely soluble in asphalt binder at temperatures above 248°F (120°C). At temperatures below its melting point, Sasobit forms a crystalline network structure in the binder that leads to added stability. Sasobit has been added at rates from 0.8 to 4 percent by mass of the binder depending on recycled binder content and desired properties of the modified binder. It can be added to the asphalt binder or mixture by a number of different methods. Sasobit can be blended directly into the asphalt binder without high-shear blending. This means direct blending can occur either at the terminal or in an asphalt tank at the contractor's plant. For drum-mix plants, Sasobit can also be added to the mix through the reclaimed asphalt pavement (RAP) collar, but it is preferred to use a specially built feeder to regulate the quantity that will be added to the drum. A pelletized form of Sasobit is typically used when adding directly to the mix. In this case, the pellets are blown into the drum at approximately same location where the asphalt binder is added *(10)*.

SonneWarmix. This is a high melt point, paraffinic hydrocarbon blend (wax) that has also been marketed as AD-RAP and SonneWarmix AR. Typical addition rates range from 0.5 to 1.5 percent by total binder weight (including RAP and RAS). Dosages greater than 0.75 percent are not recommended for virgin mixtures. At these addition rates, SonneWarmix is not expected to alter the binder grade. The product must be heated to pump, liquefying between 195-200 °F (91-93 °C). SonneWarmix is generally added to the binder at the terminal or refinery *(10)*.

Production and Construction Information

The research team collected construction data for the new projects. Documentation of the construction information for the control mix and WMA included the items listed in **Table 10**.

Table 10 Field Data for Existing Projects

Data Collected	Frequency	Equipment
Materials Information	One time	N/A
Target Mixing Temperature	Hourly	N/A
Mix Moisture Content	Twice per production day	Oven and a can
Fuel Usage/ Energy Audit	Measured hourly	Dip stick or a fuel meter
Delivery Temperature	Hourly	Temperature gun and a temperature probe
Temperature behind the Screed	Hourly	Temperature gun and a temperature probe
Lift Thickness	Once per day and then checked by cores	N/A
Densities from Cores	Seven per day	Contractor or agency coring rig
Mean Texture Depth	Three locations per mix	Sand and hockey puck

Materials Information: The engineer at the plant collected the job mix formula and warm mix asphalt dosage rate and adjustments to the mix designs.

Target Mixing Temperature: The target mixing temperature for both the HMA and WMA was obtained from the plant operator.

Mix Moisture Content: The engineer at the plant collected two mix moisture contents per day of production. The samples were tested according to AASHTO T 329. The first mix moisture content sample was collected within the first hour of mix being hauled to the paving site. The second mix moisture content sample was collected three hours after the first sample. The moisture contents were determined in the field using the ovens in the NCAT mobile laboratory.

Fuel Usage/Energy Audit: A comprehensive energy audit was conducted for multiple technology projects in conjunction with stack emissions testing.

Delivery Temperature: Delivery temperatures were recorded every 10 minutes at the beginning of each paving day until the delivery temperature stabilized. Experience has shown that the delivery temperature for both HMA and WMA tend to fluctuate at the beginning of each paving day for the first few truckloads or any time the plant starts and stops. Once the delivery temperature stabilized, delivery temperatures were recorded hourly. Identifying the differences in delivery temperatures between the HMA and WMA was important to compare the two types of mixes.

Temperature behind the Screed: Temperature readings were taken immediately behind the screed.

Lift Thickness: The target lift thickness was obtained by the engineer at the paving site. Lift thickness measurements were obtained from cores.

Densities from Cores: Cores were obtained after construction to determine the initial density of the pavement. The cores were obtained by the engineer at the paving site and the densities were determined at the main NCAT laboratory.

Sand Patch: The engineer at the site conducted the sand patch test in accordance with ASTM E 965 at three locations on the finished surface. The location of the tests was recorded using a handheld GPS. The sand patch test provided the mean texture depth of the pavement.

PERFORMANCE MONITORING

Initial Testing for Structural Homogeneity

All of the mixes sampled as part of this project were surface mixes. The comparative performance of the WMA and HMA control sections could be influenced by the underlying pavement structure. To assess this on the "new projects" in this study, FWD testing was completed by the agency or by NCAT if agency data were not available. Arizona, Florida, and Montana provided FWD test data. Virginia DOT planned on providing FWD test data, but due to equipment problems, testing was never completed. NCAT performed FWD testing for the Indiana, Michigan, and New York projects.

Generally, FWD testing was completed prior to placing the test mixes. The Montana testing was performed approximately three-years after the placement of the overlay. ModTag software was used to calculate the subgrade resilient modulus (M_r) and effective structural number (SN_{eff}) of the pavement as described in the 1993 AASHTO Pavement Design Guide (*11*). These data were used to assess the homogeneity of the sections. The backcalculated M_r was considered when selecting subgrade soil properties for the Mechanistic-Empirical Pavement Design Guide (MEPDG). The FWD test results are presented in Appendix A.

Field Performance Data Collection

In order to collect field performance data for the projects, a member of the research team carefully reviewed the entire project length by driving, and then randomly selected three evaluation sections per mix placed during construction for the new projects, or during the first field performance inspection for the existing projects. These evaluation sections were 200 ft. (61 m) in length and contained the location of the original field cores taken at the time of construction. All of the field performance inspections, regardless of whether the site was a new or existing site, included detailed visual examinations and distress mapping of each 200 ft. (61 m) evaluation section to quantify the extent of cracking, rutting, raveling, patching, potholes, shoving, and bleeding. Classification of distresses was in accordance with Long Term Pavement Performance (LTPP) distress identification manual *(12)*. Rutting was assessed by string line measurements or 6 ft. (1.8 m) straight edge. Raveling was quantified by assessing changes in surface macrotexture using the sand patch test (ASTM E 965).

Cores were obtained from one of the randomly selected evaluation sections per mix to assess in-place densification, changes in binder absorption (calculated from maximum specific gravity tests), changes in tensile strength with time, and changes in binder properties based on recovered binder testing. Three cores were taken between wheelpaths and three in the right wheelpath to assess changes in density and strength. An additional core was taken between the wheelpaths to determine the change in binder properties. Table 11 summarizes the field monitoring activities per mix placed.

Table 11 Field Inspection Activities Per Mix Placed

Activity	Section #1	Section #2	Section #3
Map Cracking	✓	✓	✓
Measure Rutting	✓	✓	✓
Map Potholes and Patches	✓	✓	✓
Map Bleeding	✓	✓	✓
Measure Surface Texture	✓	✓	✓
Map Shoving	✓	✓	✓
Obtain Cores in Rt. Wheelpath	3 cores		
Obtain Cores in Between Wheelpath	4 cores		
Windshield Evaluations	1 Pass		

Field Performance Prediction

While this project monitored and compared the short-term performance of WMA versus HMA sections, agencies are also concerned about the long-term performance of WMA. The Mechanistic-Empirical Pavement Design Guide (MEPDG) version 1.003 software with the NCHRP Project 1-37A nationally calibrated models was used to predict the performance of the new WMA and HMA test sections. A 20-year design life was used for all of the projects, although Washington reported a 40-year design life for the pavement. The following describes the data and analysis methods used in the MEPDG.

Traffic volume in vehicles per day and percent trucks were obtained from the agency. In some cases project specific information was provided while in other cases the data was obtained from the agency's online records. Two-way annual average daily truck traffic was calculated for each project from this data. With the exception of the New York project, the same traffic data was used for all of the sections of a given project. The New York project was divided by Hillside Avenue. The traffic counts differed for the Cecabase and BituTech PER sections on one side of Hillside Avenue compared to the SonneWarmix and HMA control on the other side. For the Indiana project, the Gencor foam and HMA control were in the outer lanes and the Evotherm 3G and Heritage wax were in the inner lanes. Observations on site suggested truck traffic utilized both lanes equally; therefore the same traffic was used for all of the mixes.

Expected growth factors were either provided by the agency or calculated using historical data from multiple test dates. Level 3 defaults were used for all other traffic parameters. An appropriate vehicle class distribution was selected based on the roadway functional classification, e.g. principal arterial, minor collector, or local route.

Climatic data were interpolated based on the site's latitude and longitude determined from global positioning satellite (GPS) readings taken at the time of construction except as noted for specific projects.

Subgrade moduli were backcalculated from FWD tests. However, direct input of a "representative" backcalculated subgrade modulus does not allow for seasonal variation due to changes in moisture content or frost conditions *(13)*. Soil classifications were determined using the Web Soil Survey (USDA) *(14).* The most prominent soil classification for a given project was selected and used for all of the sections. The MEPDG Level 3 default moduli for the soil classification determined from the Web Soil Survey were compared to the backcalculated FWD subgrade moduli. The backcalculated moduli were corrected to be comparable to laboratory test values by multiplying by 0.35 *(15).* A pavement design report with soil classification and moduli data was also used for the Walla Walla, WA project. The subgrade depth was entered as semi-infinite; however, the MEPDG automatically divided the subgrade into an upper 12-inch compacted subbase layer and lower semi-infinite layer.

A limited number of full-depth cores were taken at each site. These cores were used in combination with the plans (Michigan, Virginia, and Washington) or historical records (if available) to estimate the thickness of the supporting layers. Dynamic Cone Penetrometer tests were performed in Michigan to estimate the modulus and thickness of the crushed and shaped base. Ground Penetrating Radar tests were performed in Montana to estimate the thickness of the pavement layers. Visual analysis of the cores was used to determine the NMAS of the supporting asphalt layers. The mid-range of the agency's historic gradation bands was used for the Level 3 non-asphalt unbound and bound layers and asphalt mix inputs. Volume of effective asphalt was estimated based on in-place density and voids in mineral aggregate (VMA) requirements. Asphalt binder grade was estimated based on the agency's specifications or historic plans, where available. Aggregate base gradation, where applicable, was also estimated from the mid-point of the agency's specifications.

Level 1 inputs were entered for the WMA and HMA test layers. Layer thickness was the average from cores taken at the time of construction. Moduli were determined from field mixed, laboratory compacted (without reheating) samples tested according to AASHTO TP79. Asphalt binder properties were determined from the AASHTO T315 tests performed on asphalt extracted and recovered from the field cores taken at the time of construction. Effective binder content, in-place air voids, and total unit weight were calculated from the bulk specific gravity of the construction cores, average asphalt content of the field produced mix and maximum specific gravity tests, and bulk specific gravity of the aggregate blend in the job mix formula (JMF).

Creep compliance and strength testing was performed according to AASHTO T 322 on field produced mix from the Walla Walla, WA, Centreville, VA, Rapid River, MI, Baker, MT, and Griffith, IN projects. The MEPDG only accepts creep compliance and strength test data conducted at -4, 14, and 32°F. The samples from Rapid River, MI were tested at lower temperatures due to the project's PG 52-34 binder. Therefore, these data could not be used in the MEPDG. The creep compliance and strength data was entered in the MEPDG for Level 1 thermal cracking analysis for the remaining aforementioned projects. Thermal cracking was evaluated using Level 3 inputs for Rapid River, MI, Jefferson County, FL, New York City, NY, and Casa Grande, AZ.

For each new project, a comparison of the surface-down cracking length and rut depth between HMA and WMA sections is shown in Chapter 3. For the projects where Level 1 creep compliance and strength data were available, thermal cracking comparisons are also presented. Bottom-up fatigue cracking is not reported since the test sections were all wearing courses and the remaining pavement structure would have a greater influence on bottom-up fatigue cracking than the overlay.

Summary comparisons are made between the predicted (50 percent reliability) and observed performance at the field performance monitoring intervals. Comparisons are also made between the WMA and HMA predicted performance at 12 and 20 years with considerations for the observed performance during the monitoring period.

LABORATORY TESTING OF FIELD MIXES

There were two objectives addressed in the laboratory experimental plan: (1) determine the engineering properties of WMA compared to HMA, and (2) determine whether or not the recommended WMA mix design procedures are appropriate. The information to accomplish both objectives was obtained from mixtures and materials collected from existing and new WMA projects. This section details the approach adopted to address the two objectives of the laboratory research.

Engineering Properties

The first objective of the laboratory study was to determine the engineering properties of WMA and control HMA. This objective was accomplished by compiling laboratory test results from materials obtained from existing and new WMA projects.

Engineering properties of plant produced WMA and HMA were used for paired statistical comparisons. The results of the laboratory testing were also used to determine if the current testing procedures could adequately predict the performance of WMA pavements in the field. The engineering properties included those recommended in NCHRP Project 9-43 along with additional testing as agreed upon by the research team and the NCHRP project panel. The laboratory testing program evaluated recovered binder performance grade, mixture stiffness over a wide temperature range, moisture susceptibility, fatigue cracking, thermal cracking, and permanent deformation, as follows:

- Performance Grade of Extracted and Recovered Binder
- Mixture Stiffness -Dynamic Modulus (AASHTO TP 79)
- Moisture Susceptibility (AASHTO T 283)
- Hamburg Wheel Tracking Test (AASHTO T 324)
- Flow Number (AASHTO TP 79)
- AMPT Fatigue (Simplified Viscoelastic Continuum Damage –SVECD model)
- Creep Compliance and Strength (AASHTO T 322)

The following summarizes the purpose of each test selected for this study.

Recovered Binder Performance Grade

The following tests were used to extract and recover the binder from the mixes:

- AASHTO T164 Quantitative Extraction of Asphalt Binder from Hot Mix Asphalt (HMA) Method A using trichloroethylene solvent
- ASTM D5404 Practice for Recovery of Asphalt from Solution using the Rotary Evaporator

The following tests were run to determine the performance grade (PG) of the recovered binders according to AASHTO M320, *Performance Graded Asphalt Binder*, and AASHTO R29, *Grading or Verifying the Performance Grade (PG) of an Asphalt Binder*.

- AASHTO T316, *Viscosity Determination of Asphalt at Elevated Temperatures using a Rotational Viscometer*;
- AASHTO R28, *Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)*;
- AASHTO T315, *Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)*;
- AASHTO T313, *Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)*.

Extracted and recovered asphalt binders were considered to be already short-term aged; therefore the Rolling Thin Film Oven (RTFO) aging procedure normally used to short-term age binders was eliminated. The high temperature grade was determined by testing the as-recovered binder in the DSR at high temperatures as an RTFO-aged binder. The recovered binders were then long-term aged using the Pressure Aging Vessel (PAV) before testing for intermediate temperature DSR and low temperature characteristics using the BBR. Table 12 shows a summary of the binder tests, output, and criteria.

Table 12 Recovered Binder Tests and Criteria

Test	AASHTO Method	Output	Criteria
Rotational Viscosity	T316	Viscosity (Pa-S)	Viscosity ≤ 3.0 Pa-S
Dynamic Shear Rheometer	T315	G* (kPa) and δ (degrees)	RTFO aged Binder: G*/sin(δ) ≥ 2.20 kPa PAV aged binder: G*sin(δ) ≤ 5,000 kPa
Bending Beam Rheometer	T313	S (MPa) and m-value (no units)	S ≤ 300 MPa m-value ≥ 0.300
Pressure Aging Vessel	R28	aged asphalt binder for further testing	no criteria

Mixture Stiffness

Dynamic modulus testing was conducted to assess differences in mix stiffness between WMA and HMA. Also, the dynamic modulus data were used in the MEPDG along with the other pavement and materials properties to predict differences in field performance between WMA and HMA.

Moisture Susceptibility

Moisture susceptibility related to incomplete drying of the aggregate, reduced binder aging due to the lower production temperatures or poor test results that have been obtained for some laboratory and field mixes (*16*, *17*) is one of the greatest concerns for WMA pavements. The most common moisture susceptibility test in the United States is AASHTO T 283 or a modification of AASHTO T 283. NCHRP Project 9-43 recommended AASHTO T 283 for assessing moisture damage susceptibility of WMA mix designs. Additional testing was conducted herein with the Hamburg wheel tracking test (AASHTO T 324) in an effort to identify which test yields a better prediction of moisture susceptibility in the field.

AASHTO T 283 testing followed the standard method. One freeze-thaw cycle was used as part of the conditioning as stipulated in the standard. Using a freeze-thaw cycle as part of the conditioning process was believed to better identify mixes that may be prone to moisture damage.

The Hamburg wheel tracking test is an empirical measure of a mixture's moisture susceptibility and rutting performance. The secondary creep slope, stripping inflection point, and total rut depth at 10,000 cycles were determined from the Hamburg wheel tracking test. The AASHTO T 324 test procedure was followed, but with tighter tolerances for specimen air voids. The procedure allows for 7±2 percent air voids. For this project, Hamburg specimens were restricted to 7±0.5 percent air voids. Table 13 summarizes the anti-strip additives that were used on each project. For all sections within each project, same dosages were used (control HMA and WMA mixes).

Table 13 Anti-strip Additives by Project

Location	Anti-strip Additive	Dosage (%)
St. Louis, MO	N/A	0.25
Iron Mountain, MI	N/A	N/A
Silverthorne, CO	N/A	1
Franklin, TN	N/A	0.3
Graham, TX	N/A	N/A
George, WA	N/A	N/A
Walla Walla, WA	Unichem 8162	0.25
Centreville, VA	Pavebond Lite	0.5
Rapid River, MI	None	-
Baker, MT	Hydrated Lime	1.38
Munster, IN	None	-
Jefferson Co., FL	None	-
New York, NY	None	-
Casa Grande, AZ	Type II Cement	1

N/A - information not available

Fatigue Cracking

Although fatigue cracking has not been a predominant concern with WMA, the research team evaluated fatigue properties of mixes from selected projects using the uniaxial fatigue testing based on the continuum damage concept developed by Dr. Richard Kim's pavement research group at North Carolina State University (NCSU). The test, referred to as the Simplified Viscoelastic Continuum Damage (SVECD) test, was conducted in the Asphalt Mixture Performance Tester (AMPT). To characterize the fatigue characteristics of a mixture, two tests are performed. The first one is the dynamic modulus determined according to the AASHTO TP 79 test protocol to quantify the linear viscoelastic (LVE) characteristics of the mix; the second test is a controlled crosshead (CX) cyclic fatigue test is performed using software developed at NCSU to acquire the necessary fatigue data. The complete theoretical background of this method can be found elsewhere (*18*).

The results of the fatigue testing were also used to compare WMA and HMA fatigue properties. The mixtures used in the fatigue testing experiments came from the three multiple technology projects.

Thermal Cracking

Thermal cracking, like fatigue cracking, may be improved for WMA compared to HMA since WMA binders are aged less during production. An exception may exist for Sasobit and similar organic additives. Asphalt binders containing Sasobit typically have an increase in the critical low temperature which indicates that those mixes may be slightly more prone to thermal cracking. However, a demonstration site using a wax additive in northern Michigan did not exhibit any thermal cracking after two years *(19).*

A preliminary recommendation from NCHRP Project 9-43 was to evaluate thermal cracking properties of WMA using the indirect tensile (IDT) creep compliance and strength tests (AASHTO T 322). The research team tested thermal cracking potential using AASHTO T 322 on mixes from a limited number of sites where there was a higher potential for thermal cracking. The selected projects were: Walla, Walla, WA, Centreville, VA, Rapid River, MI, Baker, MT and Munster, IN.

The IDT system was used to collect the necessary data for the critical cracking temperature analysis. The testing was conducted using an MTS load frame equipped with an environmental chamber capable of maintaining the low temperatures required for this test. Creep compliance was measured at 0, -10, and -20°C and tensile strength at -10°C in accordance with AASHTO T 322. Lower test temperatures, -10, -20, and -30°C, and tensile strength at -20°C, were used for the Michigan site to correspond with the PG 52-34 binder used on that project. Four samples were prepared for each mix. The first sample was used to find a suitable creep load for that particular mix at each testing temperature. The remaining three samples were tested at this load. Specimens used for the creep and strength tests were prepared to 7±0.5 percent air voids.

Permanent Deformation

Reduced aging of binders due to the lower WMA mix production temperatures may result in WMA mixes being more prone to permanent deformation, particularly early in their service lives. Although field results, thus far, have not indicated that rutting is an issue, some laboratory permanent deformation tests have indicated a potential for more rutting. Tests that have been used for evaluating WMA permanent deformation include the Asphalt Pavement Analyzer rut test, the Hamburg wheel tracking test, and the flow number. NCHRP Project 9-43 recommended that flow number testing be used to evaluate the permanent deformation potential of WMA during mix design.

Prior to beginning this study, FHWA and NCAT had performed flow number tests on confined specimens with a deviator stress of 100 psi, a confining pressure of 10 psi and, a target air void content of 7±0.5 percent. NCHRP Project 9-33 recommended testing unconfined specimens (target air void content of 7±0.5 percent) at the 50 percent reliability high temperature determined from LTPPBind *(20).* Confined tests were believed to better represent field conditions and more accurately predict the performance of certain mix types, such as stone matrix asphalt. The research team conducted some flow number tests using both methods, confined and unconfined, so that the recommendations from NCHRP Project 9-43 could be evaluated and to provide additional information regarding which test condition best matches field performance. The results of the Hamburg testing were also used to evaluate rutting susceptibility of WMA compared to HMA.

Summary of Laboratory Performance Testing

A variety of laboratory tests were conducted to evaluate the mix properties of WMA. The results of all tests were used to compare the engineering properties of WMA to those of HMA. Table 14 provides a summary of the testing for each of the new projects.

Table 14 Summary of Mix Performance Tests

Test	Equipment	Replicates
Dynamic Modulus (AASHTO TP 79)	AMPT	3 Specimens per Mix (12)
Moisture Susceptibility (AASHTO T 283)	Marshall load frame	3 Unconditioned, 3 Conditioned per Mix (6)
Hamburg Wheel Tracking Test (AASHTO T 324)	Hamburg Wheel Tracking Device	Two Twin Sets per Mix (3)
Fatigue (SVECD)	AMPT	4 Specimens per Mix (4)
Thermal Cracking (AASHTO T 322)	MTS	3 Specimens per Mix
Flow Number (FHWA AMPT Method)	AMPT	3 Specimens per Mix
Flow Number (NCHRP Project 9-43 Method)	AMPT	3 Specimens per Mix

MIX DESIGN VERIFICATIONS

The second objective of the laboratory experiment was to determine whether or not the recommended WMA mix design procedures are appropriate. Part of this evaluation is whether or not WMA mixes produced in the laboratory matched those produced in the field.

The mixes from the multi-technology projects (Michigan, Indiana, and New York) along with the mixes from two single-technology sites (Montana and Florida) were verified according to the *Draft Appendix to AASHTO R35: Special Mixture Design Considerations and Methods for Warm Mix Asphalt (WMA)* presented in the NCHRP Project 9-43 final report *(21)*. This selection provided a range of WMA technologies, aggregate types, and production and compaction temperatures.

Determination of Optimum Asphalt Content

The same HMA and WMA design, in terms of target asphalt content and gradation, was used by the contractor for all of the projects selected for mix verification. One goal of the mix verifications was to determine if plant production of WMA could be simulated in the laboratory. Since changes in gradation during plant production would affect the measured volumetric properties, the measured field gradation for a given location and technology was used as the target for the laboratory verification instead of the target gradation from the job-mix formula. Thus, within a given project, there can be differences in the target laboratory gradation, even though all of the sections at a given location were based on the same design.

As described previously, the field asphalt content and gradation represent the average of two replicates. The binder was extracted according to AASHTO T 164 and the gradation of the recovered aggregate determined according to AASHTO T 30. Laboratory trial samples were batched and their gradation determined according to AASHTO T 11 and T 27. Adjustments were made as necessary to match field production.

WMA technologies were introduced into the mix as recommended in the Draft Appendix to AASHTO R 35. Foamed asphalt was produced with a D&H Hydrofoamer. Foamed asphalt was weighed into the aggregate batch on an external scale as described in the Draft Appendix to AASHTO R 35.

During the construction of the WMA and HMA sections, plant production temperatures and temperatures immediately behind the paver screed were measured. When a sample of the mix was taken at the plant, an estimate of the average temperature behind the screed up to that point was provided for compacting samples in the mobile laboratory. This same compaction temperature was used for the laboratory mix verifications. Laboratory samples were aged for two hours at the observed field compaction temperature prior to compaction.

Coating

Once a laboratory optimum asphalt content was determined, mixture coating was evaluated using the AASHTO T 195 Ross Count procedure. NCAT and AMS personnel met early in the project to evaluate samples with differing degrees of coating to develop a shared understanding of what would be considered coated and uncoated. The samples were mixed at the average production temperature recorded for each mix during construction. The Draft Appendix for AASHTO R 35 specifies a mixing time of 90 seconds and notes that the mixing time was developed using a planetary mixer. The commentary for AASHTO R 35 suggests that mixing times for bucket mixers will likely be longer than for planetary mixers. The NCHRP Project 9-47A research team felt that bucket mixers are more commonly used than planetary mixers and are also more economical. AMS used an HMA Lab Supply Model MX-6000 Economy Bucket Mixer with a stock paddle and optional stainless steel bucket to prepare the samples (Figure 12). Samples were mixed for the 90 seconds specified in the Draft Appendix to AASHTO R35. If the mixture produced a degree of coating which failed the specification compared to the field result, a longer mixing time would be tried. If the field degree of coating could still not be achieved, then a planetary mixer would be tried.

Figure 12 Bucket mixer used for mix verifications

Compactability

To evaluate the proposed compaction temperature, the Draft Appendix to AASHTO R 35 specifies that the ratio of the number of gyrations to 92 percent density at 30°C (54°F) below the proposed compaction temperature to the number of gyrations to 92 percent density at the proposed compaction temperature must be less than 1.25. The ratio is based on work by Leiva and West *(22)*. Both sets of samples are mixed and aged at the same temperature. One set is allowed to cool prior to compaction.

Moisture Susceptibility

Similar to Superpave mix design, the Draft Appendix to AASHTO R35 specifies the TSR test according to AASHTO T283 for WMA mix design. This procedure was used in the mix verifications. The tests were conducted at optimum asphalt content. Aging was in accordance with the test procedure. One freeze-thaw cycle was included as specified.

Rutting Resistance

For projects with greater than 3 million design ESALs, the Draft Appendix to AASHTO R35 specifies the flow number test to evaluate rutting resistance. Samples were fabricated according to AASHTO PP60. Cored and sawed samples were prepared at 7.0 ±1.0 percent air voids. Flow number tests were performed according to AASHTO TP 79. Tests were conducted at the 50 percent reliability design temperature determined using LTPP Bind Version 3.1 at a depth of 20 mm from the surface of the pavement.

Summary Comparisons

For each project verified, summary comparisons are made between the field and laboratory produced mixes. Comparisons include volumetric properties, optimum asphalt content, maximum specific gravity, binder absorption, coating, and moisture susceptibility. Comparisons are also made between compactability and in-place density achieved in the field. A summary discussion is provided on the observed changes in optimum asphalt content compared to the HMA and field performance.

CHAPTER 3

WMA FIELD PROJECTS

The existing and new projects are discussed in the chronological order of their construction.

Existing Projects

St. Louis, Missouri

This field trial was placed on Hall Street in St. Louis, Missouri. Hall Street is a four-lane roadway with an additional center turn lane through a heavily trafficked industrial area (*23)*. The approximate average annual daily traffic (AADT) for this portion of Hall Street was 21,000 vehicles per day and 7 percent trucks. The contractor for this project was Pace Construction Company, St. Louis, MO. The original surface was a concrete pavement that had been overlaid with HMA. The reflective cracking in the existing HMA was sealed with a rubberized asphalt sealant. This project originally consisted of another 2-inch HMA overlay to be placed over the existing pavement. However, during paving in cool weather, bumps began to form over the sealed cracks. It was believed that by using WMA in lieu of HMA, the lower placement temperatures might prevent the reflective bumps from occurring because the crack sealant would expand less.

The project was constructed over a 10-day period in May 2006 using three WMA technologies: Aspha-min, Sasobit, and Evotherm ET. The JMF for all mixes consisted of 12.5 mm NMAS Superpave mixture compacted to 100 gyrations. A portion of the HMA had previously been placed in the fall of 2005. The mixture used limestone and porphyry aggregates and contained 10 percent RAP. The asphalt binder used in the mixtures was a polymer-modified PG 70-22 with an anti-stripping agent (ARR MAZ) added at a rate of 0.25 percent by weight of virgin asphalt. The aggregate stockpile percentages used are shown in Table 15, and the design aggregate gradation, asphalt content, and volumetric properties are shown in Table 16.

Table 15 Aggregate Percentages for the St. Louis, MO Project

Aggregate Type	% of Total Aggregate
¾ ”	48
½”	21
Man. Sand	20
RAP	10
Mineral Filler	1

Table 16 Design Gradation, Asphalt Content and Volumetric Properties for the St. Louis, MO Project

Property	JMF
Sieve Size	% Passing
19.0 mm (3/4”)	100
12.5 mm (1/2")	97
9.5 mm (3/8")	89
4.75 mm (#4)	68
2.36 mm (#8)	49
1.18 mm (#16)	34
0.60 mm (#30)	21
0.30 mm (#50)	11
0.15 mm (#100)	7
0.075 mm (#200)	5.2
AC (%)	5.3
Air Voids (%)	4.0
VMA (%)	15.0
VFA (%)	73.0
D/A Ratio	1.10
G_{mm}	2.451

Production

The Evotherm ET addition rate was adjusted so that the resulting asphalt binder residue equaled the control HMA mix design content. Aspha-min was added at a rate of 0.30 percent by weight of total mix, while the Sasobit was added at a rate of 1.5 percent by weight of total asphalt binder. The Sasobit was added using a feeder system that injected the material directly into the mixture at the point where the asphalt binder entered the drum. The Aspha-min was added at this same location.

The production temperature for the control HMA was 320°F. The Sasobit mix was originally produced at 275°F. Once the in-place densities and constructability were deemed acceptable, the production temperature for the Sasobit mix was decreased to 240°F. The Evotherm ET mix was produced at 275°F and then decreased to 250°F. It was further decreased to 225°F once the 250°F temperature was deemed acceptable. The Aspha-min mix was produced at 275°F. Table 17 shows the production temperatures used for each WMA technology.

The plant used to produce these mixes was a CMI counter-flow drum plant using recycled oil for the burner fuel. The plant is shown in Figure 13. The average production rate was approximately 200-250 tons per hour for all of the WMA sections.

Table 17 Average Production Temperatures for St. Louis, MO

	HMA	Aspha-min	Evotherm ET	Sasobit
Average, °F	320	275	275, 250, 225[1]	275, 240[1]

[1]Temperatures were periodically reduced during production.

Figure 13 CMI Counter-Flow Drum Plant in St. Louis, MO *(23)*

Volumetric Mix Properties

During production, loose-mix samples were taken from the end-dump trucks before they left the plant. Samples were typically taken twice a day, once at the beginning of production and once towards the end of production. For each field sample, six volumetric specimens were compacted on-site without significant reheating. Samples were placed in an oven for approximately 30 minutes to account for the heat loss that occurred between sampling and splitting. A second set of volumetric samples was compacted with reheated mix to simulate the comparison between the contractor's and the state DOT's data. All specimens were compacted to 100 gyrations at temperatures equal to the compaction temperature behind the paver Table 18.

Table 18 Volumetric Test Samples for St. Louis, MO (*23*)

Mix	Sample Day	Lab Compaction Temperature, °F	SGC Volumetrics	
			Hot at Plant	Reheated at NCAT
Control	1	300	X	X
	1	250	X	X
Sasobit	2	250	X	X
	2	250	X	X
	3	225	X	X
	3	225	X	X
Evotherm ET	4	250	X	X
	4	250	X	X
	5	225	X	X
	5	200	X	X
Aspha-min	6	250		X

Figure 14 shows the air void contents for the samples compacted both hot and reheated. The error bars display plus and minus one standard deviation of the mean. Asphalt content and gradation analyses were performed according to AASHTO T 164 and AASHTO T30 respectively. These values are also shown in Figure 14. It can be seen that the asphalt content decreased for the second sample taken each day, which affected air void contents. The dust contents varied from sample to sample within mix type which confounded the effect of the compaction temperature.

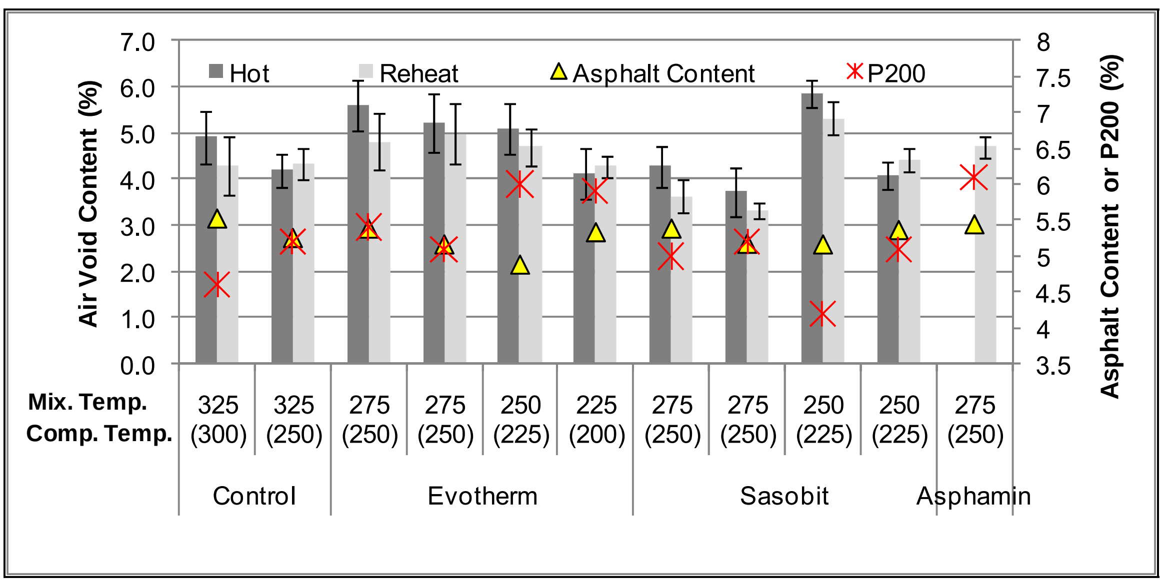


Figure 14 SGC Volumetrics for St. Louis, MO (*23*)

Construction

Paving of the trial sections was performed at night since Hall Street is a highly trafficked commercial roadway. The asphalt mixtures were delivered to the site using end-dump trucks. The haul distance between the plant and the site was approximately 15 miles, or 20 to 25 minutes. Figure 15 shows the layout of the test sections.

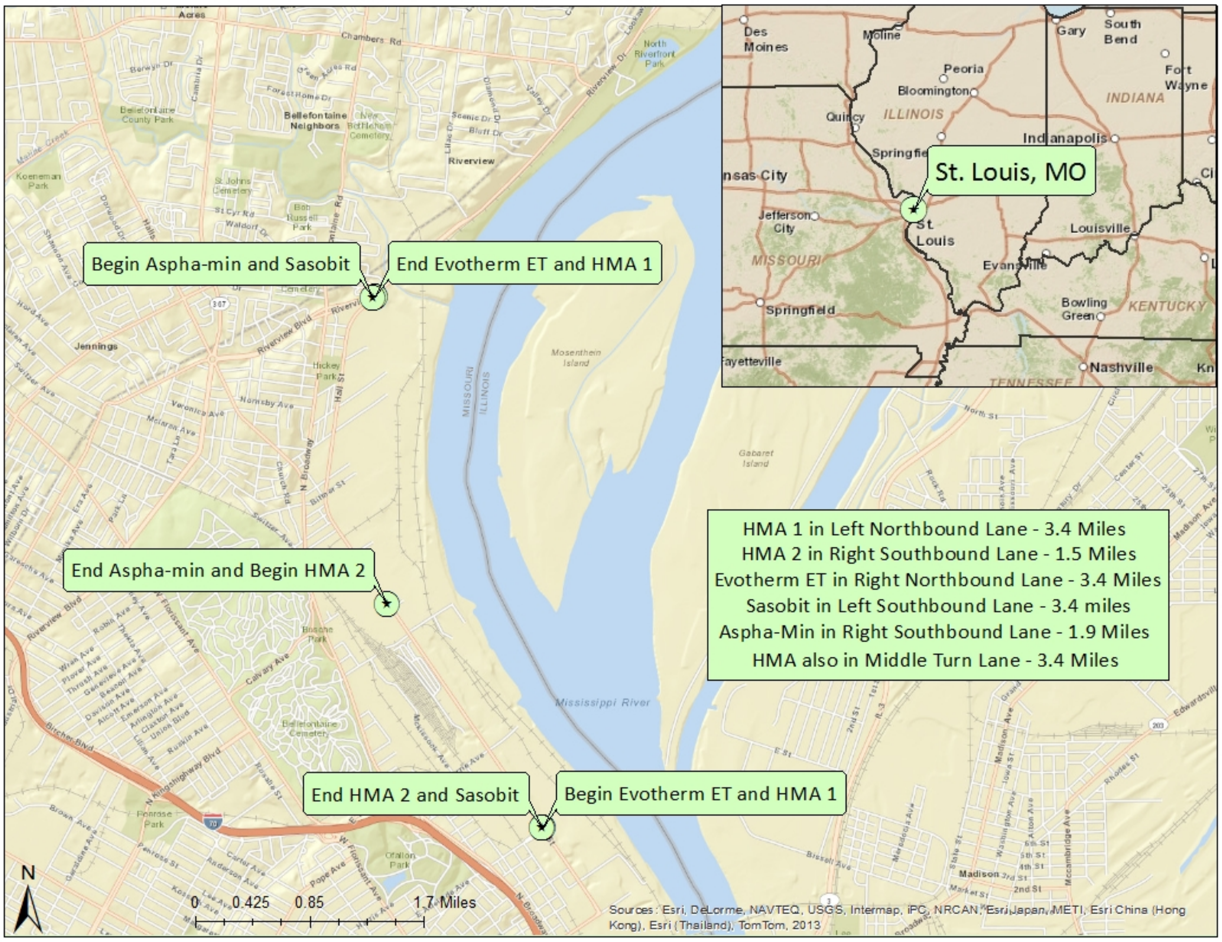


Figure 15 Location of Test Sections in St. Louis, MO

Construction Core Testing

At the time of construction, six cores were taken from both the Evotherm ET and Aspha-min sections. Five cores were taken from the Sasobit section. No construction cores were taken from the control section. Core densities were measured using AASHTO T 166, and the indirect tensile strengths were measured according to ASTM D6931 at 25°C.

Table 19 shows the results of in-place densities and tensile strengths for the three WMA technologies. The average densities were similar and acceptable for the Evotherm ET and

Sasobit WMA mixes. The Aspha-min section has a slightly high average density. The average core tensile strengths were similar for all three WMA mixes, with the Sasobit exhibiting the lowest tensile strength (118.0 psi).

Table 19 Construction Cores Test Results for St. Louis, MO

Test	Statistic	Aspha-min	Evotherm ET	Sasobit
In-place Density (%)	Average	94.9	92.8	91.2
	Standard Deviation	1.2	1.3	1.5
Tensile Strength (psi)	Average	139.4	136.4	118.0
	Standard Deviation	16.4	20.3	45.8

5-Year (64-Month) Project Evaluation

A field performance evaluation was conducted on November 16, 2011 after about 64 months of service. Data were collected on each section to document performance regarding rutting, cracking, and raveling.

The rut depths were measured at the beginning of each 200 ft. (61 m) evaluation section with a string line. Table 20 shows the average and standard deviations of the rut depths. These results show that no appreciable rutting had occurred after more than five years in service.

Table 20 Rut Depths for St. Louis, MO

Mix	Average Rut Depth (mm)	Standard Deviation (mm)
HMA	1.9	0.9
Sasobit	0.8	0.8
Evotherm ET	2.4	0.8
Aspha-min	2.4	1.6

Each evaluation section was carefully inspected for visual signs of cracking. All four mix sections had substantial reflection cracking. It should be noted that the Missouri DOT typically expects these types of overlays to last seven to ten years. This means that the roadway had lasted about 55-75 percent of its expected life at the time of this revisit.

The HMA sections exhibited the least amount of cracking, followed by the Evotherm ET and then the Sasobit. The Aspha-min sections exhibited the most cracking. Table 21 shows the total cracking by crack location and severity according to the method explained in the "Distress Identification Manual for the Long-Term Pavement Performance Program."

Table 21 Cracking Measurements for St. Louis, MO

		Wheelpath Longitudinal		Non-Wheelpath Longitudinal		Transverse		Fatigue	
Mix Section	Severity	# of Cracks	Total Length, m	# of Cracks	Total Length, m.	# of Cracks	Total Length, m	# of Locations	Total Area, m^2
HMA	Low	0	0	2.4	125	22	66.4	0	0
	Mod	0	0	0	0	0	0.0	0	0
	High	0	0	0	0	0	0.0	0	0
Sasobit	Low	0	0	1.2	201	43	128.0	0	0
	Mod	0	0	0	0	1	3.7	0	0
	High	0	0	0	0	0	0.0	0	0
Evotherm	Low	0	0	2.1	215	41	100.6	0	0
	Mod	0	0	0	0	0	0.0	0	0
	High	0	0	0	0	0	0.0	0	0
Aspha	Low	1	9.1	2.7	220	75	188.7	0	0
	Mod	0	0	0	0	4	14.6	0	0
	High	0	0	0	0	0	0.0	0	0

Figure 16 shows an example of the non-wheelpath longitudinal cracking observed in all sections. Figure 17 shows an example of the transverse cracking seen in all sections.

Figure 16 Example of Non-Wheelpath Longitudinal Cracking in St. Louis, MO

Figure 17 Example of Transverse Cracking in St. Louis, MO

The surface texture of each mixture was measured using the sand patch test according to ASTM E965. The sand patch test was conducted at the beginning of each evaluation section in the right wheelpath. The calculated mean texture depths for each mix are shown in Table 22. These values represent the average and standard deviation of the three tests conducted on each mix. A smaller mean texture depth indicates a smoother pavement, or one with less surface texture. All four mixes performed about the same, with the WMA mixtures performing slightly better than the control HMA.

Table 22 Mean Texture Depths for St. Louis, MO

Mix	Mean Texture Depth (mm)	Standard Deviation (mm)
HMA	0.90	0.22
Sasobit	0.81	0.06
Evotherm	0.78	0.08
Aspha-min	0.76	0.04

Core Testing

At the time of the five-year project inspection, seven 6-inch (150-mm) cores were obtained from each mix section. Four of these cores came from between the wheelpaths, and three came from the right wheelpath. These cores were spread throughout the test sections to minimize the damage in any one area. The densities of these cores were measured using AASHTO T 166. If the water absorption was determined to be higher than 1 percent, the samples were then tested according to AASHTO T 331 (vacuum sealing method). Six of the cores were then tested for tensile strength using ASTM D6931. These six samples were then combined and the cut-faces were removed. This mix was split into two samples that were used to determine the maximum specific gravity according to AASHTO T 209. These same two samples were then dried and extracted according to AASHTO T 164. A summary of the results from the core testing is shown in Table 23. Extracted binder tests results are summarized in Chapter 4.

All four mixes had similar gradations and asphalt contents according to these test results. In addition, the in-place densities were similar and acceptable for all four mixes after 64-months of traffic. The binder absorption was slightly higher for the HMA compared to the three WMA technologies, which was expected since the higher temperatures used for HMA production usually caused more binder to be absorbed than compared to the lower temperatures associated with WMA technologies. The tensile strengths after 64-months were all similar. The tensile strengths for the three WMA technologies had all increased compared to construction due to the stiffening of the binder over time. The virgin binder grade was a PG 70-22 at construction, so it can be seen that all mixes had stiffened slightly after 64-months as expected. The high PG grade for the HMA was substantially higher than for the WMA sections possibly due to the increased aging associated with the higher construction temperatures.

Table 23 Average Test Results for St. Louis, MO Five-Year Cores

Sieve Size	HMA	Sasobit	Evotherm ET	Aspha-min
	% Passing			
19.0 mm (3/4")	100.0	100.0	100.0	100.0
12.5 mm (1/2")	95.9	97.2	97.4	97.2
9.5 mm (3/8")	82.7	84.4	85.1	84.4
4.75 mm (#4)	53.3	55.3	55.0	55.3
2.36 mm (#8)	35.7	36.4	36.7	36.4
1.18 mm (#16)	22.3	21.8	22.9	21.8
0.60 mm (#30)	14.6	13.8	14.7	13.8
0.30 mm (#50)	9.5	8.7	9.3	8.7
0.15 mm (#100)	6.5	5.8	6.1	5.8
0.075 mm (#200)	4.8	4.1	4.2	4.1
Asphalt Content (%)	5.23	5.31	5.27	5.21
Avg. Prod. Temp. (°F)	320	275	275	275
G_{mm}	2.464	2.456	2.452	2.455
G_{mb}	2.356	2.312	2.364	2.340
In-place Density (%)	95.6	94.1	96.4	95.3
P_{ba} (%)	0.76	0.67	0.57	0.59
Tensile Strength (psi)	161.5	187.7	181.3	175.5

Table 24 shows the average densities and tensile strengths by location for the five-year cores. All mixes had slightly higher densities in the wheelpaths as expected due to densification under traffic. One other thing to note is that the tensile strength for the HMA in the wheelpath is lower than for any of the three WMA mixtures.

Table 24 In-Place Density and Tensile Strength by Location for St. Louis, MO 5-Year Cores

Location and Property	HMA	Sasobit	Evotherm ET	Aspha-min
Between Wheelpaths Density (% of G_{mm})	95.3	93.8	95.8	94.4
In the Right wheelpath Density (% of G_{mm})	96.1	94.8	97.4	96.8
Between Wheelpaths Tensile Strength (psi)	180.5	186.8	186.6	176.8
In the Right Wheelpath Tensile Strength (psi)	136.2	189.0	174.3	173.7

Iron Mountain, Michigan

A WMA field trial was placed in the northbound lanes of Michigan State Highway 95 (M95) in September 2006 *(19).* The project consisted of widening this portion of M95 to four lanes using a WMA mixture and a HMA control mixture. The WMA was placed as a 1.5-inch overlay in the northbound passing lane, and the HMA was placed 1.9 inches thick in the newly constructed northbound travel lane. The contractor for this construction was Payne and Dolan Inc., Waukesha, WI

The WMA additive used for this field evaluation was Sasobit. Sasobit was introduced into the HMA mix design with the only change being the lower production temperature. The mix design consisted of a 9.5mm NMAS Superpave design compacted to 86 gyrations. The aggregate used in the mix design was basalt, and a PG 58-34 virgin binder was used as the base binder for both mixes. No RAP was used. The stockpile percentages for both mixes are shown in Table 25, and the design aggregate gradation and volumetric properties are shown in Table 26.

Table 25 Aggregate Percentages for Iron Mountain, MI

Aggregate Type	% of Total Aggregate
½” x ¼”	18
¼” Screenings	30
Natural Sand	52

Table 26 Design Gradation, Asphalt Content and Volumetric Properties for Iron Mountain, MI

Property	JMF
Sieve Size	% Passing
12.5 mm (1/2")	100.0
9.5 mm (3/8")	99.1
4.75 mm (#4)	75.0
2.36 mm (#8)	55.9
1.18 mm (#16)	41.3
0.60 mm (#30)	27.5
0.30 mm (#50)	14.5
0.15 mm (#100)	7.5
0.075 mm (#200)	5.5
AC (%)	5.5
Air Voids (%)	4.0
VMA (%)	16.2
VFA (%)	75.4
D/A Ratio	1.08
G_{mm}	2.552

Production

For the WMA mixture, the Sasobit was pre-blended with the base binder at a rate of 1.5 percent by weight of binder. On thousand tons of WMA mix were produced. Mixing temperatures for the control HMA and the WMA were 325°F and 260°F respectively. The asphalt plant used to produce both mixes was located in Spread Eagle, Wisconsin and was a portable, parallel-flow drum plant. The plant incorporated an Adeco drum, Gencor burner, and a Cedar Rapids silo. The burner fuel for the drier was reclaimed oil. A photograph of the plant is shown in Figure 18.

Figure 18 Portable Asphalt Plant used for Iron Mountain, MI Project *(19)*

Volumetric Mix Properties

During construction, mix samples were taken from the loaded trucks before they left the plant. For each sample, six specimens were compacted hot and six were compacted after reheating the mix to determine each mixture's volumetric properties. All samples were compacted at the expected roadway compaction temperature of the respective mix. Samples were compacted without reheating on-site in a Troxler model 4141 Superpave Gyratory Compactor (SGC). Additional mix was brought to NCAT's main lab and reheated then compacted on a Pine model AFG1A SGC. Table 27 shows the average air void contents of the laboratory compacted samples for both heating conditions along with the extracted gradations and asphalt contents.

The gradations for each mix were similar, but the asphalt content for the HMA was 0.28 percent higher than the WMA. This small difference would be expected to result in slightly lower air void content in the HMA compared to the WMA. However, the WMA had a slightly higher dust content, and possibly a lower binder viscosity caused by the Sasobit, which resulted in a lower air void content for the WMA. It can also be seen that the air voids for both mixes increased after reheating as compared to the hot-compacted samples. This was expected since reheating tends to stiffen the asphalt binder and usually leads to higher binder absorption (P_{ba}). It should be noted that the HMA actually had a higher effective binder content, but this was due to the HMA having a higher overall asphalt content. The asphalt absorption was slightly higher for the HMA as expected.

Table 27 Gradation, Asphalt Content, and Volumetrics for Plant-Produced Mix

Property	HMA		Sasobit	
	Hot-Compacted	Reheated	Hot-Compacted	Reheated
Sieve Size	% Passing			
12.5 mm (1/2”)	100.0		100.0	
9.5 mm (3/8”)	98.8		99.2	
4.75 mm (#4)	75.8		79.1	
2.36 mm (#8)	57.5		62.1	
1.18 mm (#16)	43.0		47.8	
0.60 mm (#30)	29.8		34.1	
0.30 mm (#50)	15.8		18.2	
0.15 mm (#100)	8.6		9.2	
0.075 mm (#200)	6.1		6.4	
AC (%)	5.42		5.14	
G_{mm}	2.572		2.562	
G_{mb}	2.467	2.457	2.476	2.440
V_a (%)	4.1	4.5	3.4	4.8
P_{ba} (%)	0.82		0.67	
P_{be} (%)	4.64		4.51	

Construction

The asphalt mixtures were delivered to the site in both live-bottom and end-dump trucks. The haul distance from the plant to the site was approximately eight miles, which corresponded to roughly a 10-minute travel time. Figure 19 shows the project location. The control HMA section was compacted at approximately 300°F, while the WMA was compacted at approximately 250°F.

Figure 19 Location of Test Sections in Iron Mountain, MI

Construction Core Testing

After construction, six 6-inch (150-mm) cores were taken from each section. Table 28 shows the density and tensile strength results from the construction cores. The average in-place densities for both mixes are similar. The tensile strengths are similar, but low. This is due to the soft binder used in this cold climate.

Table 28 Construction Core Test Results for Iron Mountain, MI

Property	Statistic	HMA	Sasobit
In-place Density (% of G_{mm})	Average	94.3	94.6
	Standard Deviation	1.0	0.8
Tensile Strength (psi)	Average	52.2	46.0
	Standard Deviation	3.6	3.5

5-Year (59-Month) Project Evaluation

A field-performance evaluation was conducted on August 11, 2011, after approximately 59 months of service. Data were collected on both sections to document performance regarding rutting, cracking, and raveling.

Rut depths were measured at the beginning of each 200 ft. (61 m) evaluation section using a straightedge and wedge. The HMA exhibited an average of 1.4 mm of rutting with a standard deviation of 0.3 mm. The WMA showed no measurable rutting. Although the HMA had not rutted significantly after five years, it had slightly more rutting than the WMA section. The reason for this difference is more than likely due to the placement of the sections. Since the HMA was placed in the travel lane while the WMA was placed in the passing lane, the HMA was expected to have more rutting.

Each 200 ft. (61 m) evaluation section was carefully inspected for cracking. Only one of the HMA evaluation sections contained cracking, while two of the WMA sections had cracking. However, the number of cracks was fairly low and all cracking was of low severity. Table 29 shows the total cracking by crack type and severity for both mixes.

Table 29 Cracking Measurements for Iron Mountain, MI after 59-Months

Mix Section	Severity	Wheelpath Longitudinal		Non-Wheelpath Longitudinal		Transverse		Fatigue	
		# of Cracks	Total Length, m	# of Cracks	Total Length, m	# of Cracks	Total Length, m	# of Locations	Total Area, m^2
HMA	Low	1	3.7	0	0	1	0	0	0
	Mod	0	0	0	0	0	0	0	0
	High	0	0	0	0	0	0	0	0
Sasobit	Low	0	0	1	0.3	4	14	0	0
	Mod	0	0	0	0	0	0	0	0
	High	0	0	0	0	0	0	0	0

Figure 20 shows the transverse cracking observed in the Sasobit section. It can be seen that this cracking spans across both original middle lanes. The middle-right lane shown in Figure 20 is the WMA mixture, while the middle-left lane is HMA that was not part of this field evaluation. Since this transverse crack goes across both original lanes, it is likely that this is reflective cracking from the underlying concrete.

Figure 20 Transverse Cracking in WMA Section in Iron Mountain, MI

The surface texture of each mixture was measured using the sand patch test. The calculated mean texture depths for both mixtures are shown in Table 30. These values represent the average and standard deviation of the three tests conducted on each test section. A lower mean texture depth indicates a smoother pavement, or one with less surface texture. The two mixes have performed well and comparably in terms of mean texture depth after five years. Figure 21 shows an example of the surface texture of both mixes. HMA is in the far right lane while the WMA test section is shown in the middle-right lane.

Table 30 Mean Texture Depths for Iron Mountain, MI

Mix	Mean Texture Depth (mm)	Standard Deviation (mm)
HMA	0.43	0.04
Sasobit	0.51	0.03

Figure 21 Example of Surface Texture in Iron Mountain, MI

Core Testing

At the time of the five-year project inspection, seven 6-inch (150-mm) cores were taken from each mix section. Four of the cores came from between the wheelpaths, and three came from the right wheelpath. A summary of the core test results is shown in Table 31.

The gradations for the two mixes were similar at the time of the five-year inspection. However, compared to the gradations from the construction mix, both mixes have slightly lower dust contents. The difference in the asphalt contents at the 59-month revisit (0.23 percent) is consistent with the difference measured at construction (0.28 percent). The asphalt contents were about 0.20 percent higher than the results from construction. This is likely due to sampling and material variability. The in-place densities have increased for both mixes since construction as expected. Both mixes have acceptable densities after 59-months. The tensile strengths for both mixes also increased since construction as expected due to binder aging.

Table 31 Test Results from Production Mix and 59-Month Cores from Iron Mountain, MI

Property	HMA	Sasobit	HMA	Sasobit
	Production Mix (Sept. 2006)		59-Month Cores (August 2011)	
Sieve Size	% Passing		% Passing	
12.5 mm (1/2”)	100.0	100.0	100.0	100.0
9.5 mm (3/8”)	98.8	99.2	99.6	99.2
4.75 mm (#4)	75.8	79.1	76.7	75.1
2.36 mm (#8)	57.5	62.1	58.6	56.6
1.18 mm (#16)	43.0	47.8	43.7	43.0
0.60 mm (#30)	29.8	34.1	31.0	30.8
0.30 mm (#50)	15.8	18.2	15.2	15.0
0.15 mm (#100)	8.6	9.2	8.0	7.8
0.075 mm (#200)	6.1	6.4	5.4	5.2
Asphalt Content (%)	5.42	5.14	5.59	5.36
G_{mm}	2.572	2.562	2.572	2.585
G_{mb}	2.433*	2.415*	2.503	2.469
In-place Density (%)*	94.3*	94.6*	97.3	95.5
P_{ba} (%)	0.82	0.67	0.90	0.96
Tensile Strength (psi)*	52.2*	46.0*	71.2	80.7

*Data comes from construction cores

Table 32 shows the average densities and tensile strengths by location for the five-year evaluation cores. It can be seen there was little difference between core locations in regard to in-place density and tensile strength. The HMA has likely densified more than the Sasobit due to higher traffic in the lane where the HMA was placed.

Table 32 In-Place Density and Tensile Strength by Location for Iron Mountain, MI 59-Month Cores

Location and Property	HMA	Sasobit
Between Wheelpaths Density (% of G_{mm})	97.4	95.4
In Right wheelpath Density (% of G_{mm})	97.3	95.7
Between Wheelpaths Tensile Strength (psi)	78.1	76.8
In the Right Wheelpath Tensile Strength (psi)	66.6	84.5

Silverthorne, Colorado

A WMA field trial was placed on I-70 in Colorado about 70 miles west of Denver in July and August 2007 *(24)*. This portion of I-70 is at a high elevation and has a very harsh winter climate. The project began at the town of Silverthorne at milepost (MP) 204.6 and included the three uphill eastbound lanes. The project continued east, up the mountain and terminated at the west portal of the Eisenhower-Johnson Memorial Tunnel at MP 213.6. The contractor, Asphalt Paving Company, Golden, CO placed all mixes at an approximate thickness of 2.5-in.

The existing pavement consisted of 10 to 13 in. of asphalt over fill with an R-value of 75. The pavement design called for 2.5-in. to be milled to remove the pavement distresses. These distresses included thermal cracking, fatigue cracking and longitudinal cracking with some weathering and raveling. After milling, no evidence of these distresses could be seen. The 10-year design used for this field trial assumed 4.85 million 18-kip ESALs. This was calculated using an average annual daily traffic (AADT) of 30,000 and 10 percent trucks.

Three different WMA technologies were used on this field trial along with control HMA sections for each WMA section. The WMA technologies were Advera, Sasobit, and Evotherm DAT. The same Superpave mix design was used for all mixes with the addition of the WMA additive and lower temperatures being the only difference between the control and WMA sections. A fine-graded 12.5 mm NMAS mix was used for all the mixtures. The design used 75 gyrations with a PG 58-28 binder. The aggregate used for this project was from Everist Materials' Maryland Creek Ranch pit and was a crushed river rock. Hydrated lime was added as an anti-stripping agent at 1 percent by weight of aggregate. Table 33 shows the aggregate stockpile percentages.

Table 33 Aggregate Percentages for Silverthorne, CO

Aggregate Type	% of Total Aggregate
½" Gravel	15
#8s	10
Crushed Fines	54
Washed Sand	20
Hydrated Lime	1

Table 34 Design Gradation, Asphalt Content, and Volumetrics for Silverthorne, CO

Property	JMF
Sieve Size	% Passing
12.5 mm (1/2")	100
9.5 mm (3/8")	95
4.75 mm (#4)	73
2.36 mm (#8)	54
1.18 mm (#16)	40
0.6 mm (#30)	29
0.3 mm (#50)	18
0.15 mm (#100)	11
0.075 mm (#200)	6.7
AC (%)	6.3
Air Voids (%)	3.6
VMA (%)	16.8
G_{mm}	2.446

Production

For each of the three WMA technologies used on this project, a small control section of HMA was produced and placed before the WMA section. The HMA control mixtures were produced at a temperature of approximately 305°F. About 100 tons of the HMA was produced before beginning the addition of Advera WMA technology. The Advera WMA was added at a rate of 0.3 percent by total weight of mix. The target mixing temperature for the Advera WMA was 255°F, and approximately 930 total tons were produced. The Advera material was added in powder form to the drum at the same location as the liquid binder. The Advera WMA mixture was produced at between 200 and 250 tons per hour. The production temperature for the Advera ranged from 245°F to 267°F.

The Sasobit product was added at a rate of 1.5 percent by mass of liquid binder. Approximately 225 tons of the control HMA mixture was produced before introducing the Sasobit. The Sasobit mix was produced at a target temperature of 255°F, and approximately 1,020 total tons were produced. The Sasobit was added in prill form to the drum at the same location as the liquid binder. It was fed through a modified fiber feeder. The Sasobit mixture was produced at approximately 250 tons per hour, and the production temperature ranged from 253°F to 257°F.

Evotherm DAT in liquid form was added at a rate of 0.5 percent by weight of binder. Approximately 100 tons of the control HMA was produced before introducing the Evotherm DAT. A pump was used to add the Evotherm DAT material into the binder line through a modified ½-inch inlet. The Evotherm mixture was produced at approximately 250 tons per hour, and the production temperature ranged from 242°F to 257°F. An Astec Double-Barrel plant was used to produce all mixtures on this project.

Volumetric Mix Properties

Test results for asphalt content and volumetric properties were completed by Colorado DOT's Quality Assurance laboratory. Only one or two sets of volumetrics samples were tested for each section. This testing was done on field-produced mix with no reheating. The HMA was compacted at a temperature of 280°F, while the WMA mixtures were all compacted at 250°F. All samples were immediately compacted once they reached the specified laboratory compaction temperature. The compactive effort was 75 gyrations in an SGC to be consistent with the mix design. Table 35 shows the results from the quality assurance testing.

The asphalt contents for all mixes were similar. The air void contents and VMA results for the WMA were lower than for the HMA. The lower air void contents and VMA results may have been due to increased compactability associated with the WMA technologies, slightly higher effective asphalt contents as a result of less adsorption of asphalt into the aggregates due to the lower mixing temperature, or both. Colorado DOT results for the individual G_{mm} tests were not available to calculate the asphalt absorption values. The Hveem stability results were similar for all of the plant produced HMA and WMA mixtures.

Table 35 Asphalt Content and Volumetric Properties for Silverthorne, CO

Property	Target	Control HMA	Advera WMA	Control HMA	Sasobit WMA	Control HMA	Evotherm WMA
AC (%)	6.3	6.23	6.38	6.41	6.32	6.04	6.38
Air Voids (%)	3.6	3.1	1.8	3.0	2.4	3.6	2.2
VMA (%)	16.8	16.5	15.7	16.5	15.9	16.3	15.8
Hveem Stability	39	36	34	35	36	35	34

Construction

Paving was performed at night due high traffic volumes during the day. Distance to the paving sites from the plant varied from 5 to 15 miles, which corresponded to a 10 to 25 minute haul time. The target compaction temperatures for the Advera, Sasobit, and Evotherm DAT were 235°F, 235°F, and 230°F respectively. Table 36 provides the locations of the test sections; Figure 22 shows a map of the test sections.

Table 36 Section Layout for Silverthorne, CO (*21*)

Paving Start Date	Section	Starting MP	Ending MP	Starting Station	Ending Station	Length (feet)
7-24-07	HMA Control	207.42	207.80	179+20	199+20	2000
7-24-07	Advera WMA	207.80	208.86	199+20	255+30	5610
7-26-07	HMA Control	208.86	209.07	255+30	266+20	1090
7-26-07	Sasobit WMA	209.07	210.17	266+20	324+30	5810
8-13-07	HMA Control	210.17	210.28	324+30	330+60	630
8-13-07	Evotherm WMA	210.28	211.38	330+60	388+50	5790

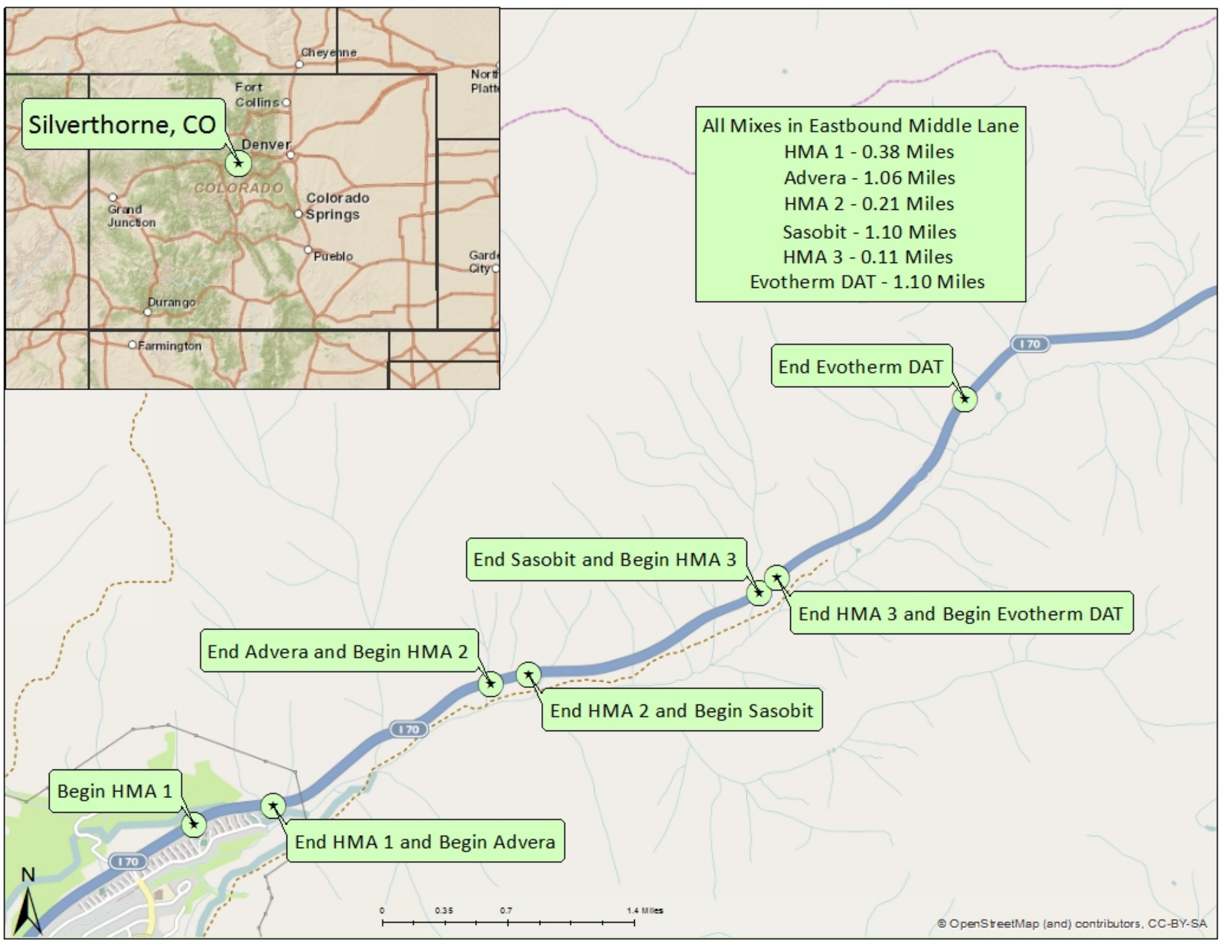


Figure 22 Location of Test Sections in Silverthorne, CO

Construction In-Place Densities

The in-place densities were measured for each section using a nuclear gauge that was correlated to cores. The average in-place densities for each section are shown in Table 37. All densities were acceptable and similar except for the HMA control placed before the Sasobit section. This section had a slightly high density of 95.7 percent. However, only one reading was taken for this mix whereas the others had multiple readings.

Table 37 In-place Densities by Nuclear Gauge in Silverthorne, CO (*24*)

Statistic	Control	Advera	Control	Sasobit	Control	Evotherm
Avg. (%G_{mm})	93.8	93.3	95.7	93.2	93.7	94.7
Number of Tests	4	4	1	4	2	4
Std. Dev. (%G_{mm})	0.21	0.74	N/A	1.03	0.28	0.81

Three-Year (38-Month) Project Inspection

A field performance evaluation was conducted in October 2010 after 38 months of traffic applied to the roadway. Data were collected on each section to document performance regarding rutting, cracking, and raveling. It should be noted that all test sections were placed in the middle lane. The outside lane serves as the truck-climbing lane; this lane was paved entirely with the HMA mix and was not performing very well. This was expected since concentrated truck loading with chained tires historically causes distresses to propagate more rapidly.

The rut depths were measured at the beginning of each 200 ft. (61 m) evaluation section with a straightedge and wedge. Table 38 shows the average rut depths at the time of the three-year inspection. All mixes were performing well at the time of the inspection.

Table 38 Rut Depths for Silverthorne, CO as of October 2010

Mix	Average Rut Depth (mm)
HMA 1	5.0
Advera	4.0
HMA 2	5.0
Sasobit	6.0
HMA 3	8.0
Evotherm DAT	6.0

Each evaluation section was inspected throughout its length for cracking and other distresses. All control HMA and WMA sections had performed well through three years of service. The length, location, and severity of each crack were recorded. The majority of the cracks were transverse cracks. A small area of fatigue cracking observed in the Evotherm DAT section was believed to be reflective cracking from a soft area deeper in the pavement. The only cracking observed in the control HMA sections was in the Evotherm control section, which had some transverse cracking along with one longitudinal crack. Table 39 shows the cracking by crack type and severity for all four mixtures. Figure 23 shows an example of the transverse cracking observed in one of the WMA sections.

Table 39 Cracking Measurements for Silverthorne, CO

Mix Section	Severity	Wheelpath Longitudinal		Non-Wheelpath Longitudinal		Transverse	
		# of Cracks	Total Length, m	# of Cracks	Total Length, m	# of Cracks	Total Length, m
HMA-Advera Control	Low	0	0	0	0	0	0
	Moderate	0	0	0	0	0	0
	High	0	0	0	0	0	0
HMA Sasobit Control	Low	0	0	0	0	0	0
	Moderate	0	0	0	0	0	0
	High	0	0	0	0	0	0
HMA Evotherm Control	Low	1	0.3	5	7.6	0	0
	Moderate	0	0	1	1.5	0	0
	High	0	0	0	0	0	0
Advera	Low	0	0	1	0.3	0	0
	Moderate	0	0	0	0	0	0
	High	0	0	0	0	0	0
Sasobit	Low	0	0	2	0.9	0	0
	Moderate	0	0	0	0	0	0
	High	0	0	0	0	0	0
Evotherm	Low	0	0	0	0	1	5.5
	Moderate	0	0	0	0	0	0
	High	0	0	0	0	0	0

Figure 23 Transverse Cracking in WMA Section in Silverthorne, CO (*24*)

Sand patch tests were conducted at the beginning and end of each evaluation section between the wheelpaths. The sand patch test was also performed on the cores taken during the three-year inspection. The calculated mean texture depths for each mix are shown in Table 40 . Surface textures were similar for all of the sections, but differed somewhat between the in-situ measurements and those taken later on the cores. These results indicate that the pavements were performing well with regard to surface wear in this extreme climate. Figure 24 shows an example of the pavement texture for all mixtures.

Table 40 Mean Texture Depths (mm) for Silverthorne, CO (*24*)

Mix Section	Measured in the Field on the Pavement	Measured in the Laboratory on the Cores (IWP*)	Measured in the Laboratory on the Cores (BWP*)
HMA Control	0.37	0.27	0.30
Advera WMA	0.34	0.24	0.27
Sasobit WMA	0.33	0.29	0.31
Evotherm WMA	0.38	0.25	0.24

*IWP - in the wheel path, BWP - between the wheel paths

Figure 24 Surface Texture of Test Sections in Silverthorne, CO

Core Testing

At the time of the three-year inspection, cores were obtained between the wheelpaths and in the right wheelpath. A summary of the results of tests on the cores is shown in Table 41.The gradations and asphalt contents of the WMA mixes were similar to the HMA at the time of the inspection. The in-place density for the Advera mix was high, greater than 98 percent. The asphalt absorption values and tensile strengths were similar for all mixes.

Table 42 shows the average in-place densities and tensile strengths by location. It can be seen that the in-place densities were very similar for all mixes and were similar in and between the wheelpaths. The Advera mixture had the highest in-place density, approximately 98 percent. The Sasobit mix had slightly lower density as might be expected from the binder stiffening effect of the Sasobit. Tensile strengths were also similar for most of the sections and did not vary substantially for the two locations except for the Sasobit cores taken in the right wheelpath. That set of cores had a slightly lower tensile strength. However, there were no signs of moisture damage or cracking in those cores.

Table 41 Test Results on 38-Month Cores from Silverthorne, CO

Property	HMA	Advera	Sasobit	Evotherm
Sieve Size	% Passing			
19.0 mm (3/4")	100.0	100.0	100.0	100.0
12.5 mm (1/2")	99.2	99.7	99.8	99.5
9.5 mm (3/8")	96.2	97.3	96.1	95.6
4.75 mm (#4)	80.5	79.7	76.9	76.0
2.36 mm (#8)	60.6	58.6	57.7	56.3
1.18 mm (#16)	45.5	43.9	43.6	42.4
0.60 mm (#30)	31.5	31.1	30.9	29.9
0.30 mm (#50)	20.4	20.6	20.3	19.9
0.15 mm (#100)	12.5	12.8	12.5	12.7
0.075 mm (#200)	7.3	7.7	7.3	7.9
Asphalt Content (%)	6.46	6.59	6.65	6.27
G_{mm}	2.445	2.434	2.435	2.444
G_{mb}	2.379	2.387	2.351	2.369
In-place Density (%)	97.3	98.1	96.5	96.9
P_{ba} (%)	0.40	0.32	0.41	0.29
Tensile Strength (psi)	62.8	60.2	56.1	60.8

Table 42 In-Place Density and Tensile Strength by Location in Silverthorne, CO

Location and Property	HMA	Advera	Sasobit	Evotherm
Between Wheelpaths Density (%)	97.7	98.3	96.1	96.8
In Right Wheelpath Density (%)	96.7	97.8	97.1	97.1
Between Wheelpaths Tensile Strength (psi)	62.5	61.8	62.8	57.4
In Right Wheelpath Tensile Strength (psi)	60.0	58.7	49.4	64.2

Franklin, Tennessee

This WMA trial project was placed on Tennessee State Road 46 (SR-46) near Franklin, Tennessee. SR-46 is a two-lane roadway with mostly automobile traffic *(17).* The AADT for this portion of SR-46 was 10,492. The Tennessee DOT (TDOT) performed a pavement condition survey before the WMA trial project was constructed. The existing asphalt surface was cracked with crack sealant applied to several locations. The TDOT pavement condition survey is summarized in Table 43.

Table 43 Existing Pavement Condition Survey for Franklin, TN (*17*)

Beginning Mile	End Mile	Roughness Index (PSI)	IRI (in./mi)	Rut Depth (mm)	Distress Index (PDI)	Pavement Quality Index (PQI)
0	1	2.31	146.3	3.8	5.00	3.97
1	2	2.47	129.9	4.1	5.00	4.04
2	3	2.91	100.0	3.6	4.88	4.18
3	4	3.11	87.8	3.8	4.97	4.32
4	5	3.03	91.8	3.8	4.97	4.28
5	5.64	2.71	118.9	4.3	4.84	4.07

The project consisted of a 1.25-inch overlay. The contractor for the project was LoJac Inc. Six different mixes, two HMA and four WMA, were produced out of three different nearby plants. One of the HMA mixes, the Advera mix, and the Sasobit mix were produced at the LoJac plant in Franklin. Each of these mixtures used the same 75-blow Marshall Mix design with a 12.5 mm NMAS gradation. A second HMA was produced at LoJac's Danley plant along with the Evotherm DAT mixture. Finally, the Astec DBG mixture was produced at the LoJac Murfeesboro plant. Although separate mix designs were completed for the Danley and Murfeesboro plants, the designs were essentially the same. The three mix designs used the same aggregate percentages with no RAP. The only difference was that the limestone D-Rock source for the Franklin plant was from Bon Aqua, TN, while the other two plants used D-Rock from Springfield, TN. The PG 70-22 asphalt binder produced by Ergon Asphalt and Emulsions Inc. was used for all mixes. Table 44 shows the aggregate stockpile percentages. Table 45 shows the design aggregate gradations, asphalt contents, and volumetric properties for all three designs.

Table 44 Aggregate Percentages for the Franklin, TN WMA Project Mixes

Aggregate Type	% of Total Aggregate		
	Murfreesboro Plant	Franklin Plant	Danley Plant
Limestone D-Rock	50	50	50
#10 Screenings	10	10	10
Natural Sand	25	25	25
#10 Washed Screenings	15	15	15

Table 45 Design Gradations and Asphalt Contents for Franklin, TN

Property	Murfreesboro Plant	Franklin Plant	Danley Plant
Sieve Size	% Passing		
19.0 mm (3/4")	100	100	100
12.5 mm (1/2")	99	98	99
9.5 mm (3/8")	85	86	85
4.75 mm (#4)	59	56	59
2.36 mm (#8)	46	41	46
0.6 mm (#30)	26	24	26
0.3 mm (#50)	10	10	10
0.15 mm (#100)	6	6	6
0.075 mm (#200)	4.0	4.1	4.0
AC (%)	5.3	5.3	5.3
D/A Ratio	0.75	0.77	0.75
G_{mm}	2.428	2.415	2.428

Production

The two HMA mixtures were placed prior to the WMA sections on October 1, 2007. The placement of the two HMA mixtures was not observed by NCAT. However, notes from the contractor show that the mixture was produced at approximately 320°F and no problems were encountered during construction.

On October 2, the Astec DBG mixture was produced at the Murfreesboro plant using 0.1 percent water by total weight of mix. The mixture also contained an anti-striping agent, Pavegrip 650, at a rate of 0.3 percent by weight of asphalt. Approximately 775 tons were produced at an average production rate of 250 tons per hour. The target production temperature was 260°F.

The Advera mixture was produced and placed on October 3, 2007 from the Franklin plant, which is an Astec Double Barrel plant. Advera was introduced into the plant at a rate of 0.3 percent by weight of total mix by a pneumatic system that fed the additive into the outer mixing drum. Approximately 1,150 tons of the Advera mixture was produced at a rate of 250 tons per hour. The target production temperature was 250°F.

The Evotherm DAT mixture was produced on October 4, 2007 from the Danley plant, another Astec Double-Barrel plant. The target production temperature was 230°F.The Sasobit mixture was produced on October 5, 2007 from the Franklin plant. The Sasobit was added at 1.5 percent by weight of asphalt. Approximately 750 tons of the Sasobit mix were produced at a target production temperature of 230°F. All three Franklin mixes contained the anti-stripping agent AD-Here 77-00 at a rate of 0.3 percent by weight of asphalt. Table 46 shows a summary of production temperatures and facilities for all mixtures included in this project.

Table 46 Summary of Mixtures for the Franklin, TN WMA Project

Mixture	Production Temperature	Production Facility	Aggregate Source
HMA 1	320°F	Franklin	Bon Aqua, TN
Advera	250°F	Franklin	Bon Aqua, TN
Sasobit	250°F	Franklin	Bon Aqua, TN
HMA 2	320°F	Danley	Springfield, TN
Evotherm DAT	240°F	Danley	Springfield, TN
Astec DBG	260°F	Murfeesboro	Springfield, TN

Volumetric Mix Properties

Mixes were sampled during production to fabricate volumetric samples to compare air void contents. All WMA mix samples were compacted on-site in the NCAT mobile lab to avoid reheating. The two HMA mix samples were compacted from reheated mix. A lab compactive effort of 60 gyrations was used since the state of Tennessee still uses the Marshall mix design method instead of the Superpave mix design method. The mixes were extracted in accordance with AASHTO T 319. Table 47 shows the average air void contents of the lab compacted samples, the extracted gradations and asphalt contents. The gradations and asphalt contents for all mixes were similar. Minor differences in the air void contents among the mixtures are probably attributed to material variations of the mixtures and the differences in sample preparation (hot compacted versus reheated).

Table 47 Tested Gradations, Asphalt Contents, and Air Voids for Franklin, TN

Property	HMA 1	Advera	Sasobit	HMA 2	Evotherm DAT	Astec DBG
Sieve Size	% Passing					
19.0 mm (3/4”)	100	100	100	100	100	100
12.5 mm (1/2”)	97	97	98	98	98	98
9.5 mm (3/8”)	84	85	84	88	83	86
4.75 mm (#4)	57	58	52	60	55	57
2.36 mm (#8)	46	42	40	44	43	43
1.18 mm (#16)	37	32	30	33	34	33
0.60 mm (#30)	28	24	22	24	25	24
0.30 mm (#50)	10	10	8	10	10	10
0.15 mm (#100)	6	6	4	5	6	6
0.075 mm (#200)	4.5	5.2	4.1	4.4	5.1	5.1
Asphalt Content (%)	5.2	5.1	4.9	5.3	4.9	4.8
Air Voids (%)	2.7	3.1	3.9	3.0	3.4	2.9

Construction

The average compaction temperature for all four WMA mixtures was 230°F. The approximate haul times from the three plants were 10, 25, and 45 minutes for the Franklin, Danley, and Murfreesboro plant respectively. Figure 25 shows the test section layout for the site.

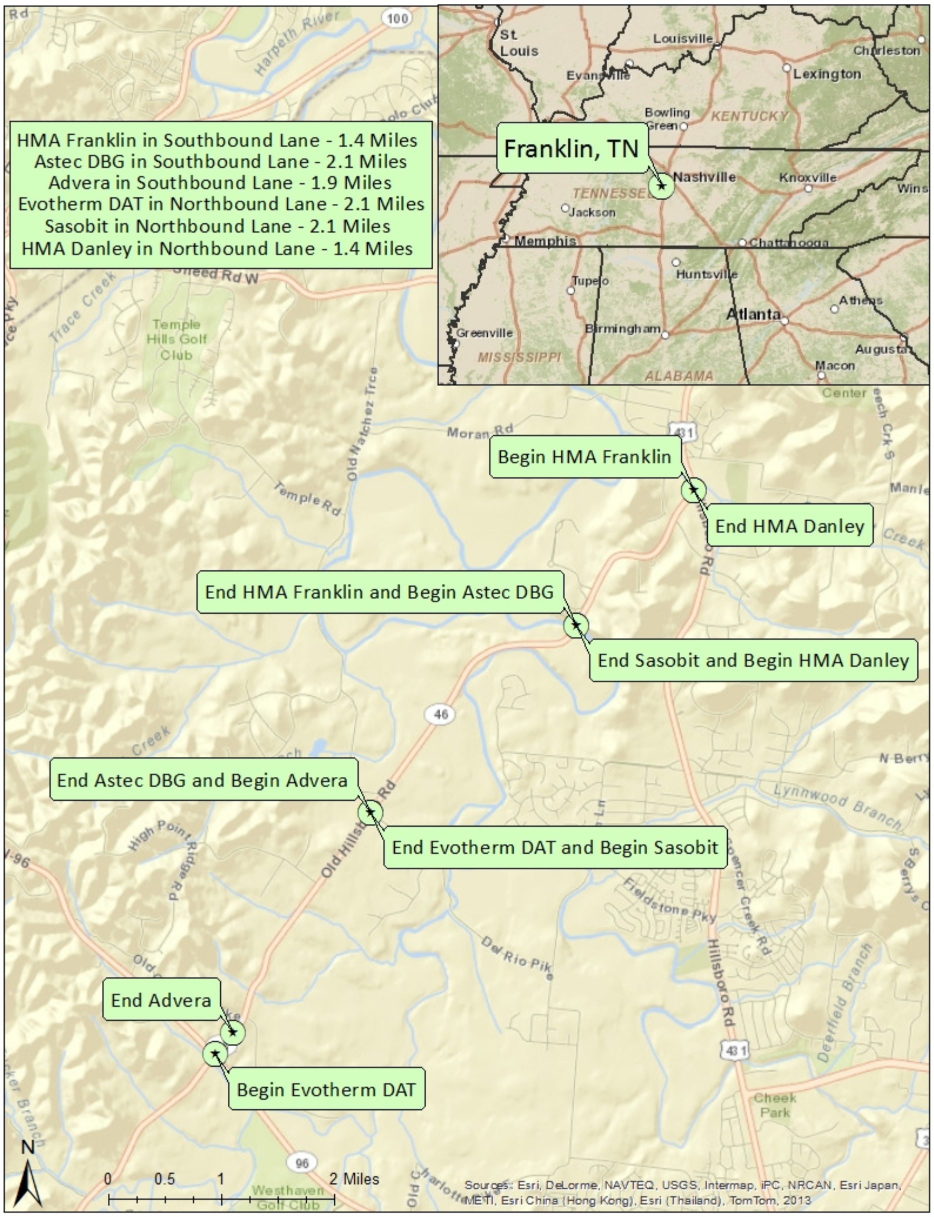


Figure 25 Location of Test Sections in Franklin, TN

Construction Core Testing

Cores were taken by the contractor immediately following construction from each section and tested to determine densities in accordance with AASHTO T166. These initial cores were taken at the beginning of each test section. The density results for the WMA cores were quite low, so the contractor obtained a second set of cores. The low density in the first set of cores may be due to their proximity to the beginning of the section. The number of cores in the second set was decided by the contractor and varied from section to section, ranging from two to ten. The Astec DBG, Advera, Evotherm DAT, and Sasobit sections had ten, five, four, and two cores respectively. A set of 10 cores was taken from both HMA sections. Table 48 shows a summary of the density results for each set of cores. Although the densities of the WMA sections were low for the initial set of cores, the second set indicated that the in-place density results for the WMA sections were consistent with the density results for the HMA sections.

Table 48 In-Place Density Results (% of G_{mm}) for Franklin, TN

Set	Statistic	HMA 1	Advera	Sasobit	HMA 2	Evotherm DAT	Astec DBG
Set #1	Avg.	92.1	89.0	90.3	93.0	90.4	87.0
	Std. Dev.	1.4	1.2	1.6	1.4	1.1	1.1
Set #2	Avg.	--	93.0	92.2	--	91.2	91.9
	Std. Dev.	--	0.6	0.5	--	2.4	0.6

3-Year (41-Month) Project Inspection

A field-performance evaluation was conducted on March 11, 2011 after about 41 months of traffic. Data were collected on each section to document performance regarding rutting, cracking, and raveling. Rut depths were measured at the beginning of each evaluation section with a straight edge and a wedge. Table 49 shows the average and standard deviations of the rut depth measurements for each section. None of the sections have a significant amount of rutting, which was expected since this roadway experiences mostly light vehicle traffic.

Table 49 Rut Depths for Franklin, TN

Mix	Average Rut Depth (mm)	Standard Deviation (mm)
HMA Franklin	0.0	0.0
HMA Danley	0.0	0.0
Advera	0.5	0.5
Astec DBG	0.4	0.6
Evotherm DAT	0.0	0.0
Sasobit	0.0	0.0

Each 200 ft. (61 m) evaluation section was carefully inspected for cracking. Although all six test sections had some cracking, it was all low severity. Table 50 shows the total cracking by crack type. The Sasobit and Advera section showed the most cracking, and the Evotherm was the only section to exhibit fatigue cracking. However, fatigue cracking had been documented in the existing pavement where the Evotherm WMA was placed.

Table 50 Cracking Measurements for Franklin, TN

Mix Section	Wheelpath Longitudinal		Non-Wheelpath Longitudinal		Transverse		Fatigue	
	# of Cracks	Total Length, m	# of Cracks	Total Length, m	# of Cracks	Total Length, m	# of Cracks	Total Length, m
HMA 1	2	11.0	0	0	0	0	0	0
HMA 2	4	7.3	0	0	0	0	0	0
Advera	5	16.8	6	25.9	1	0.9	0	0
Astec DBG	2	6.1	0	0	4	11.6	0	0
Evotherm	1	12.5	0	0	0	0	2	13.7
Sasobit	7	57.9	2	29.0	3	2.0	0	0

Figure 26 shows an example of the wheelpath longitudinal cracking observed in all mix sections. Figure 27 shows the fatigue cracking observed in the Evotherm section.

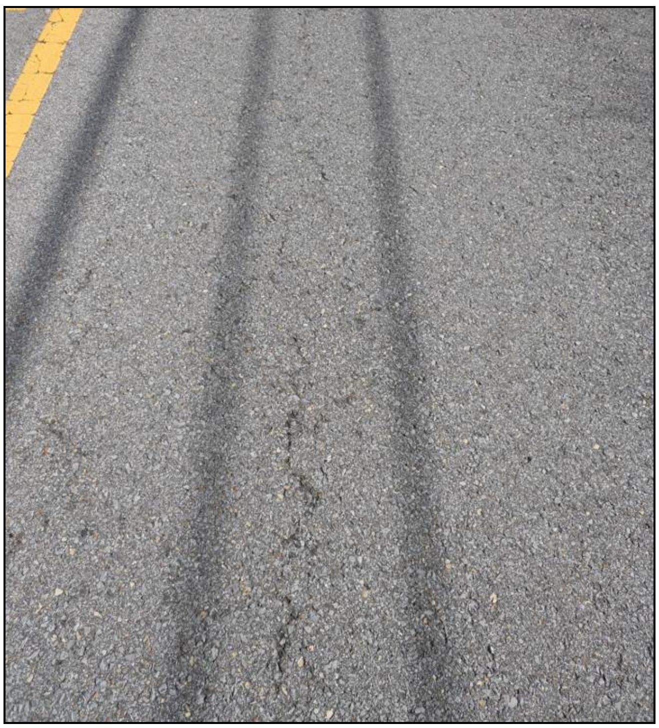


Figure 26 Example of Wheelpath Longitudinal Cracking in Franklin, TN

Figure 27 Fatigue Cracking in Evotherm Section in Franklin, TN

Sand patch tests were conducted at the beginning of each evaluation section in the right wheelpath. The results of the sand patch tests are shown in Table 51. Based on the magnitude of the texture depths, these sections are showing significant raveling. In addition, based on visual observations in the field, all six mix sections had weathered significantly. However, all mixes looked to have experienced the same amount of weathering. Figure 28 shows an example of the surface texture of the mix sections in Franklin, Tennessee.

Figure 28 Example of Surface Texture in Franklin, TN

Table 51 Mean Texture Depths for Franklin, TN

Mix	Mean Texture Depth (mm)	Standard Deviation (mm)
HMA 1	0.94	0.02
Advera	1.01	0.05
Sasobit	0.99	0.10
HMA 2	0.82	0.02
Evotherm DAT	0.77	0.09
Astec DBG	0.78	0.01

Core Testing

At the time of the three-year project inspection, seven 6-inch (150-mm) cores were taken from each mix section similar to previous projects. During tensile strength testing, two of the between wheelpath cores from the HMA 2 (Danley) and Advera sections broke incorrectly because they were too thin. Instead of fracturing, the tops of the samples were simply crushed. All of these cores from this project were very thin, but these were the only four that failed in this manner.

A summary of the results of the core tests are shown in Table 52. It can be seen that there were significant variations in gradations and asphalt contents among the results for the different sections. The dust content varies from 5.8 to 9.7 percent, while the asphalt content varies from 4.50 to 5.38 percent. The in-place densities were low for all mixes except the first HMA mix.

These low densities were similar to the results of the initial cores obtained after construction and indicate that the test sections were likely not well compacted during construction. This would have contributed to the raveling previously noted. The tensile strengths of the WMA were higher than for the two HMA mixes. These results may have been affected by the thin cores, but can also indicate the binder in the WMA sections were aging at a faster rate due to the low densities.

Table 52 Test Results from Franklin, TN Three-Year Cores

Property	HMA 1	Advera	Sasobit	HMA 2	Evotherm DAT	Astec DBG
Sieve Size	% Passing					
19.0 mm (3/4")	100.0	100.0	100.0	100.0	100.0	100.0
12.5 mm (1/2")	97.4	98.3	97.2	97.0	94.9	97.2
9.5 mm (3/8")	84.8	83.5	84.3	84.3	79.9	83.9
4.75 mm (#4)	55.2	52.3	52.4	54.6	52.0	58.5
2.36 mm (#8)	40.3	38.2	39.1	42.5	39.3	43.7
1.18 mm (#16)	31.5	30.6	31.4	34.9	31.2	34.2
0.60 mm (#30)	23.7	24.3	24.2	27.6	23.3	25.5
0.30 mm (#50)	11.1	14.3	11.0	11.8	11.5	12.7
0.15 mm (#100)	7.1	10.9	7.4	7.4	8.1	8.6
0.075 mm (#200)	5.8	9.7	6.3	6.0	7.0	6.9
Asphalt Content (%)	5.38	4.50	4.61	4.92	4.53	5.02
Avg. Production Temp. (°F)	320	250	250	320	250	250
G_{mm}	2.444	2.475	2.465	2.467	2.476	2.476
G_{mb}	2.306	2.191	2.128	2.192	2.180	2.201
In-place Density (%)	94.3	88.5	86.3	88.9	88.0	88.9
Tensile Strength (psi)	122.9	162.2	152.9	139.3	176.3	156.9

Table 53 shows the average density and tensile strength results by location for the 41-month cores. In general, densities were similar for the cores taken in and between the wheelpaths. Tensile strengths were also similar for the cores taken in and between the wheelpaths.

Table 53 In-Place Density and Tensile Strength by Location for Franklin, TN Three-Year Inspection

Location and Property	HMA 1	Advera	Sasobit	HMA 2	Evotherm DAT	Astec DBG
Between Wheelpaths Density (%)	93.9	88.5	86.0	87.5	86.6	89.4
In Right Wheelpath Density (%)	95.0	88.6	86.8	90.6	89.9	88.2
Between Wheelpaths Tensile Strength (psi)	107.4	173.1	150.8	153.5	168.5	150.8
In Right Wheelpath Tensile Strength (psi)	138.3	158.6	155.0	134.5	184.1	163.1

Graham, Texas

A field trial was placed north of Graham, Texas on Texas State Highway 251 in June 2008 by RK Hall Construction Ltd, Paris, TX. The trial sections were placed north of the intersection of Broadway Avenue on SH 251 in New Castle. The project consisted of placing a test WMA mixture along with a control HMA mixture. The HMA was placed in the northbound lane and the WMA was placed in the southbound lane. The average annual daily traffic for this portion of SH 251 was 1,171 with 10.9 percent trucks. Both mixes consisted of a two-inch overlay on existing pavement.

The WMA technology used for this trial evaluation was the Astec DBG foaming process. The mix design, which consisted of fine-graded 9.5 mm nominal maximum aggregate size mixture, was the same for both mixtures. A PG 70-22 binder was used for both mixtures with the addition of one percent Kling-Beta 2550HM manufactured by Akzo Nobel as an anti-stripping agent. No RAP was used in either mixture, and the aggregate type was limestone. The aggregate stockpile percentages for both mixes are shown in Table 54, and the design aggregate gradation and volumetrics are shown in Table 55.

Table 54 Aggregate Percentages for Graham, Texas

Aggregate Type	% of Total Aggregate
Type D Rock	48
Type F Rock	15
C-33	21
Manufactured Sand	9
Kreel Sand	6
Lime	1

Table 55 Design Gradation, Asphalt Content and Volumetrics for Graham, Texas

Property	JMF
Sieve Size	% Passing
12.5 mm (1/2")	100
9.5 mm (3/8")	97.2
4.75 mm (#4)	69.7
2.36 mm (#8)	38.7
1.18 mm (#16)	--
0.60 mm (#30)	17.4
0.30 mm (#50)	12.2
0.15 mm (#100)	--
0.075 mm (#200)	4.5
AC (%)	5.3
Air Voids (%)	3.0
VMA (%)	15.3
VFA (%)	80.4
G_{mm}	2.459

Production

The HMA mixture was produced at temperatures between 320°F and 335°F, while the WMA was produced between 275°F and 290°F. The asphalt plant used to produce both mixes was a portable Astec DBG plant that was located approximately two miles east of the test sections on US 380. The plant can be seen in Figure 29. Figure 30 shows the Astec DBG drum. The point of water injection can be seen at the top of the drum.

Figure 29 Portable Asphalt Plant used for Graham, TX Project

Figure 30 Drum and Point of Water Injection in Graham, TX Plant

Construction

The asphalt mixtures were delivered to the site in live bottom trucks and then transferred into a RoadTec 2500 material transfer device. The haul distance from the plant to the portion of the trial section observed by NCAT was between two to seven miles. Figure 31 shows the location of the test sections in Graham, Texas.

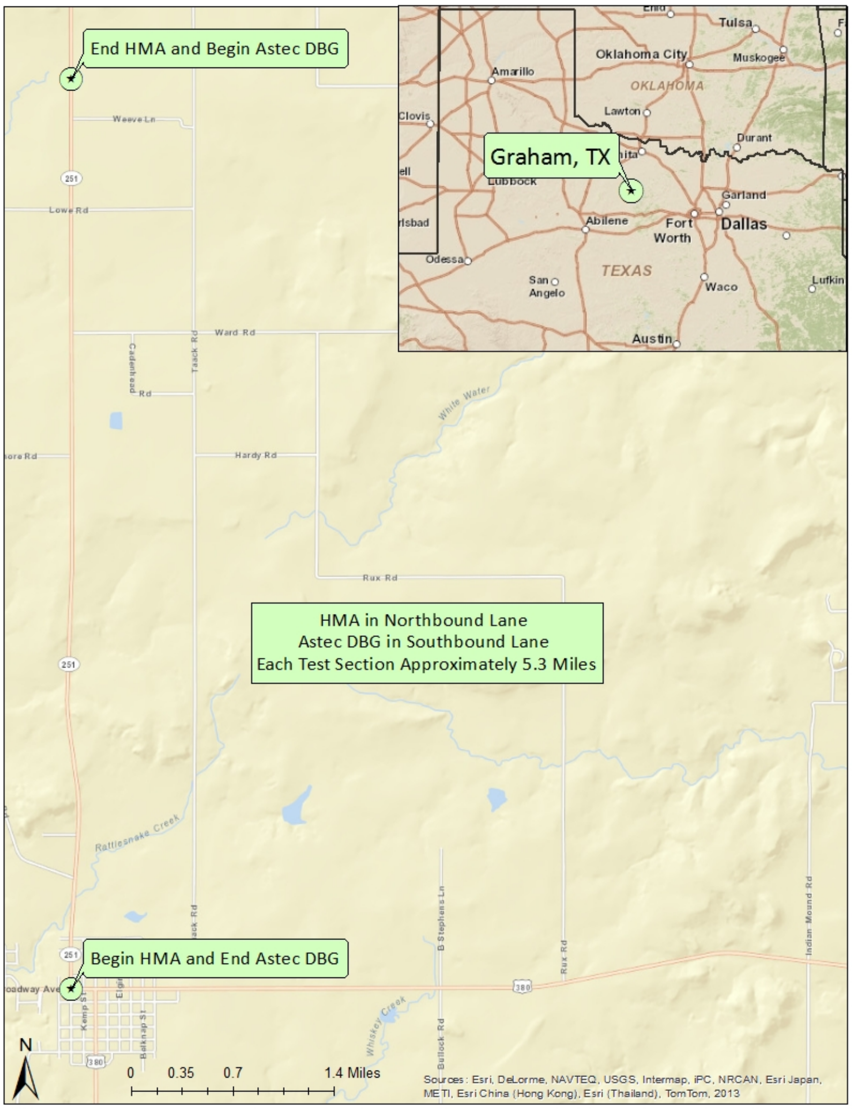


Figure 31 Location of Test Sections in Graham, TX

The material transfer device transferred the mixes into a 2005 RoadTec 190 paver. Figure 32 shows the material transfer device and paver used for both trial mixtures. Two rollers were used for both mixtures: a Caterpillar 634 double drum and a 25-ton Dynapac pneumatic roller.

Figure 32 Material Transfer Device and Paver used for Graham, TX Project

3-Year (30-Month) Project Inspection

A field performance evaluation was conducted on December 9, 2010, after about 30 months of traffic were applied to the test sections. Data were collected on the WMA and HMA sections to document performance regarding rutting, cracking, and raveling within three evaluation sections as described previously.

Rut depths were measured at the beginning of each evaluation section using a straightedge and wedge. Neither section had any measurable rutting after 30 months of traffic had been applied to the overlay.

Each evaluation section was carefully inspected for signs of cracking. Both the HMA and WMA sections had small amounts transverse reflective cracking. Cores were taken on some of the cracks to verify that they were reflective cracks as shown in Figure 33. Table 56 shows the total cracking by crack type and severity for both mixes. It can be seen that the amount of low severity cracking in the two different mix sections was comparable. However, the WMA mix sections also had some moderate cracking. Figure 34 show an example of the transverse cracks after 30 months of performance.

Figure 33 Photo Showing Core Taken on a Transverse Crack to demonstrate it was Reflecting from Underlying Pavement Layers

Table 56 Cracking Measurements for Graham, TX

Mix Section	Severity	Wheelpath Longitudinal		Non-Wheelpath Longitudinal		Transverse		Fatigue	
		# of Cracks	Total Length, m	# of Cracks	Total Length, m	# of Cracks	Total Length, m	# of Loca-tions	Total Area, m^2
HMA	Low	0	0	0	0	9	17.7	0	0
	Mod	0	0	0	0	0	0	0	0
	High	0	0	0	0	0	0	0	0
Astec DBG	Low	0	0	0	0	4	10.2	0	0
	Mod	0	0	0	0	4	14.6	0	0
	High	0	0	0	0	0	0	0	0

Figure 34 Photos of transverse cracks on the Graham, TX project after 30 months

The calculated mean texture depths from sand patch tests are shown in Table 57. These data indicate that the two mixes have performed comparably in terms of mean texture depth after three years.

Table 57 Mean Texture Depths for Graham, TX

Mix	Mean Texture Depth (mm)	Standard Deviation (mm)
HMA	0.93	0.06
Astec DBG	1.06	0.03

Core Testing

At the time of the three-year project inspection, cores were taken from both sections for analysis of densities, tensile strengths, gradations, asphalt contents, and recovered binder properties. A summary of the core testing is shown in Table 58. It can be seen that the average asphalt contents and gradations for the two mixes were very similar, as were the average tensile strengths. The in-place density for the WMA was slightly lower compared to the HMA. However, the difference could possibly be accounted for by material and sampling variability. Both mixes have performed equally after three years.

Table 59 shows the average densities and tensile strengths by location for the 30-month inspection cores. The HMA cores in the wheelpath were slightly denser than the cores from between the wheelpaths as expected. The WMA had similar densities for both locations. Tensile strengths do not appear to be affected by location.

Table 58 Test Results from Thirty-Month Cores from Graham, TX

Property	HMA	Astec DBG
	30-Month Cores (December 2010)	
Sieve Size	% Passing	
12.5 mm (1/2”)	100.0	100.0
9.5 mm (3/8”)	97.5	97.7
4.75 mm (#4)	71.9	71.3
2.36 mm (#8)	37.8	40.0
1.18 mm (#16)	25.1	26.8
0.60 mm (#30)	17.9	19.3
0.30 mm (#50)	12.9	14.0
0.15 mm (#100)	7.3	8.1
0.075 mm (#200)	4.9	5.3
Asphalt Content (%)	4.80	4.78
G_{mm}	2.480	2.476
G_{mb}	2.380	2.335
In-place Density (%)	96.0	94.3
Tensile Strength (psi)	257.9	255.9

Table 59 In-Place Density and Tensile Strength by Location for Graham, TX Thirty-Month Cores

Location and Property	HMA	Astec DBG
Between Wheelpaths Density (%)	95.2	94.4
In Right Wheelpath Density (%)	97.0	94.2
Between Wheelpaths Tensile Strength (psi)	263.9	247.3
In Right wheelpath Tensile Strength (psi)	251.9	264.4

George, WA

A field trial was placed in the right lane of I-90 eastbound in June 2008 to evaluate the WMA additive Sasobit *(25)*. HMA was also placed as the control mixture for this field evaluation. The project was located west of the town of George between the Columbia River at milepost 137.82 and the town of George at milepost 148.45. This portion of I-90 consists of two lanes and a paved shoulder in both directions, and has an average daily traffic (ADT) between 6,448 and 7,327 with 27 percent trucks according to data from the 2008 Washington State Pavement Management System. The contractor for this project was Central Washington Asphalt Inc., Moses Lake, WA. The existing pavement in the right travel lane had low severity alligator and transverse cracking. The rehabilitation for this project included milling three inches of the existing pavement and replacing with the same depth of HMA or WMA.

The WMA additive used for this field evaluation was the organic additive Sasobit. The mix designs of the two mixtures were identical except for the addition of the Sasobit in the WMA mixture. The mix design consisted of a 12.5 mm NMAS mix designed with a 100-gyration compactive effort according to the Superpave mix design procedure. The mix also called for 20 percent RAP. However, in the state of Washington, RAP is not used in the design process. The RAP used for this project came from the three-inches of milling on the project prior to the overlay. A PG 76-28 asphalt binder was used for both mixtures. Table 60 shows the aggregate percentages used in mix design and production. Table 61 shows the design aggregate gradation and volumetric properties for both mixes.

Table 60 Aggregate Percentages for George, WA

Aggregate Type	% of Total Aggregate	
	Design	Production
¾" - #4	27	27
⅜" – 0	73	53
RAP	0	20

Table 61 Design Gradation, Asphalt Content and Volumetrics for George, WA

Property	JMF
Sieve Size	% Passing
19.0 mm (3/4")	100.0
12.5 mm (1/2")	95.0
9.5 mm (3/8")	84.0
4.75 mm (#4)	55.0
2.36 mm (#8)	34.0
1.18 mm (#16)	22.0
0.60 mm (#30)	15.0
0.30 mm (#50)	11.0
0.15 mm (#100)	8.0
0.075 mm (#200)	6.3
AC (%)	5.5
Air Voids (%)	3.7
VMA (%)	14.9
VFA (%)	75.0
P_{be} (%)	4.7%
P_{ba} (%)	0.91
G_{mm}	2.577
G_{mb}	2.482

Production

The Sasobit was added at a rate of 2 percent by weight of virgin binder. With the inclusion of the 20 percent RAP, the Sasobit had an effective addition rate of 1.6 percent by total weight of binder. The Sasobit was added to the virgin binder prior to shipping. Approximately 4,724 total tons of the WMA mixture was produced between June 23 and June 24, 2008. The average production temperature of the WMA mixture was approximately 290°F. Approximately 7,813 tons of the HMA mixture was produced between June 11 and June 16, 2008. The average mixing temperature was 330°F, about 40°F higher than the WMA. Both mixtures were produced using a portable drum plant manufactured by Gencor.

Volumetric Mix Properties

Volumetric and gradation data was compiled from the results of the QC tests performed on the nine HMA sublots and five WMA sublots. All gradation tests were in tolerance. The air void levels on two of the HMA lots were out of tolerance. Both were 5.7 percent air voids, which was out of the tolerance band of 2.5 to 5.5 percent. In addition, the dust to asphalt ratio on one of the HMA sublots was 1.7, just above the limit of 1.6. This same dust to asphalt ratio of 1.7 was seen on one of the WMA sublots as well. All other properties from the 14 sublot tests were in tolerance. Table 62 shows the average results of these tests for both mixtures.

Table 62 Gradation, Asphalt Content, and Volumetrics for Production Mix in George, WA

Property	JMF	HMA	Sasobit	Tolerance Limit
Sieve Size	% Passing			
19.0 mm (3/4”)	100.0	100.0	100.0	99 - 100
12.5 mm (1/2")	95.0	93.8	95.2	90 – 100
9.5 mm (3/8")	84.0	83.1	85.0	78 – 90
4.75 mm (#4)	55.0	54.1	55.2	51 – 61
2.36 mm (#8)	34.0	34.2	35.0	31 – 39
1.18 mm (#16)	22.0	22.1	22.4	--
0.60 mm (#30)	15.0	15.3	15.8	--
0.30 mm (#50)	11.0	11.4	12.0	--
0.15 mm (#100)	8.0	8.7	9.0	--
0.075 mm (#200)	6.3	6.4	6.7	4.3 – 7.0
AC (%)	5.2	5.1	5.4	4.7 – 5.7
Air Voids (%)	3.7	4.9	4.5	2.5 – 5.5
VMA (%)	14.9	14.8	14.7	12.5 min.
VFA (%)	75.0	67.2	69.4	--
D/A	1.4	1.5	1.6	0.6 – 1.6

Construction

The HMA was placed between mileposts 137.82 and 144.53, while the WMA was placed between mileposts 144.53 to 148.45. Haul times ranged from 30 to 45 minutes for the HMA and 25 to 35 minutes for the WMA. Figure 35 shows the location of the test sections.

Figure 35 Location of Test Sections in George, WA

The mixtures were delivered to the site in uncovered, end-dump trailers. The trucks dumped the mixtures into a windrow device and a windrow was created. A windrow elevator was the used to transfer the mix from the windrow to the Ingersoll-Rand PF-5510 paver. This paver was equipped with an Omni 3E screed. Mix delivery was sometimes inconsistent which lead to several paver stops. Otherwise, the placement of both mixtures went smoothly. Figure 36 shows the windrowed material being transferred to the paver, and Figure 37 shows the paver laying down the mix.

Figure 36 Windrow Elevator Transferring Mix to Paver in George, WA

Figure 37 Paver Spreading Mix in George, WA

Paving temperatures were measured and recorded for the HMA and WMA mixtures on June 16 between 9:30 a.m. and 11:30 a.m. and on June 23 between 8:00 a.m. and 10:30 a.m. respectively.

Table 63 shows the temperatures measured on these two days. It can be seen that there were differences from 30 to 50°F between the HMA and WMA.

Table 63 Temperatures On-Site for George, WA (*25*)

Location	Average Temperature (°F)	
	HMA	Sasobit
Leaving Truck	328	286
Windrow Elevator	322	272
Paving Machine Augers	306	276

In-Place Densities after Construction

Density tests were conducted on both mixtures following construction. For the HMA, 95 total density tests were completed. Of these, six failed the required minimum of 91.0 percent density. For the WMA, only one of the 55 tests failed to reach the minimum density requirement. This yields 6.3 and 1.8 percent failing the density requirements for the HMA and WMA respectively. Table 64 shows the results of these density checks.

Table 64 In-Place Density Results for George, WA

Property	Statistic	HMA	Sasobit
In-place Density (%)	Average	93.5	93.7
	Standard Deviation	1.58	1.36

4-Year (50-Month) Project Inspection

A field performance evaluation was conducted on August 27, 2012, after about 50 months of traffic were applied to the test sections. Data were collected on each section to document performance regarding rutting, cracking, and raveling. Rut depths were measured at the beginning of each evaluation section using a string line. The average results from these rutting measurements are shown in Table 65. It can be seen that both mixes show similar rut depths, with the WMA section being only slightly more rutted. Overall, both mixes have performed well in terms of rutting.

Table 65 Rut Depths for George, WA

Mix	Average Rut Depth (mm)	Standard Deviation (mm)
HMA	5.6	0.8
Sasobit	6.0	0.3

Each 200 ft. (61m) evaluation section was carefully inspected for visual signs of cracking. Minimal cracking was evident in each mixture section. The only type of cracking observed was transverse cracking that looked to be reflective cracking since it propagated across all lanes, not just the test lanes. However, this possible cause was not verified with cores. Table 66 shows the total cracking by crack type and severity for both mixtures. Figure 38 shows an example of the transverse cracking seen in both mix sections.

Table 66 Cracking Measurements for George, WA

Mix Section	Severity	Wheelpath Longitudinal		Non-Wheelpath Longitudinal		Transverse		Fatigue	
		# of Cracks	Total Length, m	# of Cracks	Total Length, m	# of Cracks	Total Length, m	# of Loca-tions	Total Area, m^2
HMA	Low	0	0	0	0	9	24.7	0	0
	Mod	0	0	0	0	0	0	0	0
	High	0	0	0	0	0	0	0	0
Sasobit	Low	0	0	0	0	5	3.7	0	0
	Mod	0	0	0	0	0	0	0	0
	High	0	0	0	0	0	0	0	0

Figure 38 Example of Transverse Cracking in George, WA

The surface texture of each mixture was measured using the sand patch test according to ASTM E965. The sand patch test was conducted at the beginning of each evaluation section in the right wheelpath. The calculated mean texture depths for each mix are shown in Table 67. These values represent the average and standard deviation of the three tests conducted on each mix. Based on the results of the sand patch tests, both mixes have raveled significantly. Both mixes have performed equally in terms of mean texture depth after four years. Figure 39shows an example of the surface texture of the mixes.

Table 67 Mean Texture Depths for George, WA

Mix	Mean Texture Depth (mm)	Standard Deviation (mm)
HMA	1.04	0.12
Sasobit	1.09	0.01

Figure 39 Example of Surface Texture in George, WA

Core Testing

At the time of the 50-month project inspection, seven 6-inch (150-mm) cores were taken from each mix section. The cores were first tested for density according to AASHTO T 166 and then tested for tensile strength using ASTM D6931 and then combined and the cut-faces were removed. This mix was split into two samples that were used to determine the maximum specific gravity according to AASHTO T 209. These same two samples were then dried and extracted according to AASHTO T 164. A summary of the core testing is shown in Table 68. The two mixes exhibited similar gradations, except for the dust content, which was 0.5 percent lower for the WMA. However, the asphalt content of the WMA was 0.38 percent higher than the HMA. The higher asphalt content along with the fact that WMA typically yields higher densities than HMA even at the lower temperatures, probably led to the slightly higher in-place density for the WMA compared to the HMA. The binder absorption and tensile strengths of the WMA are all comparable to the HMA.

Table 68 Test Results from Four-Year Cores from George, WA

Property	HMA	Sasobit
Sieve Size	% Passing	
25.0 mm (1”)	100.0	100.0
19.0 mm (3/4”)	99.5	100.0
12.5 mm (1/2”)	95.0	93.3
9.5 mm (3/8”)	81.8	82.0
4.75 mm (#4)	51.9	53.9
2.36 mm (#8)	33.6	35.0
1.18 mm (#16)	21.6	22.0
0.60 mm (#30)	15.1	15.1
0.30 mm (#50)	11.1	10.8
0.15 mm (#100)	8.4	7.9
0.075 mm (#200)	6.0	5.5
Asphalt Content (%)	4.91	5.29
G_{mm}	2.614	2.601
G_{mb}	2.501	2.505
In-place Density (%)	95.7	96.3
P_{ba} (%)	1.10	1.15
Tensile Strength (psi)	188.6	174.8

Table 69 shows the average densities and tensile strengths by location for the four-year inspection cores. The wheelpath cores actually show slightly lower densities than the cores from between the wheelpaths, which was not expected. However, the difference is very small and can be attributed to sampling and material variability.

Table 69 In-Place Density and Tensile Strength by Location for George, WA

Location and Property	HMA	Sasobit
Between Wheelpaths Density (%)	96.0	96.5
In Right Wheelpath Density (%)	95.3	96.1
Between Wheelpaths Tensile Strength (psi)	187.0	148.9
In Right Wheelpath Tensile Strength (psi)	190.2	200.7

New Projects

Walla Walla, Washington

A WMA field evaluation was placed on US-12 in Walla Walla, Washington in April 2010. The WMA technology used on this project was an asphalt foaming system using water injection developed by Maxam Equipment. This WMA technology is referred to by the trade name Aquablack. The WMA and HMA were produced and placed on a new section of US-12. The estimated two-way AADT for this section of roadway was approximately 6,900 vehicles per day with 17 percent trucks. The production of the WMA and HMA control took place on April 19 and 20, 2010 and the contractor was Granite Northwest Inc., Pasco, WA.

The asphalt mixture used for this trial consisted of a coarse-graded 12.5-mm NMAS Superpave mix design with a compactive effort of 100 gyrations. The mix design used for the HMA was also used for the WMA without any changes. The aggregate used for the design was a basalt and natural sand blend including 20 percent RAP. The materials percentages used for mix design submittal and production are shown in Table 70.

The Washington State Department of Transportation (WSDOT) allows the substitution of up to 20 percent RAP without changing the virgin binder grade. The asphalt mixture used a PG 64-28 asphalt binder. A liquid anti-stripping agent was added to the asphalt binder at a rate of 0.25 percent by weight of liquid binder. The design aggregate gradation, optimum asphalt content, design volumetrics, specifications, and allowable tolerances are shown in Table 71. It should be noted that the design was done without RAP, as is common in the state of Washington.

Table 70 Aggregate Percentages for Walla Walla, WA

Aggregate Type	Mix Design (%)	Production (%)
Coarse Chips	21	12
Fine Chips	76	62
Natural Sand	3	6
RAP	0	20

Table 71 Design Gradation, Asphalt Content, and Volumetrics for Mix Design for Walla Walla, WA

Property / Sieve Size	JMF	Specifications	Tolerances
19.0 mm (3/4”)	100	100	99-100
12.5 mm (1/2")	94	90-100	90-100
9.5 mm (3/8")	81	90 Max	75-87
4.75 mm (#4)	52	--	47-57
2.36 mm (#8)	34	28-58	30-38
1.18 mm (#16)	23	--	--
0.60 mm (#30)	16	--	--
0.30 mm (#50)	12	--	--
0.15 mm (#100)	8	--	--
0.075 mm (#200)	5.6	2.0-7.0	3.6-7.0
AC (%)	5.2	0-10	4.7-5.7
Air Voids (%)	3.7	2.5-5.5	2.5-5.5
VMA (%)	14.7	14 min.	12.5 min.
VFA (%)	75	65-75	65-75
D/A Ratio	1.2	0.6-1.6	0.6-1.6

Production

The WMA was produced using the Aquablack WMA system developed by Maxam Equipment, Inc. This system, shown in Figure 40, uses a foaming gun (enlarged for detail on the right side of the figure) to create the foam. For this field trial, water was added at a rate of 2.5 percent by weight of the virgin asphalt binder.

For the WMA, 2,286 tons were produced, while 1,974 tons of HMA were produced the following day. Production temperature for the WMA was approximately 275°F (135°C), and for the HMA control, approximately 325°F (163°C). The asphalt plant used to produce the asphalt mixtures was a portable, parallel-flow Cedar Rapids drum mix plant that incorporated a Hauck SJO-580 Starjet burner. Figure 41 shows the asphalt plant used for this field trial.

Figure 40 Aquablack WMA System used in Walla Walla, WA

Figure 41 Granite Northwest Portable Asphalt Plant used in Walla Walla, WA

Volumetric Mix Properties

Samples of each mixture were obtained during production to compare moisture contents, percent coating, and volumetric properties between the HMA and WMA. Samples were taken from trucks leaving the plant.

AASHTO T 329 was used to evaluate the moisture content of loose plant-produced mix. The average moisture contents were 0.07 and 0.23 percent for the HMA and WMA, respectively. These results are well below the allowable maximum moisture content in WSDOT specifications. A higher moisture content of about 0.1 percent for the WMA was expected due to the addition of water for foaming (2.5 percent by weight of virgin asphalt binder, which is about 0.1 percent, by weight of total mix). It is possible the higher moisture content of the WMA might also have been partially due to the lower mix production temperature for WMA, which could have left some residual moisture in the aggregate or RAP. However, is more likely that the difference in moisture content was influenced by sampling variability.

AASHTO T 195 was used to evaluate asphalt coating of the loose plant-produced mix (one sample per mix per day). Mix obtained from truck samples was sieved over a 3/8 in. (9.5 mm) sieve. Visual inspections of the particles retained on the 3/8 in. (9.5 mm) sieve were conducted, which consisted of classifying a particle as partially or completely coated. The percent of completely coated particles was then calculated. The percent of coated particles was 99.3 percent for the HMA and 100.0 percent for the WMA. Thus, the WMA and HMA exhibited similar coating characteristics.

Specimens were compacted using 100 gyrations of the Superpave gyratory compactor (SGC) at compaction temperatures of 300°F for the HMA samples and 250°F for the WMA samples. Water absorptions of the specimens were below 1 percent; therefore bulk specific gravities were determined in accordance with AASHTO T 166. Average test results are summarized in Table 72.

The gradation results for both the HMA and WMA were within the JMF tolerances. The asphalt content of the WMA (5.11 percent) was close to the JMF (5.2 percent). Although the asphalt content of the HMA (5.66 percent) was higher than the WMA, it was still within the acceptable range of 5.2 ± 0.5 percent. The percentage of absorbed asphalt was also higher for the HMA than the WMA. Higher binder absorptions might be expected with higher production temperatures. However, the air voids of both mixes were equivalent and met the specifications.

Table 72 Gradation, Asphalt Content, and Volumetrics for Plant-Produced Mix from Walla Walla, WA

Property	HMA	WMA	JMF
Sieve Size	% Passing		
19.0 mm (3/4”)	100.0	100.0	100
12.5 mm (1/2”)	94.0	95.4	94
9.5 mm (3/8”)	80.1	81.0	81
4.75 mm (#4)	51.9	49.5	52
2.36 mm (#8)	33.4	31.3	34
1.18 mm (#16)	23.2	21.9	23
0.60 mm (#30)	17.6	16.8	16
0.30 mm (#50)	14.3	13.8	12
0.15 mm (#100)	9.5	9.7	8
0.075 mm (#200)	6.0	6.6	5.6
Asphalt Content (%)	5.66	5.11	5.2
G_{mm}	2.606	2.597	--
G_{mb}	2.517	2.509	--
Air Voids (%)	3.4	3.4	3.7
P_{ba} (%)	1.15	0.63	--

Construction

A new section of US-12 was built approximately parallel to the existing roadway. The produced WMA and HMA were placed as the surface course directly on top of the new intermediate asphalt pavement layer. The WMA was placed in the passing lane and the HMA in the traveling lane. Figure 42 illustrates the location of the test sections. The WMA section monitored for this project began before the HMA section. The green flag on the map indicates the location of asphalt plant. The target thickness was 1.5 inches.

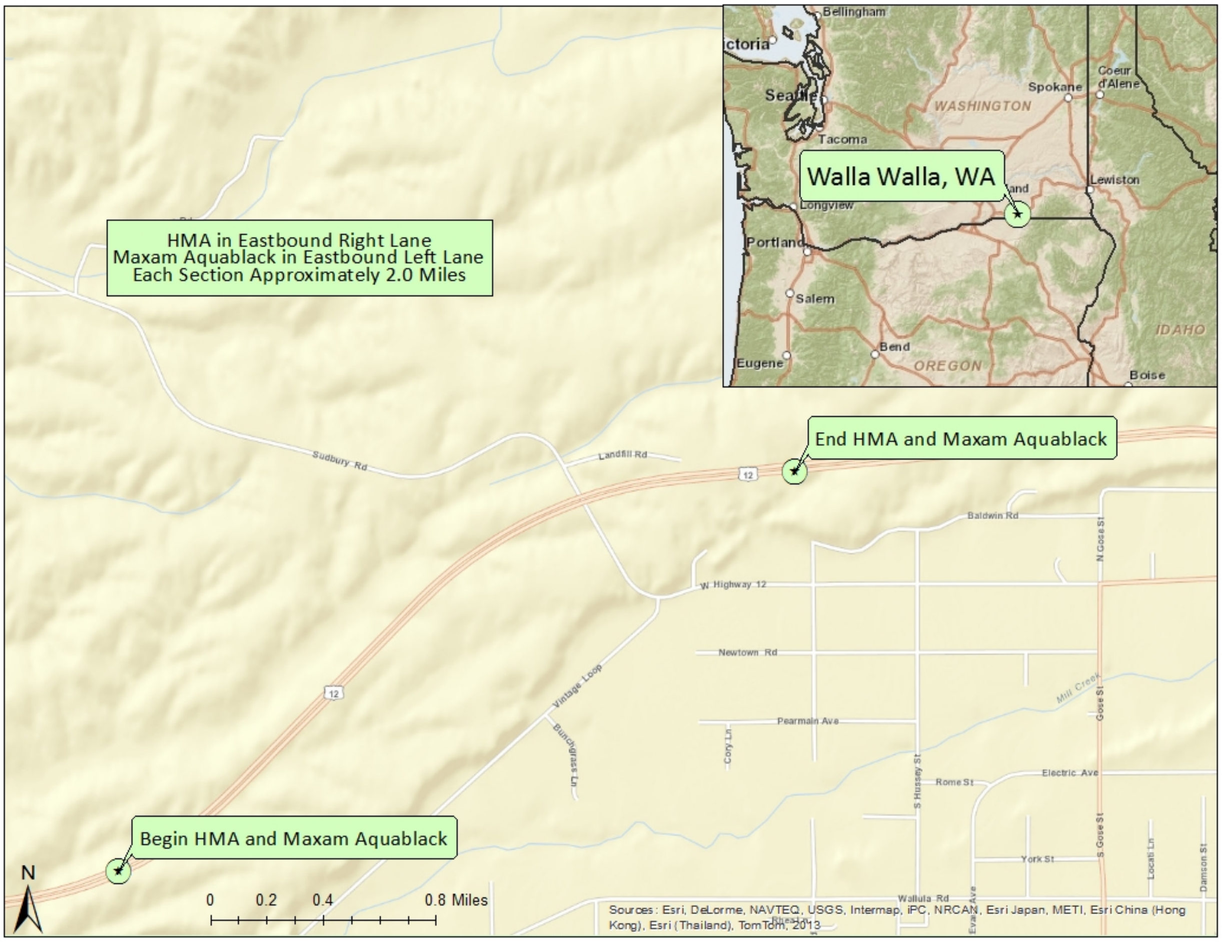


Figure 42 Location of Test Sections in Walla Walla, WA

The haul distance from the plant to the roadway was less than five miles, so there was little production stoppage due to lack of trucks during the day. The delivery temperature of the WMA ranged between 244 and 259°F while that of the HMA ranged between 272 and 295°F. A RoadTec SB-2500D material transfer vehicle (MTV) was used to collect the windrowed mix (see Figure 43 and Figure 44).

Figure 43 Material Transfer Vehicle used in Walla Walla, WA

Figure 44 Material Transfer Device and Windrow in Walla Walla, WA

The MTV discharged the mix into a Blaw-Knox PF 6110 paver as shown in Figure 45. The screed heater was on during WMA and HMA construction, set to 250°F and 270°F during WMA and HMA construction, respectively. The temperature of the WMA behind the screed ranged from 246 to 255°F. The HMA mat temperature behind the screed was between 251 and 287°F.

Figure 45 Blaw-Knox Paver used in Walla Walla, WA

The temperature behind the paver was monitored using temperature probes, which collected temperature data every 30 seconds. Data from the probes were processed to determine the rate at which the mat cooled. Regression was used to fit an equation to the mat temperature and time data collected**.** Figure 46 shows the regression equations for WMA and HMA. From this analysis, the WMA and HMA mixtures had similar cooling rates.

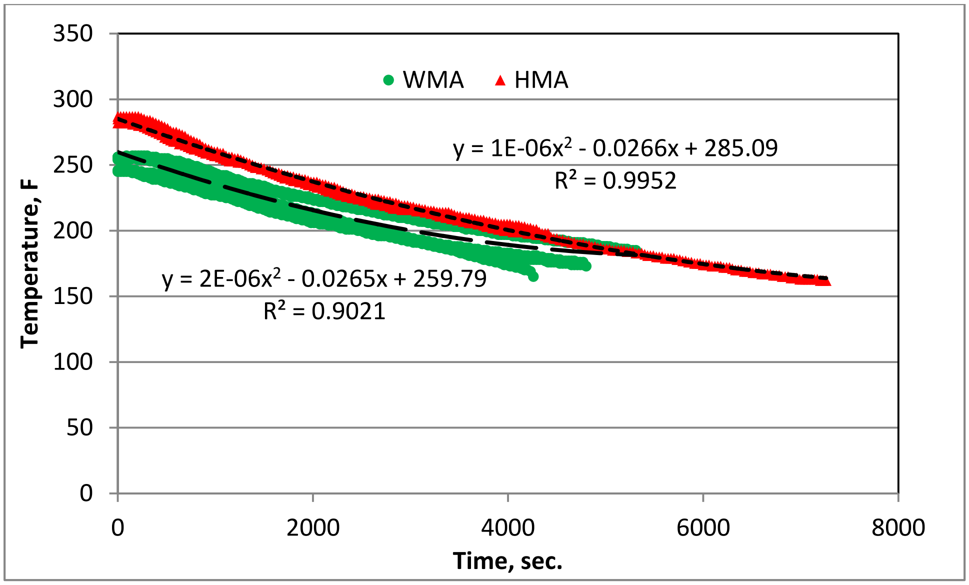


Figure 46 Mix Cooling Trends in Walla Walla, WA

Hourly weather data was collected at the paving location using a handheld weather station. The ambient temperature during the WMA paving ranged between 54.2°F and 87°F (12.3°C and 30.5°C), while the ambient temperature during the HMA paving ranged between 75.6°F and 80.2°F (24.2°C and 26.8°C). The wind during the WMA paving was between 0 and 2.1 mph, and for the HMA paving, between 0 and 9.6 mph. The humidity during the WMA paving was between 33.7 to 68.9 percent. The humidity during the HMA paving was between 26.5 and 38.2 percent.

The mix was compacted using three rollers, and the rolling pattern was the same for both mixes. The WMA breakdown roller was an Ingersoll Rand DD 130HF steel wheel roller, while the HMA breakdown roller was an Ingersoll Rand DD 138 steel wheel roller. A different breakdown roller was used for the HMA since the roller used on the WMA section was mistakenly transported to another site. The difference in rollers was not due to expected changes in compaction. The intermediate roller was a Caterpillar PS 360C rubber tire roller with a tire pressures between 90 and 100 psi. The finish roller was an Ingersoll Rand DD 110HP, which was operated in the static mode.

Construction Core Testing

Field cores were obtained from each section (WMA and HMA) following compaction. Core densities were determined in accordance with AASHTO T 166. Five cores were tested for tensile strength, and additional cores were combined for solvent extraction (AASHTO T 164) and gradation analysis. Average test results are shown in Table 73.

Gradation results for both mixes were very similar. As was the case with the results from the plant mix during production, the asphalt content of the HMA cores (5.69 percent) was higher than that of the WMA cores (4.87 percent). The asphalt content of the HMA cores was very close to the plant mix asphalt content (5.66 percent), while the asphalt content of the WMA cores was slightly less than that of the WMA plant mix (5.11 percent). The difference between the core and field mix asphalt contents for the WMA can probably be attributed to sampling variability. The G_{mm} and other test results for the cores from the WMA and HMA sections are very similar which therefore suggest the asphalt content results for the WMA cores was not correct. Average core densities were similar for both mixes, at 94.6 percent of theoretical maximum specific gravity for the HMA, and 94.4 percent for the WMA. Tensile strengths were also similar for the HMA and WMA.

Table 73 Construction Core Test Results from Walla Walla, WA

Property	HMA	WMA
Sieve Size	% Passing	
25.0 mm (1")	100.0	100.0
19.0 mm (3/4")	100.0	100.0
12.5 mm (1/2")	96.6	94.1
9.5 mm (3/8")	84.5	82.5
4.75 mm (#4)	56.3	54.5
2.36 mm (#8)	37.4	37.2
1.18 mm (#16)	27.2	27.5
0.60 mm (#30)	21.2	21.8
0.30 mm (#50)	17.5	18.1
0.15 mm (#100)	11.5	11.8
0.075 mm (#200)	7.3	7.3
Asphalt Content (%)	5.69	4.87
G_{mm}	2.598	2.606
G_{mb}	2.459	2.459
In-place Density (%)	94.6	94.4
P_{ba} (%)	1.04	0.62
Tensile Strength (psi)	160.9	165.4

Note: Gradation and asphalt content results are based on one sample per mix

Field Performance at 13-Month and 27-Month Project Inspections

A field-performance evaluation was conducted on May 17, 2011, after about 13 months of traffic were applied to the test sections. A second performance evaluation was performed on August 28, 2012, after about 27 months of traffic. Data were collected on each section to document performance regarding rutting, cracking, and raveling following the same procedure described for previous projects. Cores were used to determine the in-place density, indirect tensile strengths, theoretical maximum specific gravity, gradation and asphalt content.

Neither the HMA nor WMA showed significant rutting after 13 months, with the HMA having an average rut depth of 1.0 mm, and the WMA having no measurable rut depth. At the 27-month inspection, the HMA sections exhibited an average rut depth of 4.6 mm, while the WMA sections still had no measurable rutting. The difference in rutting measurements between the HMA and WMA can likely be attributed to the HMA being placed in the travel lane, while the WMA was placed in the passing lane. These results are summarized in Table 74.

Table 74 Rut Depths for Walla Walla, WA

Mix	13-Month Inspection		27-Month Inspection	
	Avg. (mm)	Std. Dev. (mm)	Avg. (mm)	Std. Dev. (mm)
HMA	1.0	0.4	4.6	0.3
WMA	0	0	0	0

Each 200 ft. (61 m) evaluation section was carefully inspected for visual signs of cracking. At the time of both inspections, no cracking was evident in either the HMA or WMA sections.

The surface textures of both the HMA and WMA test sections were measured using the sand patch test according to ASTM E965. The calculated mean texture depths for each mix are shown in Table 75. These values represent the average and standard deviation of the three tests conducted on each section. A smaller mean texture depth indicates a smoother pavement, or one with less surface texture.

Table 75 Mean Texture Depths for Walla Walla, WA

Mix	13-Month Inspection		27-Month Inspection	
	Mean Texture Depth (mm)	Standard Dev. (mm)	Mean Texture Depth (mm)	Standard Dev.(mm)
HMA	1.00	0.13	0.96	0.10
WMA	0.74	0.05	0.86	0.02

These results show that the HMA had a higher mean texture depth at the time of both inspections, which indicates that the HMA has raveled slightly more than the WMA. The difference in textures is likely due to the HMA being placed in the travel lane while the WMA was placed in the passing lane. As shown in Figure 47, Figure 48, and Figure 49, the raveling is visually apparent. It is not clear if this amount of raveling is typical of pavements in this region of the country, but it is greater than what is typical of coarse-graded pavements after one year of traffic in the milder climates of the southeastern United States. However, it can be seen that there is little difference in texture measurements between the 13-month and 27-month inspection for either mixture. Figure 50 shows an example of the surface texture observed at the time of the 27-month inspection.

Figure 47 WMA (Foreground) and HMA (Background) Sections at 13-Month Inspection to Walla Walla, WA

Figure 48 HMA Surface Texture at 13-Month Inspection in Walla Walla, WA

Figure 49 WMA Surface Texture at 13-Month Inspection in Walla Walla, WA

Figure 50 Example of Surface Texture at 27-Month Inspection in Walla Walla, WA

Core Testing

During both project performance inspections, seven 6-inch (150-mm) cores were taken from each mix section. All cores were taken from a location near the construction cores. The densities of these cores were measured using AASHTO T 166. Six of the cores were then tested for tensile strength using ASTM D6931. These six samples were then combined and the cut-faces were removed. This mix was split into two samples that were used to determine the maximum specific gravity according to AASHTO T 209. A summary of the 13-month and 27-month core testing compared to the construction core testing is shown in Table 76 .

Table 76 Test Results on Construction, 13-, and 27-Month Cores in Walla Walla, WA

Property	HMA	WMA	HMA	WMA	HMA	WMA
	Construction Cores (April 2010)		13-Month Cores (May 2011)		27-Month Cores (August 2012)	
Sieve Size	% Passing		% Passing		% Passing	
19.0 mm (3/4”)	100.0	100.0	100.0	100.0	100.0	100.0
12.5 mm (1/2”)	96.6	94.1	95.4	94.1	94.0	94.6
9.5 mm (3/8”)	84.5	82.5	81.9	80.6	82.2	81.9
4.75 mm (#4)	56.3	54.5	51.9	52.8	52.6	53.2
2.36 mm (#8)	37.4	37.2	34.5	36.5	35.8	36.5
1.18 mm (#16)	27.2	27.5	25.2	27.4	25.4	26.0
0.60 mm (#30)	21.2	21.8	19.8	21.9	20.2	20.8
0.30 mm (#50)	17.5	18.1	16.5	18.4	16.7	17.1
0.15 mm (#100)	11.5	11.8	11.4	12.5	11.2	11.4
0.075 mm (#200)	7.3	7.3	7.7	8.2	7.6	7.7
Asphalt Content (%)	5.69	4.87	5.88	5.78	5.19	5.72
G_{mm}	2.598	2.606	2.613	2.617	2.619	2.612
G_{mb}	2.459	2.459	2.506	2.490	2.521	2.500
In-Place Density (%)	94.7	94.4	95.9	95.2	96.3	95.7
P_{ba} (%)	1.04	0.62	1.40	1.40	1.03	1.28
Tensile Strength (psi)	160.9	165.4	104.9	120.4	176.6	165.3

The gradations were very similar for the HMA and WMA and had not changed significantly from the cores taken at construction. There were some variations in asphalt contents for the HMA and WMA at each point it time. The asphalt content from the 13-month HMA cores (5.88 percent) was slightly higher than the asphalt content of the construction cores (5.69 percent), but the 27-month HMA cores had a slightly lower asphalt content (5.19 percent). An extra sample was tested and verified the result for the 27-month HMA cores. The 13-month WMA asphalt content (5.78 percent) was significantly higher than that of the construction cores (4.87 percent) and plant mix sampled during construction (5.11 percent). The variations in asphalt content are likely attributed to sampling and testing variability.

The in-place densities increased at 13 months and 27 months due to densification under traffic load. The densification of the HMA cores in the first 13 months was slightly higher than for the WMA probably because the HMA is in the travel lane and the WMA is in the passing lane.

The tensile strengths of the 13-month cores were lower than the strengths of the construction cores and the 27-month cores. This can probably be attributed to the fact that four-inch cores were taken at construction, while six-inch cores were taken at the 13-month inspection. Theoretically, this should not affect the results from the tensile strength test since the diameter of the specimen is an input in the equation to determine the tensile strength. However, a similar decrease has been observed on other projects. To further investigate this issue, 4- and 6-inch cores were obtained from the NCAT Test Track and tested. Two pavement sections were chosen, and six cores were taken from each section. Three of these cores were 4-inch diameter and three were 6-inch diameter. The cores were all then tested according to ASTM D6931. It was observed that the peak failure load for both the 4-inch and 6-inch cores were very similar between samples in the same mix. This yielded higher tensile strengths for the 4-inch cores compared to the 6-inch cores. These results are shown in Table 77. This indicates that 4-inch cores will typically yield higher tensile strengths compared to 6-inch cores for a given mix.

Table 77 Comparison of Tensile Strength on 4-inch versus 6-inch Cores at the NCAT Test Track

Section ID	Average In-Place Density (%)	Core Diameter (in.)	Average Failure Load (lbs.)	Average Tensile Strength (psi)	Percent Difference
E9	96.0	6	2567	137.0	28.7%
	96.0	4	2567	192.2	
S13	95.4	6	3733	237.7	10.2%
	95.6	4	2667	264.8	

Table 78 shows the average in-place densities and tensile strength results by location for the 13-month and 27-month inspection cores. As expected, the in-place densities were higher in the wheelpaths as compared to those between the wheelpaths for both the HMA and WMA at the time of both inspections. In addition, the tensile strengths for both mixes were slightly lower in the wheelpaths than between the wheelpaths at both inspections.

Table 78 In-Place Density and Tensile Strengths by Location in Walla Walla, WA

Location and Property	HMA	WMA	HMA	WMA
	13-Month Inspection		27-Month Inspection	
Between Wheelpaths In-Place Density (% of G_{mm})	95.7	95.0	96.0	95.6
In Right Wheelpath In-Place Density (% of G_{mm})	96.2	95.4	96.6	95.9
Between Wheelpaths Tensile Strength (psi)	114.6	126.4	177.4	166.3
In Right wheelpath Tensile Strength (psi)	95.3	114.3	175.7	164.3

Performance Predictions

The initial average annual daily truck traffic (AADTT) for Walla Walla, WA was 1,173 trucks per day with two lanes in each direction. A traffic growth factor of 5 percent was provided by Washington DOT. US-12 was classified as a minor arterial. The same traffic was used for the performance predictions for both sections. However, the WMA was placed in the passing lane so it is expected to receive less truck traffic.

Table 79 summarizes the pavement structure. Washington DOT used a subgrade M_r = 11,000 psi in their 40-year pavement design *(26).* Integrated Climatic Model (ICM) calculated moduli were used for the MEPDG analysis.

Table 79 Walla Walla, WA Pavement Structure

Layer	Thickness, in. [cm]
WMA/HMA surface course	1.8 [4.6]
Superpave ½-inch HMA - 12.5 mm NMAS with PG 64-28	6.0 [15.2]
Crushed stone aggregate base	10.0 [25.4]
AASHTO A-4 Subgrade	Semi-infinite

Figure 51 shows a comparison of the predicted rutting for the WMA and HMA sections. The MEPDG predicts that the WMA section (subtotal of rutting in all asphalt layers) will exceed 0.25 in. (6.4 mm) of rutting after 50 months of service, and the HMA section after 52 months of service. After 20 years, the difference in predicted asphalt rutting is negligible, 0.53 in. (13.5 mm) for the HMA and 0.56 in. (14.2 mm) for the WMA. Essentially the same differential (0.04 inch) in predicted rutting is expected for the WMA and HMA surface layers with 0.21 in. (5.3 mm) and 0.17 in. (4.3 mm) at 20 years, respectively.

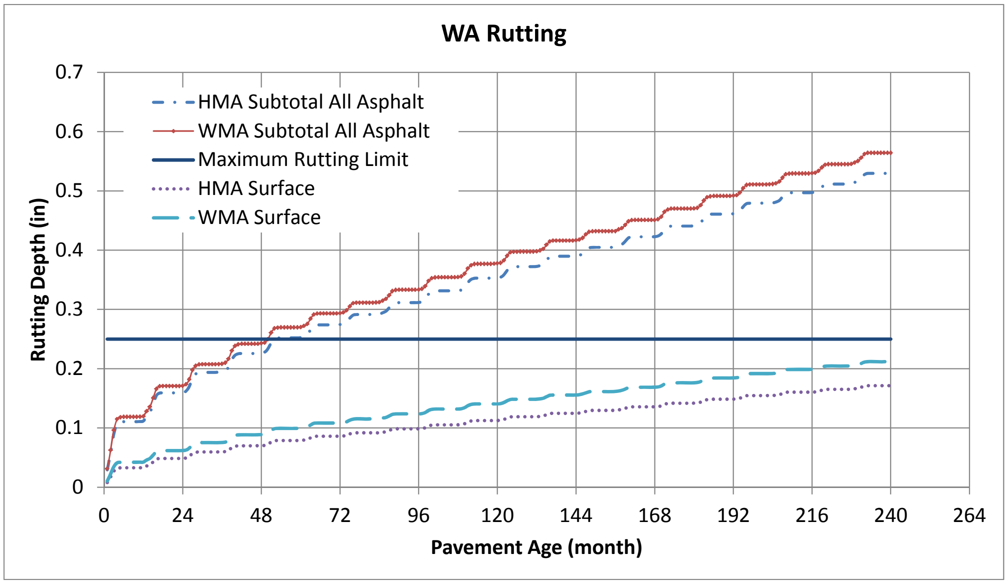


Figure 51 MEPDG Predicted Asphalt Rutting for Walla Walla, WA

Figure 52 compares the predicted longitudinal cracking for US-12 over the design life. Although the MEPDG predicts slightly more cracking for the WMA compared to the HMA, 61.7 versus 34.8 feet per mile (11.7 versus 6.6 m/km) at 20 year, the difference is negligible and the predicted performance of both sections is very good.

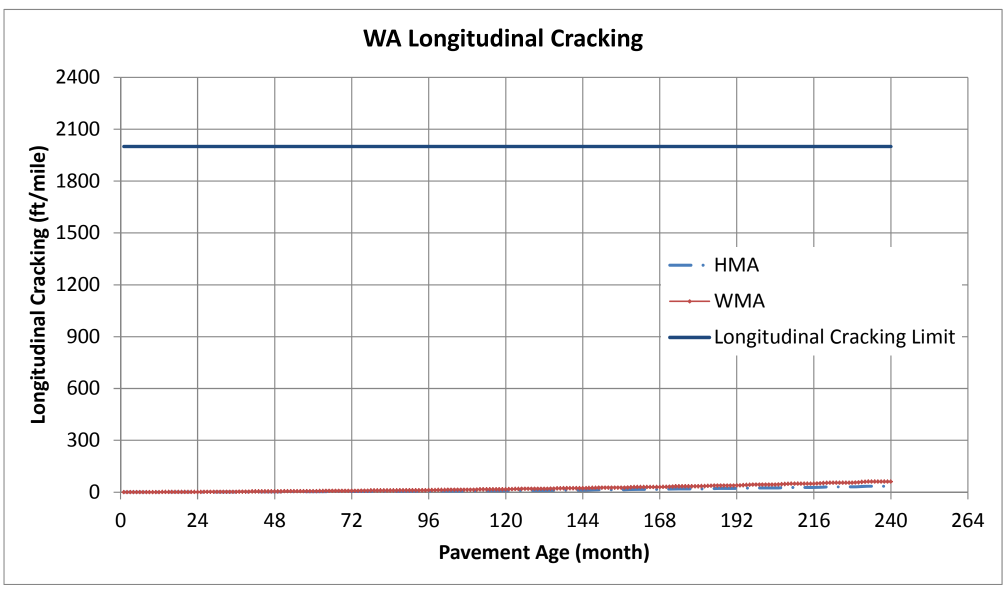


Figure 52 MEPDG Predicted Longitudinal Cracking for Walla Walla, WA

Level 1 IDT thermal cracking inputs were available for the Walla Walla, WA project. The MEPDG predicted zero feet per mile of cracking for both the WMA and HMA sections after 20-years of service. Therefore, the data is not presented graphically.

Centreville, Virginia

A WMA field evaluation was placed on I-66 eastbound near Centreville, Virginia in June 2010. The WMA technology used on this project was the Astec Double-Barrel Green asphalt foaming system using water injection. The WMA and HMA were produced and placed on a highly trafficked section of I-66 eastbound near Centreville, VA. This section of I-66 is about 30 miles west of Washington D.C. The estimated one-way AADT for this section of roadway was approximately 59,000 vehicles per day with 9 percent trucks. The production of the WMA and companion HMA control took place on June 21 and 22, 2010 respectively, with Superior Paving Corp., Bristow, VA as the contractor.

The asphalt mixture used for this trial consisted of a fine-graded 12.5 mm NMAS Superpave mix design, with a compactive effort of 65 gyrations. The mix design used for the HMA was also used for the WMA without any changes. The aggregate used for the design was a diabase and limestone blend including 15 percent RAP. The materials percentages used for mix design submittal and production are shown in Table 80. The asphalt mixture used a polymer modified PG 76-22 asphalt binder supplied by Nustar in Baltimore, Maryland. A liquid anti-stripping agent, Pave Bond™ Lite, manufactured by the Dow Chemical Company, was added to the asphalt binder at a rate of 0.50 percent by weight of liquid binder. The laboratory and production JMFs, optimum asphalt contents, specifications, and allowable tolerances are shown in Table 81.

Table 80 Aggregate Percentages for Centreville, VA

Aggregate Type	%, Mix Design	%, Production
#78 Stone	30	30
#60 Stone	10	10
Stone Sand	15	15
Grade A Sand	15	15
#10 Stone	15	15
Crushed RAP	15	15

Table 81 Design Gradation, Asphalt Content, and Volumetrics for Mix Design for Centreville, VA

Property	Lab JMF	Production JMF	Specifications	Tolerances
Sieve Size	% Passing			
19.0 mm (3/4")	100	100	100	--
12.5 mm (1/2")	96	96	95-100	± 4
9.5 mm (3/8")	87	87	Max 90	± 4
2.36 mm (#8)	41	40	34-50	± 4
0.075 mm (#200)	5.2	5.3	2-10	± 1
AC (%)	5.2	5.3	--	± 0.3
Air Voids (%)	3.9	3.4	--	--
VMA (%)	15.4	14.6	--	--
VFA (%)	74.7	76.7	--	--
D/A Ratio	1.10	1.16	--	--

Production

The WMA was produced using the Astec DBG asphalt foaming system, with water added at a rate of 2.0 percent by weight of the virgin asphalt binder.

For the WMA, 1,027 tons were produced, while 460 tons of HMA were produced the following day. Production temperature for the WMA was approximately 288°F (142°C), and for the HMA control, approximately 318°F (159°C).

Table 82 shows the maximum, minimum, average, and standard deviation production temperatures for both the WMA and HMA. The asphalt plant used to produce the asphalt mixtures was a counter-flow Astec Double-Barrel drum mix plant that incorporated three 200-ton storage silos. Figure 53 shows the asphalt plant used for this field trial.

Table 82 Production Temperatures in Centreville, VA

Statistic	HMA	Astec DBG
Average (°F)	317.5	287.9
Std. Dev. (°F)	11.9	10.1
Maximum (°F)	327	320
Minimum (°F)	294	280

Figure 53 Superior Paving Astec DBG Asphalt Plant used in Centreville, VA

Volumetric Mix Properties

Samples of each mixture were obtained during production to compare moisture contents, percent coating, and volumetric properties between the HMA and WMA. Samples were taken from trucks leaving the plant.

AASHTO T 329 was used to evaluate mix using loose plant-produced mix. The average moisture contents were 0.04 percent and 0.14 percent for the HMA and WMA, respectively. These results are both fairly low and reasonable. It was expected that the WMA would have slightly higher mix moisture content for two reasons. First, the addition of 2 percent water by weight of virgin binder for the foaming process is approximately equal to about 0.1 percent of the total mix, and the WMA had about a 0.1 percent higher mix moisture content. In addition, it is possible the higher moisture content for the WMA was partially due to the lower mix production temperature for WMA, which could have left more residual moisture in the aggregate or RAP going through the plant as compared to the HMA mixture. It is also possible that the difference in moisture content is influenced by sampling variability.

The percent of completely coated particles according to AASHTO T 195 was calculated. The percent of coated particles was 100 percent for both the HMA and WMA mixtures. Thus, the WMA and HMA exhibited similar coating characteristics.

Specimens were compacted using 65 gyrations in the Superpave gyratory compactor (SGC) at compaction temperatures of 310°F for the HMA samples and 260°F for the WMA samples. These laboratory compaction temperatures were determined using the average compaction temperature observed on the test section through the first couple of hours of construction for each mixture. These volumetric samples were plant mixed then compacted on-site in the NCAT mobile laboratory to avoid reheating which could affect asphalt absorption and other volumetric properties. Water absorption of the compacted specimens were below 1 percent, therefore bulk specific gravities (G_{mb}) were determined in accordance with AASHTO T 166. Asphalt contents were determined in accordance with AASHTO T 164. Gradations of the extracted aggregates were determined according to AASHTO T 30. Average test results are summarized in Table 83. The gradation and asphalt content results for both the HMA and WMA were within the JMF tolerances. The asphalt content of the WMA (5.4 percent) was close to the production JMF (5.2 percent). On the other hand, the asphalt content of the HMA (5.0 percent) was a good bit lower than the WMA but was still within the acceptable range of 5.3 ± 0.3 percent. The percentages of absorbed asphalt were essentially equivalent for the two mixtures. However, the air voids for the WMA were significantly lower compared to the HMA. This most likely resulted from the higher asphalt content for the WMA. Improved compactability of the WMA may also have contributed to the lower voids.

Table 83 Gradation, Asphalt Content, and Volumetrics for Plant-Produced Mix from Centreville, VA

Property	Production	HMA	Astec DBG	Tolerances
Sieve Size	% Passing			
19.0 mm (3/4”)	100.0	100.0	100.0	--
12.5 mm (1/2”)	96.0	95.3	97.8	± 4
9.5 mm (3/8”)	85.0	81.0	83.6	± 4
4.75 mm (#4)	--	51.0	54.9	--
2.36 mm (#8)	40.0	36.3	39.3	± 4
1.18 mm (#16)	--	26.9	29.4	--
0.60 mm (#30)	--	19.2	21.1	--
0.30 mm (#50)	--	12.3	13.5	--
0.15 mm (#100)	--	7.6	8.3	--
0.075 mm (#200)	5.3	4.8	5.0	± 1
AC (%)	5.3	5.0	5.4	± 0.3
G_{mm}	2.599	2.620	2.605	--
G_{mb}	2.511	2.510	2.534	--
Air Voids (%)	3.4	4.2	2.8	--
P_{ba} (%)	0.75	0.88	0.92	--

Construction

The eastbound portion of I-66 near Centreville, VA was widened from two lanes to four lanes. The test section for this study runs from approximately milepost 42.2 to the bridge for US-29 which crosses over I-66 (~MP 43.05). The two new lanes were placed to the left of the two original lanes, and were paved with WMA. The center-left travel lane was the lane being paved while NCAT was on-site and was designated as the WMA test section. The HMA was overlaid on the two right (existing) lanes. The center-right travel lane was designated as the HMA test section for this project. The HMA was placed over a milled section of asphalt roadway and the WMA was paved over new asphalt construction. Figure 54 illustrates the location of the test sections. Both the HMA and WMA test sections were paved as the surface (wearing) course and had a target thickness of 1.5 inches. A trackless tack coat was applied before paving both sections.

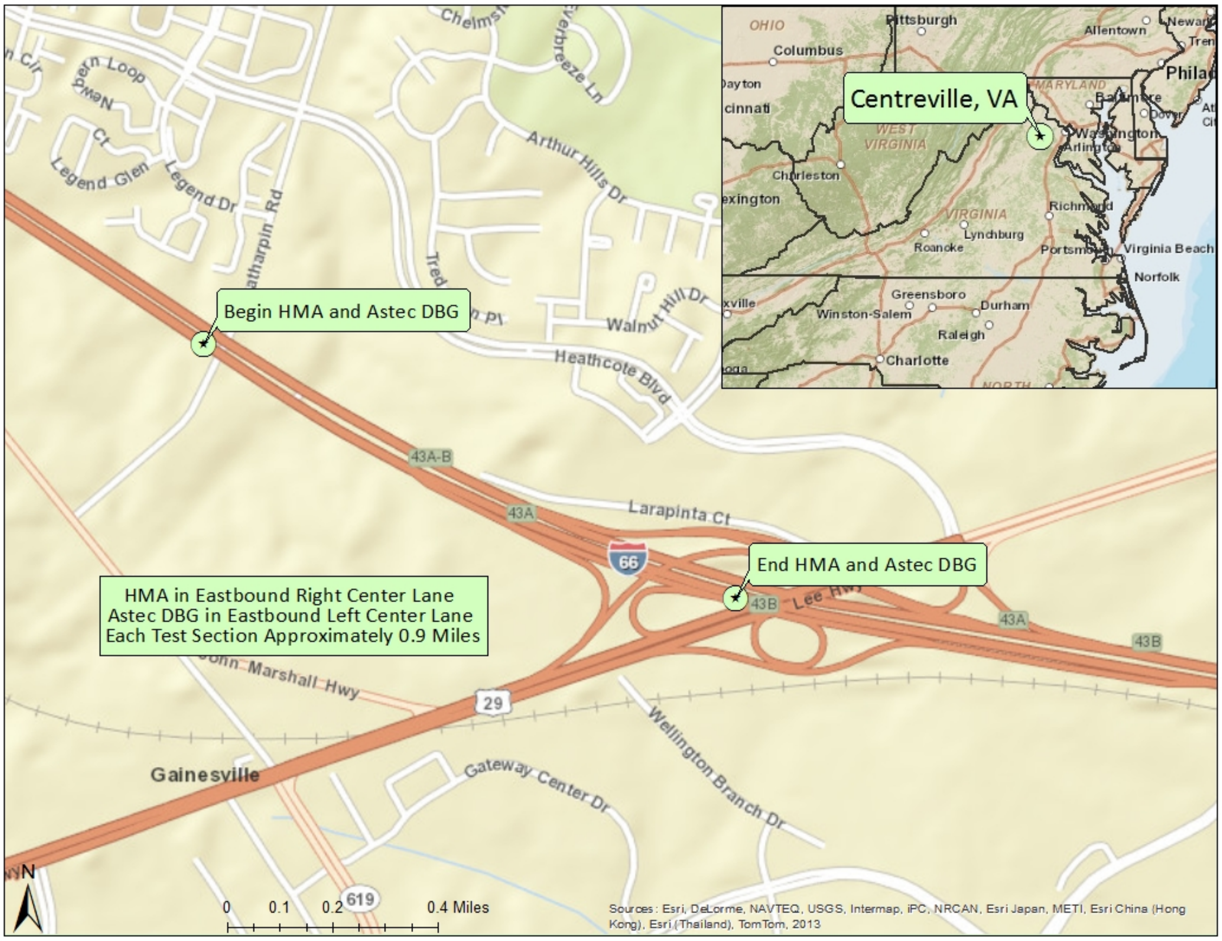


Figure 54 Location of Test Sections in Centreville, Virginia

The asphalt mixtures were delivered using tarped dump trucks. The haul distance from the plant to the roadway was approximately 12 miles. The travel time between the plant and site varied from 20 to 40 minutes depending on traffic. Figure 55 shows a truck dumping into the MTV.

Figure 55 Truck Dumping into MTV in Centreville, VA

A RoadTec SB-1500D MTV was used to transfer the mixtures from the delivery trucks to the paver. A RoadTec RP-190 was the paver used for this project. Figure 56and Figure 57 show the MTV and paver used respectively.

Figure 56 RoadTec SB-1500D Material Transfer Vehicle used in Centreville, VA

Figure 57 RoadTec RP-190 Paver used in Centreville, VA

The temperature of the mix behind the paver was measured using both a hand-held temperature gun and the PAVE-IR system manufactured by the MOBA Corporation. The PAVE-IR system consists of 12 infrared sensors that measure and record pavement temperatures across the mat and display on a mounted monitor. In addition to recording pavement temperatures for research purposes, the PAVE-IR system allows real-time adjustments to be made to help mitigate thermal segregation if it becomes apparent. The PAVE-IR system is shown in Figure 58.

Figure 58 PAVE-IR System

On the day of WMA production, there were some technical difficulties with the PAVE-IR system and it was not fully functional until about 2:00 pm. Table 84shows the temperatures from behind the screed using both measuring techniques. It should be noted that since the PAVE-IR system takes continuous readings some differences are expected as compared to the temperature gun readings taken periodically.

Table 84 Temperatures behind the Screed in Centreville, VA

Temperature (°F)	Measuring Device	HMA	Astec DBG
Average	Temperature Gun	292.0	258.5
	PAVE-IR	293.5	267.5
Standard Deviation	Temperature Gun	14.9	6.1
	PAVE-IR	12.5	8.9
Maximum	Temperature Gun	308.0	265.0
	PAVE-IR	323.0	307.0
Minimum	Temperature Gun	276.0	248.0
	PAVE-IR	245.0	221.0

Weather data was collected hourly at the paving location using a handheld weather station. The ambient temperature during the WMA paving ranged between 87.7°F and 100°F (30.9°C and 37.8°C), while the ambient temperature on-site during the HMA paving ranged between 95.1°F and 101.8°F (35.1°C and 38.8°C). The wind during the WMA paving was between 0.9 and 2.0 mph, and for the HMA paving, between 1.2 and 2.4 mph. The humidity during the WMA paving was between 29.1 and 43.7 percent. The humidity during the HMA paving was between 37.8 and 43.4 percent. There was no rain during the paving of either mix.

Three rollers were used to compact both mixes. The breakdown roller used was an Ingersoll Rand DD110 steel wheel roller operated in the vibratory mode. Both the intermediate and finishing rollers were Ingersoll Rand DD70 steel wheel rollers operated in the static mode. The rolling pattern used for all three rollers for the majority of placement was four passes on each side and then back up the joint. The rolling pattern was the same for both mixes.

Construction Core Testing

After construction, seven 6-inch (150 mm) cores were obtained from each section (HMA and WMA). Core densities were determined in accordance with AASHTO T 166. If the water absorption was determined to be higher than 1 percent, the samples were then tested according to AASHTO T 331. Six of the cores from each mix were also tested for tensile strength according to ASTM D6931. Average test results are shown in Table 85.

Average core densities were similar for both mixes, at 89.1 percent of maximum theoretical specific gravity (G_{mm}) for the HMA and 89.9 percent of G_{mm} for the WMA. These results are lower than commonly expected for most new asphalt pavement layers. The tensile strengths for both mixes were reasonable and similar.

Table 85 Construction Cores Test Results for Centreville, VA

Property	Statistic	HMA	Astec DBG
In-place Density (% of G_{mm})	Average	89.1	89.9
	Standard Deviation	1.7	1.2
Tensile Strength (psi)	Average	131.9	135.8
	Standard Deviation	10.9	12.9

Field Performance at 15-Month and 24-Month Project Inspections

A field-performance evaluation was conducted on September 26 and 27, 2011, after about 15 months of traffic were applied to the test sections. A second performance evaluation was performed on June 26 and 27, 2012 after about 24 months of traffic Data were collected on each section to document rutting, cracking, and raveling. In addition, three 6-inch (150 mm) diameter cores were taken from the right wheelpath, and four 6-inch (150 mm) diameter cores were taken from between the wheelpaths for both sections. These cores were used to determine the in-place density, indirect tensile strengths, theoretical maximum specific gravity, gradation, asphalt content, and the recovered true binder grade for each mix.

The rut depths were measured at the beginning of each 200 ft. (61 m) evaluation section with a straight edge and a wedge. Neither of the mixes had any measurable rutting (greater than 1/16 of an inch, or 1.5-mm) in any of the three evaluation sections at the time of the 15-month inspection. At the time of the 24-month inspection, a string line was used to measure rutting so that more precision could be achieved. The HMA section had an average rutting depth of 3.2 mm, while the WMA section had an average of 2.7 mm of rutting. Both mixes performed comparably in terms of rutting.

Each evaluation section was carefully inspected for visual signs of cracking. No cracking was visible at the time of either inspection.

Surface textures of the HMA and WMA test sections were measured using the sand patch test at the beginning of each evaluation section in the right wheelpath. The calculated mean and standard deviations of the texture depths for each mix are shown in Table 86.

Table 86 Mean Texture Depths for Centreville, VA

Mix	15-Month Inspection		24-Month Inspection	
	Mean Texture Depth (mm)	Standard Dev. (mm)	Mean Texture Depth (mm)	Standard Dev. (mm)
HMA	0.55	0.04	0.62	0.03
WMA	0.48	0.07	0.61	0.03

These results show similar mean texture depths for the two mixes. Although the 15-month mean texture depth for the WMA section was slightly lower than for the HMA section, the small difference may have been due to the sections being in different lanes. Overall, the results of the sand patch test show that both mixes have performed well in terms of raveling and weathering. As expected, the mean texture depths increased for both sections after 24-months. Figure 59 shows both sections with the HMA on the right and the WMA on the left.

Figure 59 WMA and HMA Sections at 15-Month Inspection in Centreville, VA

Core Testing

At the time of each project inspection, seven 6-inch (150-mm) cores were taken from each mix section. Four of these cores came from between the wheelpaths, and three came from the right wheelpath. These cores were spread throughout the mix sections to avoid having patched core holes in close proximity on this highly trafficked road. The densities of these cores were measured using AASHTO T 166. If the water absorption was determined to be higher than 1 percent, the samples were then tested according to AASHTO T 331. Six of the cores were then tested for tensile strength using ASTM D6931. These six samples were then combined and the cut-faces were removed. This mix was split into two samples that were used to determine the maximum specific gravity according to AASHTO T 209. A summary of the 15- and 24-month core testing compared to the construction data is shown in Table 87.

The results indicate that the surface layers densified under traffic at 15-months but did not change over the next year. The maximum specific gravities for both mixes were almost the same and were consistent with the construction data. At 15-months, the average tensile strength for the HMA was about 20-psi lower than the construction cores, but at 24 months, the HMA tensile strengths were higher and similar to the results for the WMA section.

Table 87 Test Results on Production Mix, 15-, and 24-Month Cores from Centreville, VA

Property	HMA	Astec DBG	HMA	ASTEC DBG	HMA	Astec DBG
	Production Mix (June 2010)		15-Month Cores (September 2011)		24-Month Cores (September 2012)	
Sieve Size	% Passing		% Passing		% Passing	
19.0 mm (3/4”)	100.0	100.0	100.0	100.0	100.0	100.0
12.5 mm (1/2”)	95.3	97.8	97.9	98.0	97.5	97.4
9.5 mm (3/8”)	81.0	83.6	87.9	85.7	85.6	85.5
4.75 mm (#4)	51.0	54.9	56.7	56.0	55.7	54.6
2.36 mm (#8)	36.3	39.3	40.9	40.5	39.8	39.8
1.18 mm (#16)	26.9	29.4	29.2	29.3	28.2	28.7
0.60 mm (#30)	19.2	21.1	21.2	21.5	20.1	20.9
0.30 mm (#50)	12.3	13.5	13.8	13.6	12.5	13.0
0.15 mm (#100)	7.6	8.3	8.8	8.5	7.5	7.7
0.075 mm (#200)	4.8	5.0	5.9	5.4	4.6	4.7
Asphalt Content (%)	5.0	5.4	5.1	5.2	5.0	4.8
G_{mm}	2.620	2.605	2.600	2.612	2.614	2.613
G_{mb}	2.333*	2.341*	2.449	2.439	2.451	2.440
In-place Density (%)	89.1*	89.9*	94.0	93.5	93.8	93.4
P_{ba} (%)	0.88	0.92	0.61	0.91	0.78	0.61
Tensile Strength (psi)	131.9*	135.8*	110.8	141.8	166.3	176.5

*Data comes from construction cores, not mix sampled during production as indicated by the column header.

Table 88 shows the average densities and tensile strength results by location for both project inspections. For the HMA at the first inspection, the average density in the wheelpath was slightly lower than the average density between the wheelpaths, which was not expected. However, this difference is minimal (0.3 percent), and can be attributed to variability in sampling and testing. At the second inspection, the HMA densities were as expected with the wheelpath densities slightly higher (0.4 percent) than between the wheelpaths. For the WMA, the right wheelpath cores had higher densities than the cores between the wheelpath at both inspections as expected. The tensile strengths for the HMA at both inspections were lower in the wheelpath as compared to the cores between the wheelpath. However, the WMA cores had higher tensile strengths at both inspections for the cores in the wheelpath. The difference is most likely attributed to sampling and testing variability since all of the cores were taken at different longitudinal locations.

Table 88 In-place Density and Tensile Strengths by Location in Centreville, VA

Property	HMA	Astec DBG	HMA	Astec DBG
	15-Months		24-Months	
Between Wheelpaths In-Place Density (% of G_{mm})	94.5	93.0	93.6	93.2
In Right Wheelpath In-Place Density (% of G_{mm})	94.2	94.2	94.0	93.9
Between Wheelpaths Tensile Strength (psi)	135.9	130.5	191.4	146.0
In Right wheelpath Tensile Strength (psi)	94.1	153.0	141.1	206.9

Performance Predictions

The initial AADTT for I-66 near Centreville, VA was 10,620 trucks per day with four lanes in each direction. Traffic counts have varied for this route over the past ten years with increases followed by decrease with an overall trend of approximately 3 to 4 percent growth. A traffic growth factor of 3 percent was used for the MEPDG. The WMA and HMA were not placed in the same lanes. At this location, I-66 has three travel lanes and a high occupancy vehicle (HOV) lane. The HMA was placed in the center travel lane; the WMA in the left travel lane. Half of the width of the center travel lane, the left travel lane and HOV lanes were new construction. For the MEPDG performance predictions, both the WMA and HMA were treated as if they were in the design (right) travel lane and were new construction. Table 89 summarizes the pavement structure used to model the I-66 sections.

Table 89 I-66 Centreville, VA Pavement Structure

Layer	Thickness, in. [cm]
WMA/HMA Surface course	1.5 [3.8]
IM 19.0 D – 19.0 mm NMAS with PG 70-22	3.0 [7.6]
BM 25.0A - 25.0 mm NMAS with PG 64-22	13.0 [33.0]
21A Cement treated Aggregate Base, E = 2,000,000 psi	10.0 [25.4]
AASHTO A-4 Subgrade	Semi-infinite

Figure 60 shows a comparison of the predicted rutting for the WMA and HMA sections. The predicted rutting shown is the subtotal for all of the asphalt layers. The predictions are identical for both the WMA and HMA mixes. The total predicted asphalt rutting after 20-years of service is 0.24 in. (6.1 mm) for both mixes.

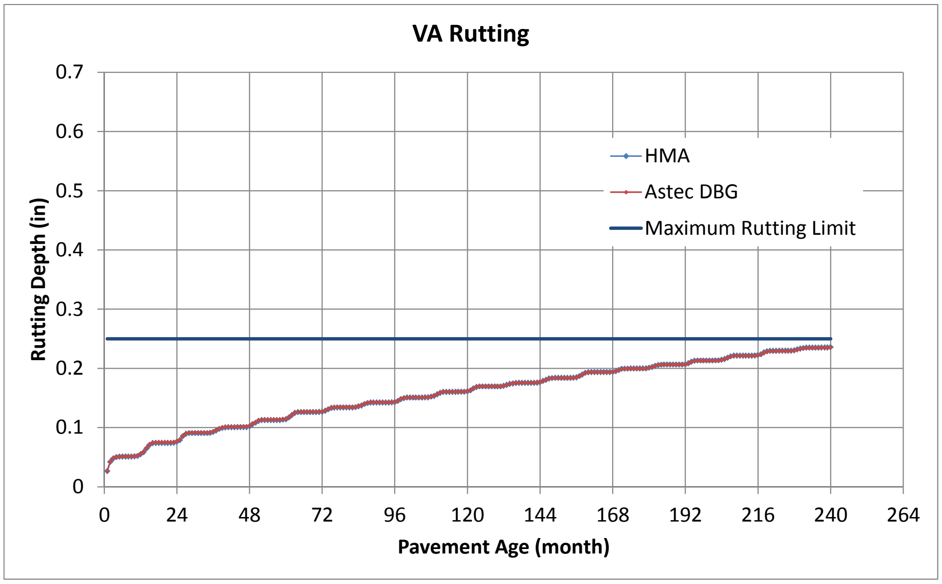


Figure 60 MEPDG Predicted Asphalt Rutting for I-66 Centreville, VA

Figure 61 compares the predicted longitudinal cracking for the WMA and HMA sections. The predicted cracking after 20-years of service was almost identical with 9.9 ft./mile (1.9 m/km) for the WMA and 21.0 ft./mile (4m/km) for the HMA. Level 1 IDT data was available for I-66. The MEPDG predicted 0.01 ft./mile (0.002m/km) of thermal cracking after 222 months for the WMA. No thermal cracking was predicted for the HMA.

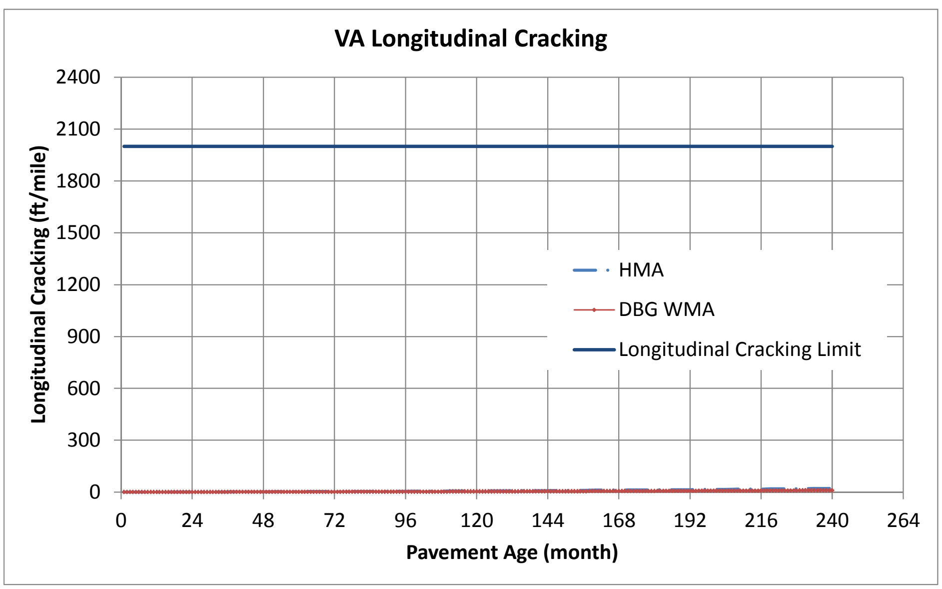


Figure 61 MEPDG Predicted Longitudinal Cracking for I-66, Centreville, VA

Rapid River, Michigan

A WMA field project was constructed on CR 513 near Rapid River, Michigan in July 2010. Payne and Dolan, Inc., Waukesha, WI was the contractor for this project. The first WMA technology used on this project was the foaming additive Advera WMA manufactured by the PQ Corporation. The other WMA technology used was the chemical additive Evotherm 3G developed by MeadWestvaco Asphalt Innovations. The estimated two-way AADT for CR 513 was 1000 vehicles per day with 6 percent trucks. The production and construction of the HMA, Advera, and Evotherm 3G surface mixes took place on July 19, 20, and 22 respectively.

The asphalt mixture used for this trial consisted of a fine-graded 12.5-mm NMAS Marshall mix design compacted to 50 blows on each side. A correlation was then performed by the contractor to determine the equivalent Superpave gyration level. A compactive effort of 30 gyrations was determined to yield 4 percent air voids to match the Marshall mix design. The mix design used for the HMA was also used for both WMA technologies without any changes. All three mixes contained local gravel and 17 percent RAP. The material percentages used for mix design and production are shown in Table 90. A PG 52-34 asphalt binder supplied by Payne and Dolan was used for all three mixes. The design values from the JMF are shown in Table 91.

Table 90 Aggregate Percentages Used in Mix Design and Production for Rapid River, MI

Aggregate Type	Cold Feed (%)
¾” x ½”	11
½” x ¼”	13
Man. Sand	20
Natural Sand	32
Fine Sand	7
RAP	17

Table 91 Design Gradation and Volumetrics for Rapid River, MI

Property	JMF
Sieve Size	% Passing
19.0 mm (3/4")	100.0
12.5 mm (1/2")	93.1
9.5 mm (3/8")	85.2
4.75 mm (#4)	66.1
2.36 mm (#8)	49.3
1.18 mm (#16)	35.8
0.60 mm (#30)	24.9
0.30 mm (#50)	16.9
0.15 mm (#100)	9.2
0.075 mm (#200)	5.8
AC (%)	5.30
Air Voids (%)	4.0
VMA (%)	14.6
VFA (%)	72.6
D/A Ratio	0.79
P_{be} (%)	4.55
P_{ba} (%)	0.79

Production

Both WMA additives were metered into the plant. The Advera WMA was metered into the plant at a rate of 3.75 pounds per ton. The device used to meter the Advera WMA is shown in Figure 62, and the point of entry into the plant is shown in Figure 63. The Evotherm 3G was metered in at the plant at a rate of 0.4 percent by weight of virgin binder.

Figure 62 Advera WMA Hopper in Rapid River, MI

Figure 63 Point of Advera Feed in Rapid River, MI

Table 92 shows the production temperatures for each surface mix placed on this project. The plant was a portable parallel-flow drum plant manufactured by Dillman Equipment, Inc. The plant can be seen in Figure 64.

Table 92 Production Temperatures in Rapid River, MI

Statistic	HMA	Advera	Evotherm
Average (°F)	299.8	268.6	269.4
Standard Deviation (°F)	10.9	15.4	6.3
Maximum (°F)	314	309	279
Minimum (°F)	273	254	258

Figure 64 Parallel Flow Portable Drum Plant in Rapid River, MI

Volumetric Mix Properties

Samples of each mixture were obtained during production to compare moisture contents, percent coating, and volumetric properties between the HMA and WMA. Samples were taken from trucks leaving the plant.

AASHTO T 329 was used to determine the moisture content of loose plant-produced mix (two samples per mix per day). The temperature stipulated in AASHTO T 329 was not used due to limited oven space in the NCAT mobile laboratory, which prevented one oven being used solely for moisture-content testing. The oven temperature was set to the target compaction temperature plus 20°F. This was the temperature needed to get the gyratory samples to reach compaction temperature quickly. Each sample was approximately 1000 g. The samples were heated to a constant mass (less than 0.05 percent change), as defined by AASHTO T 329.

The average moisture contents were 0.07, 0.04, and 0.07 percent for the HMA, Advera, and Evotherm 3G respectively. All three mixes had similar mix moisture content which indicates that incomplete aggregate drying was not an issue for this project.

AASHTO T 195 was used to evaluate asphalt coating of the loose plant-produced mix. The percent of coated particles was 100, 100, and 99.6 percent for the HMA, Advera, and Evotherm 3G respectively. A minimum of 95 percent coating is recommended for WMA *(21)*. Thus, all three mixes exhibited similar coating characteristics.

Specimens were compacted using 30 gyrations in the SGC at compaction temperatures of 300°F for the HMA and 250°F for both WMA mixes. These laboratory compaction temperatures were determined using the average compaction temperature observed on the test sections through the first couple of hours of construction for each mixture. These volumetric samples were plant mixed and compacted on-site in the NCAT mobile laboratory so that the mixes would not have to be reheated. Water absorption levels of the compacted specimens were below 1 percent, therefore bulk specific gravities (G_{mb}) were determined in accordance with AASHTO T 166. Samples of the mixes were transported to the main NCAT laboratory where solvent extractions were conducted in accordance with AASHTO T 164. The gradation of the extracted aggregate was determined according to AASHTO T 30. Average test results are summarized in Table 93.

The average gradations for all three mixes are fairly close to the design targets. The average air void content for the HMA volumetric samples was only 0.1 percent lower than the target 4 percent. The two WMA technologies on the other hand had lower air void contents compared to the target value as commonly seen with WMA even at lower compaction temperatures.

Table 93 Gradation, Asphalt Content, and Volumetrics for Plant-Produced Mix from Rapid River, MI

Property	JMF	HMA	Advera	Evotherm
Sieve Size	% Passing			
19.0 mm (3/4”)	100.0	100.0	100.0	100.0
12.5 mm (1/2”)	93.1	94.2	94.5	95.0
9.5 mm (3/8”)	85.2	86.0	86.7	84.2
4.75 mm (#4)	66.1	67.3	68.0	63.9
2.36 mm (#8)	49.3	50.7	51.3	48.4
1.18 mm (#16)	35.8	37.6	37.9	36.1
0.60 mm (#30)	24.9	26.1	26.3	25.5
0.30 mm (#50)	16.9	17.4	17.8	17.6
0.15 mm (#100)	9.2	9.5	9.9	10.1
0.075 mm (#200)	5.8	5.7	6.0	6.4
AC (%)	5.30	5.26	5.34	5.00
G_{mm}	2.489	2.479	2.484	2.493
G_{mb}	2.390	2.384	2.401	2.410
Air Voids (%)	4.0	3.9	3.4	3.0
P_{ba} (%)	0.79	0.59	0.73	0.66
P_{be} (%)	4.55	4.70	4.65	4.37

Construction

The location of the project was approximately 9 miles from the plant, which resulted in about a 15-20 minute haul time for the trucks. Construction of the HMA began at the north end of CR 513 at the intersection of US-2 and continued in the southbound lane the length of the project. The HMA test section examined for this study ends approximately 4.2 miles from the beginning of the project. The Advera mix was produced in the northbound lane parallel to the HMA. The Evotherm surface mix was paved in the northbound lane between approximately 4.5 to 5.9 miles from the beginning of the project. As stated earlier, the HMA extends the entire southbound lane, so visual comparisons of the HMA to the two WMA technologies are possible. The existing asphalt roadway was pulverized and recycled in-place to create the new base. Then a new intermediate asphalt pavement course was placed before the construction of the surface mixes. All three surface mixes had a target thickness of 2 inches. Figure 65 shows the location of the test sections.

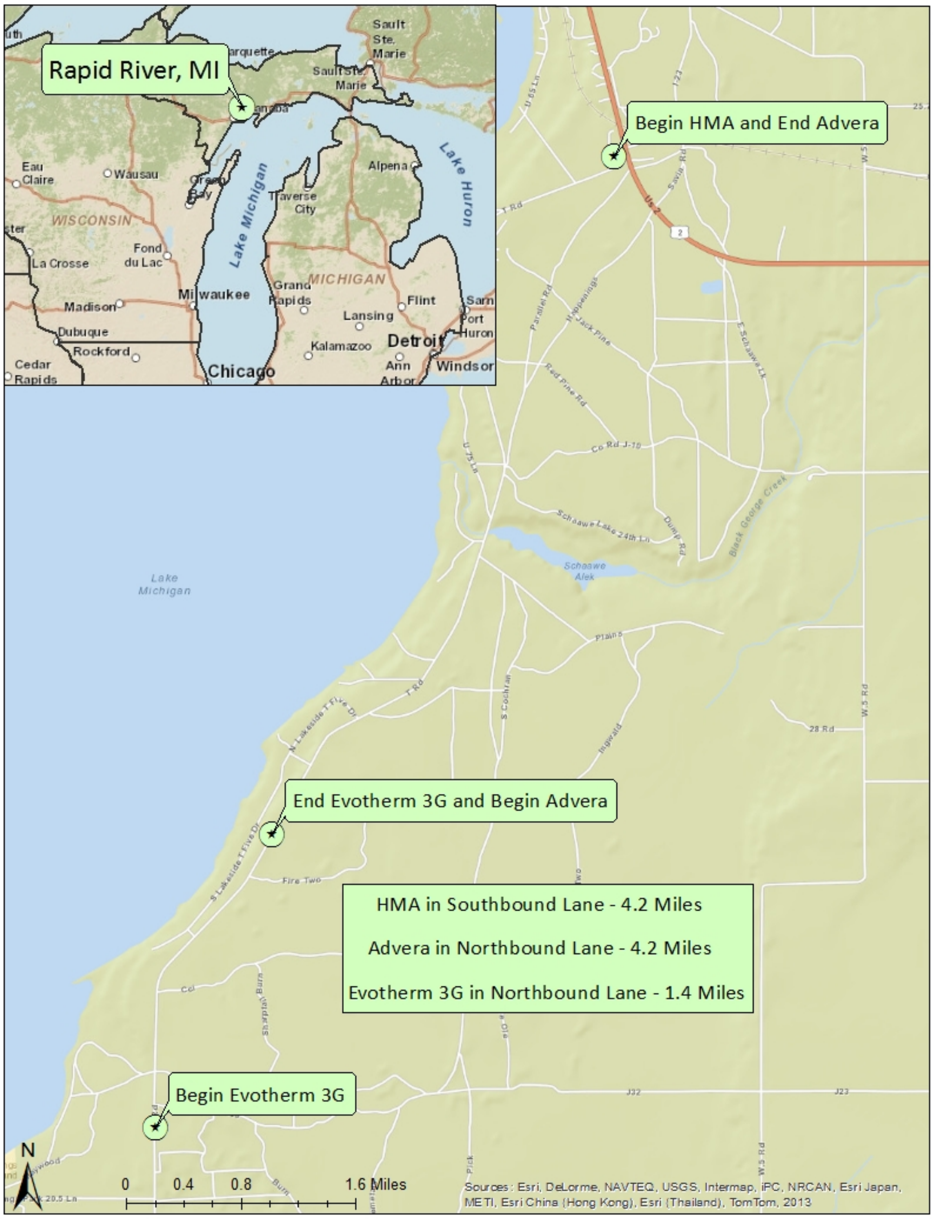


Figure 65 Location of Test Sections in Rapid River, Michigan

The temperature of the mix behind the paver was measured using both a hand-held temperature gun and the PAVE-IR system. Table 94 shows the temperatures from behind the screed using both measuring techniques. Since the PAVE-IR system takes continuous readings some differences are expected as compared to the periodic measurements using the temperature gun. For the temperature gun measurements, several readings were taken and the results averaged to give one temperature reading for that point in time.

Table 94 Temperatures behind the Screed

Statistic	Measuring Device	HMA	Advera	Evotherm
Average (°F)	Temperature Gun	N/A	269.9	248.0
	PAVE-IR	255.0	227.0	239.0
Standard Deviation (°F)	Temperature Gun	N/A	8.3	6.7
	PAVE-IR	16.4	12.3	14.4
Maximum (°F)	Temperature Gun	N/A	282.0	255.0
	PAVE-IR	300.0	278.0	274.0
Minimum (°F)	Temperature Gun	N/A	262.0	237.0
	PAVE-IR	185.0	189.0	204.0

Weather data was collected hourly at the paving location using a handheld weather station. Ambient temperature, wind speed, and humidity were recorded are shown in Table 95.

Table 95 Weather Conditions during Construction in Rapid River, MI

Measurement	Statistic	HMA	Advera	Evotherm
Ambient Temperature (°F)	Average	66.2	82.8	79.4
	Range	60.8 - 71.6	64.6 – 90.6	77.6 – 81.1
Wind Speed (mph)	Average	3.2	1.5	2.2
	Range	0 – 5.4	0 – 3.0	1.0 – 3.6
Humidity (%)	Average	78.0	57.9	61.1
	Range	68.0 – 94.0	30.2 – 85.9	54.3 – 74.7

Three rollers were used for compaction of all three mixes, and the rolling pattern was kept the same throughout. The breakdown performed five passes, in vibratory mode up and static mode back. The intermediate roller was a rubber tire roller that rolled continuously within its operating range. The finishing roller was a steel wheel roller that performed three passes in the static mode.

Construction Core Testing

After construction of each mix, seven 4-inch (101.6-mm) cores were obtained from all three sections. Core densities were determined in accordance with AASHTO T 166. If the water absorption was determined to be higher than 1 percent, the samples were then tested according to AASHTO T 331. Six of the cores from each mix were also tested for tensile strength according to ASTM D6931. Average test results are shown in Table 96.The average core densities for the three mixes were very consistent and reasonable. The tensile strengths are consistent, but low due to the soft virgin binder (PG 52-34) used on the project.

Table 96 Construction Cores Test Results for Rapid River, MI

Property	Statistic	HMA	Advera	Evotherm
In-place Density (%)	Average	94.1	95.0	94.3
	Standard Dev.	1.0	0.6	0.9
Tensile Strength (psi)	Average	53.5	58.5	49.8
	Standard Dev.	3.5	4.4	3.7

Field Performance at 13-Month and 22-Month Project Inspections

A field performance inspection was conducted on August 10, 2011, after about 13 months of traffic were applied to the test sections. A second inspection was conducted on June 19, 2012 after about 22 months of traffic. Data were collected on each section to document rutting, cracking, and raveling. Three 6-inch (150 mm) diameter cores were taken from the right wheelpath, and four 6-inch (150 mm) diameter cores were taken between the wheelpaths to determine the in-place density, indirect tensile strengths, theoretical maximum specific gravity, gradation, asphalt content, and the recovered true binder grade for each mix.

The rut depths were measured at the beginning of each 200 ft. (61 m) evaluation section with a straight edge and a wedge. None of the mixes had any measurable rutting at the time of either inspection.

Each evaluation section was carefully inspected for visual signs of cracking. The HMA section had no cracking at the time of the first inspection. At the second inspection, only one non-wheelpath, longitudinal crack about one foot in length was observed in one of the HMA evaluation sections. For the Advera mix, one small longitudinal crack about 0.5 feet (0.15 m) in length was evident during the first inspection. No other cracks had developed in the Advera sections at the time of the second inspection. For the Evotherm 3G mix, the first evaluation section contained two, non-wheelpath longitudinal cracks totaling one foot in length. The second evaluation section contained no visual cracking, and the third section had a small longitudinal crack less than a foot long. No other cracks had propagated in any of the Evotherm sections after 22 months. Overall, all three mixes were performing very well in terms of cracking.

The surface textures of the HMA and WMA test sections were measured using the sand patch test. The calculated mean and standard deviations of texture depths for each mix are shown in Table 97.

Table 97 Mean Texture Depths for Rapid River, MI

Mix	13-Month Inspection		22-Month Inspection	
	Mean Texture Depth (mm)	Standard Dev. (mm)	Mean Texture Depth (mm)	Standard Dev. (mm)
HMA	0.34	0.03	0.30	0.03
Advera	0.30	0.01	0.30	0.02
Evotherm 3G	0.40	0.04	0.39	0.05

These results show similar mean texture depths for all three mixes. The Evotherm section had a slightly higher mean texture depth, which indicates it has experienced the most weathering as compared to the other two mixes. The Advera mix performed the best in terms of weathering. All three mixes had similar results at both inspections. The results of the sand patch test show that all three mixes have performed well in terms of raveling and weathering. Figure 66, Figure 67 and Figure 68 show examples of the surface of the HMA, Advera, and Evotherm 3G sections respectively at the time of the 22-month inspection.

Figure 66 HMA Control Section at 22-Month Inspection in Rapid River, MI

Figure 67 Advera Section at 22-Month Inspection in Rapid River, MI

Figure 68 Evotherm 3G Section at 22-Month Inspection in Rapid River, MI

Core Testing

At the time of each project inspection, cores were taken near the construction cores. The testing procedures used were the same as previous projects. A summary of results for the core testing from the 13-month inspection compared to the construction data is shown in Table 98.

The gradations were similar for all mixes. The asphalt contents at the first inspection were slightly higher for all mixes compared to the production mixes. This difference can probably be attributed to the difference between loose-mix and cores. All three mixes exhibited similar asphalt contents at the first inspection. The 13-month inspection cores had higher densities compared to the construction cores due to densification under traffic. The HMA averaged 3.5 percent higher density compared to the construction cores, while the Advera and Evotherm 3G averaged 1.5 percent and 2.6 percent higher density, respectively, at the 13-month inspection. The maximum specific gravities for all three mixes were slightly higher than was tested at construction. This may have been due to the binder wearing off the surface, continued binder absorption over time, or both. The tensile strengths from the one-year inspection were very similar to those tested at construction. The Advera section had a slight increase in tensile strength after one year.

Table 98 Test Results from Production Mix and 13-Month Cores in Rapid River, MI

Property	HMA	Advera	Evotherm	HMA	Advera	Evotherm
	Production Mix (July 2010)			13-Month Cores (August 2011)		
Sieve Size	% Passing			% Passing		
19.0 mm (3/4")	100.0	100.0	100.0	100.0	100.0	100.0
12.5 mm (1/2")	94.2	94.5	95.0	95.9	93.9	94.8
9.5 mm (3/8")	86.0	86.7	84.2	88.1	87.5	87.6
4.75 mm (#4)	67.3	68.0	63.9	71.1	70.3	68.7
2.36 mm (#8)	50.7	51.3	48.4	53.6	54.1	52.1
1.18 mm (#16)	37.6	37.9	36.1	37.5	39.0	37.0
0.60 mm (#30)	26.1	26.3	25.5	26.0	27.9	26.3
0.30 mm (#50)	17.4	17.8	17.6	16.6	18.1	17.3
0.15 mm (#100)	9.5	9.9	10.1	9.5	9.8	9.4
0.075 mm (#200)	5.7	6.0	6.4	6.1	5.8	5.7
Asphalt Content (%)	5.26	5.34	5.00	5.55	5.41	5.48
G_{mm}	2.479	2.484	2.483	2.485	2.499	2.495
G_{mb}	2.333*	2.359*	2.341*	2.424	2.412	2.417
In-place Density (%)	94.1*	95.0*	94.3*	97.6	96.5	96.9
P_{ba} (%)	0.59	0.73	0.66	0.88	1.04	1.01
Tensile Strength (psi)	53.5*	58.5*	49.8*	47.7	67.2	53.9

*Data comes from construction cores, not mix sampled during production as specified in column header.

The results from the 13-month and 24-month inspection are presented in Table 99. The gradations for all three mixes were similar and did not change significantly since the first inspection. The asphalt contents were also similar for the test sections and appear to have slightly decreased between inspections. This can probably be attributed to variability in sampling and testing since other properties and characteristics have changed very little between inspections. The in-place densities of all three mixes were high after 13-months of traffic and have not changed significantly between inspections. The average tensile strengths for all three mixes have increased slightly between inspections as expected due to binder stiffening.

Table 99 Test Results from 13-Month and 22-Month Cores in Rapid River, MI

Property	HMA	Advera	Evotherm	HMA	Advera	Evotherm
	13-Month Cores (August 2011)			22-Month Cores (June 2012)		
Sieve Size	% Passing			% Passing		
19.0 mm (3/4”)	100.0	100.0	100.0	100.0	100.0	100.0
12.5 mm (1/2”)	95.9	93.9	94.8	95.5	93.6	95.1
9.5 mm (3/8”)	88.1	87.5	87.6	88.4	86.4	87.1
4.75 mm (#4)	71.1	70.3	68.7	69.3	68.4	66.3
2.36 mm (#8)	53.6	54.1	52.1	52.4	52.5	50.7
1.18 mm (#16)	37.5	39.0	37.0	36.7	38.0	36.2
0.60 mm (#30)	26.0	27.9	26.3	25.2	27.2	25.6
0.30 mm (#50)	16.6	18.1	17.3	16.5	18.1	17.0
0.15 mm (#100)	9.5	9.8	9.4	9.2	10.0	9.2
0.075 mm (#200)	6.1	5.8	5.7	5.6	5.8	5.3
Asphalt Content (%)	5.55	5.41	5.48	5.31	5.23	5.14
G_{mm}	2.485	2.499	2.495	2.488	2.502	2.502
G_{mb}	2.424	2.412	2.417	2.402	2.426	2.402
In-place Density (%)	97.6	96.5	96.9	96.6	97.0	96.0
P_{ba} (%)	0.88	1.04	1.01	0.78	0.97	0.91
Tensile Strength (psi)	47.7	67.2	53.9	71.1	78.9	66.3

Table 100 shows the average density and tensile strength results by location for the cores from both inspections. As noted for the as-constructed cores, the in-place densities for the test sections were high and remained high at the time of both inspections. The wheelpath cores had slightly higher densities compared to those between the wheelpaths for the HMA and Evotherm sections as expected. However, for the Advera section, the average density in the wheelpaths was slightly lower than between the wheelpaths at the time of both inspections. The tensile strengths for all three mixes were similar for wheelpath and between wheelpath cores. Tensile strengths increased as expected between the first and second inspection for all of the sections.

Table 100 In-place Density and Tensile Strengths by Location in Rapid River, MI

Property	HMA	Advera	Evotherm	HMA	Advera	Evotherm
	13-Month Cores			22-Month Cores		
Between Wheelpaths Density (% of G_{mm})	97.4	97.1	96.7	95.9	97.2	95.7
In Right Wheelpath Density (% of G_{mm})	97.8	95.8	97.1	97.4	96.6	96.5
Between Wheelpaths Tensile Strength (psi)	50.3	68.3	55.2	72.5	77.0	63.4
In Right Wheelpath Tensile Strength (psi)	45.1	66.0	52.6	69.8	80.8	67.2

Performance Prediction

The initial AADTT for CR-513 near Rapid River, MI was 60 trucks per day with one lane in each direction. The MEPDG suggests a typical minimum of 100 trucks per day and this was used in the analysis. A growth factor of 0.3 percent was calculated based on the future traffic prediction shown on the project plans. CR-513 was classified as a local route. Table 101summarizes the pavement structure. The MEPDG would not accept the Evotherm dynamic modulus data. The 14 °F data was stiffer than the HMA; the data at the other four test temperatures was less stiff. A Level 2 analysis was used for the Evotherm mix.

Table 101 CR-513 Rapid River, MI Pavement Structure

Layer	Thickness, in. [cm]
WMA/HMA surface course	1.5 [3.8]
WMA/HMA Intermediate Course (Same as surface mix)	2.0 [5.1]
Cold recycled asphalt – pulverized in-place modulus 20,000 psi	6.0 [15.2]
AASHTO A-6 Subgrade	Semi-infinite

Figure 69 shows a comparison of the predicted rutting for the WMA and HMA sections. The rut depth after 20-years of service was predicted to be 0.08 in. (2 mm) for both the HMA and Evotherm sections and 0.05 in. (1.3 mm) for the Advera section.

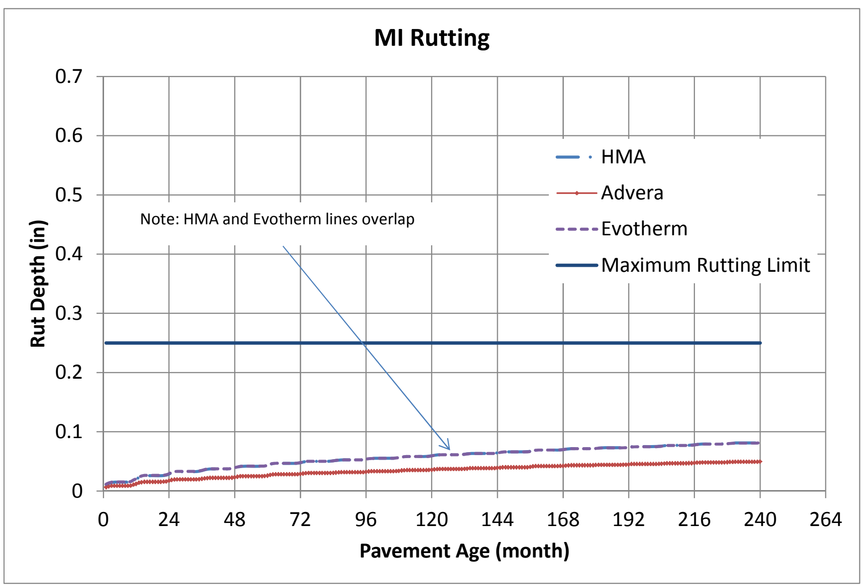


Figure 69 MEPDG Predicted Asphalt Rutting for CR-513 Rapid River, MI

Figure 70 compares the predicted longitudinal cracking for CR-513 over the design life. The MEPDG predicts 550, 139, and 434 ft./mile (104, 26, 82 m/km) of longitudinal cracking for the HMA, Advera WMA, and Evotherm WMA mixes, respectively after 20-years of service. One obvious difference between the Advera WMA and the other two mixes is in-place density. The Advera WMA averaged 5.0 percent voids at the time of construction while the Evotherm and HMA averaged 5.7 and 5.9 percent respectively. As noted previously, a Level 2 analysis was used for the Evotherm.

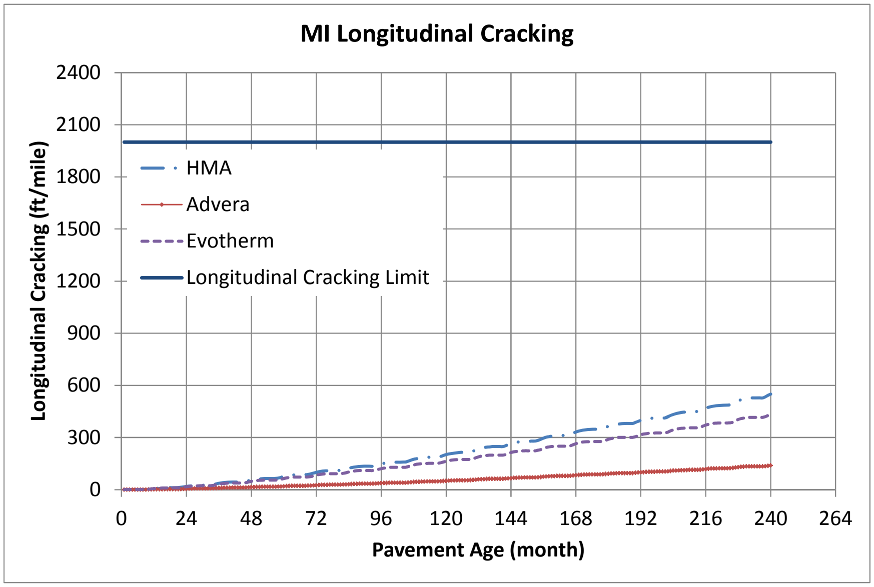


Figure 70 MEPDG Predicted Longitudinal Cracking for CR-513 Rapid River, MI

Baker, Montana

A WMA field project was constructed in August 2010 on Montana Route 322 in Fallon County, approximately 7 miles south of Baker, Montana. The WMA technology used on this project was the chemical additive Evotherm DAT produced by MeadWestvaco Asphalt Innovations. This section of Route 322 has an estimated two-way AADT of only 430 vehicles per day with 12 percent trucks. The production of the HMA and WMA test sections took place on August 11 and 12, 2010 respectively. The contractor for this project was Prince Inc., Forsyth, MT

The asphalt mixture used for this trial consisted of a fine-graded 19.0 mm NMAS Superpave mix design with a compactive effort of 75 gyrations. The mix design used for the HMA was also used for the WMA without any changes. The aggregate used for the design was a virgin crushed gravel blend with no RAP. The materials percentages used for mix design submittal and production are shown in Table 102. Both mixes used a polymer modified PG 64-28 asphalt binder. Hydrated lime was used as an anti-stripping agent in both mixes. The design JMF and limits are shown in Table 103.

Table 102 Aggregate Percentages Used in Mix Design for Baker, MT

Aggregate Type	Mix Design (%)	Production, HMA (%)	Production, WMA (%)
Coarse Gravel	39.4	39.4	41.4
⅜" Gravel	13.8	13.8	11.8
Crushed Fines	45.4	45.4	45.2
Hydrated Lime	1.4	1.4	1.6

Table 103 Design Gradation, Asphalt Content, and Volumetrics for Mix Design for Baker, MT

Property	JMF	Limits
Sieve Size	% Passing	
19.0 mm (3/4")	100	90 - 100
12.5 mm (1/2")	81	90
9.5 mm (3/8")	69	--
4.75 mm (#4)	51	--
2.36 mm (#8)	31	23 - 49
1.18 mm (#16)	20	--
0.60 mm (#30)	14	--
0.30 mm (#50)	10	--
0.15 mm (#100)	7	--
0.075 mm (#200)	5	2 - 8
AC (%)	5.8	--
Air Voids (%)	3.73	3.4 – 4.0
VMA (%)	15.2	13.0 min.
VFA (%)	75.5	65 - 78
D/A Ratio	0.99	0.6 – 1.6
P_{be} (%)	5.11	--
P_{ba} (%)	0.73	--

Production

The WMA was produced by metering in the Evotherm DAT at the plant at a rate of 0.5 percent by weight of binder. Figure 71 and Figure 72 show the metering system and point of Evotherm DAT entry respectively. Table 104 shows the production temperatures recorded in the tower for both mixes.

The plant used for both mixes was a portable parallel-flow drum plant that used liquid propane as fuel. The plant incorporated a Hauck burner with a Boeing Drum and CEI binder tanks. The plant had only one silo. The plant is shown in Figure 73 and Figure 74. It should be noted that during production of both mixes, the aggregate stockpile were very dry as was the plant location in general, which caused very dusty conditions on-site.

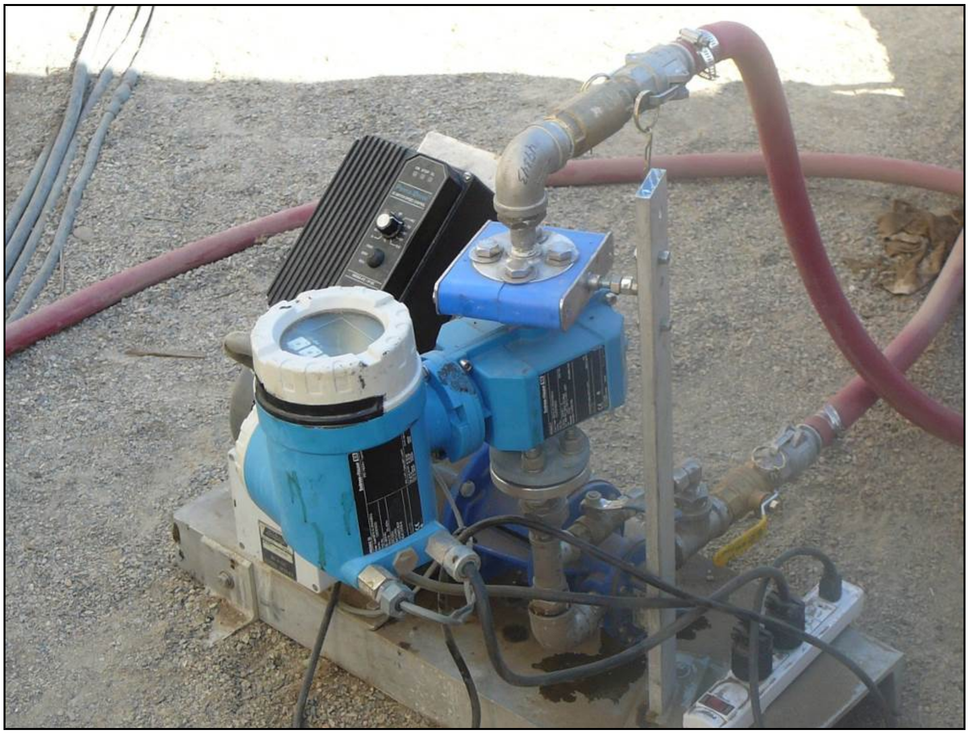


Figure 71 Evotherm DAT Metering System

Figure 72 Point of Evotherm DAT Entry

Table 104 Production Temperatures in Baker, MT

Statistic	HMA	Evotherm DAT
Average, °F	298.2	261.9
Standard Dev., °F	3.4	7.7
Maximum, °F	304.0	286.0
Minimum, °F	292.0	252.0

Figure 73 Portable Parallel Flow Drum Plant in Baker, MT

Figure 74 Portable Parallel Flow Drum Plant in Baker, MT

Volumetric Mix Properties

Samples of each mixture were obtained during production to compare moisture contents, percent coating, and volumetric properties between the HMA and WMA. Samples were taken from trucks leaving the plant.

The average moisture contents were 0.18 and 0.09 percent for the HMA and WMA, respectively. These results are both low and reasonable. Although the average moisture content of the HMA was slightly higher than the WMA, the difference can likely be attributed to sampling and testing variability.

The percent of coated particles using AASHTO T 195 was 98.0 and 99.0 percent for the HMA and WMA mixes respectively. Thus, the WMA and HMA exhibited similar coating characteristics and incomplete coating was not a concern for either mixes.

Specimens were compacted using 75 gyrations in the Superpave gyratory compactor at compaction temperatures of 270°F for the HMA samples and 235°F for the WMA samples. These laboratory compaction temperatures were determined using the average compaction temperature observed on the test section through the first couple of hours of construction for each mixture. These volumetric samples were compacted on-site in the NCAT mobile laboratory so that the mixes would not have to be reheated. Bulk specific gravity of the compacted specimens (G_{mb}) was determined in accordance with AASHTO T 166. The gradation of the extracted aggregate was determined according to AASHTO T 30. Average test results are summarized in Table 105.

The measured asphalt content of both mixes was very close to the JMF value of 5.8 percent. The gradation of both mixes was determined to be slightly finer than the JMF, but were within the allowable control points. Both mixes contained about 1 percent less dust (P_{200}) than the JMF. The air voids of the HMA were low and out of tolerance, whereas the WMA was in tolerance.

Table 105 Gradation, Asphalt Content, and Volumetrics for Plant-Produced Mix from Baker, MT

Property	JMF	HMA	Evotherm DAT	Control Points
Sieve Size	% Passing			
19.0 mm (3/4”)	100.0	100.0	100.0	90 - 100
12.5 mm (1/2”)	81.0	87.3	89.1	90
9.5 mm (3/8”)	69.0	75.5	75.2	--
4.75 mm (#4)	51.0	55.3	53.9	--
2.36 mm (#8)	31.0	33.8	32.9	23 - 49
1.18 mm (#16)	20.0	22.0	20.6	--
0.60 mm (#30)	14.0	14.5	13.4	--
0.30 mm (#50)	10.0	10.0	9.2	--
0.15 mm (#100)	7.0	6.6	6.2	--
0.075 mm (#200)	5.0	4.1	4.0	2 - 8
Asphalt Content (%)	5.80	5.69	5.76	--
G_{mm}	2.412	2.413	2.407	--
G_{mb}	2.322	2.341	2.313	--
Air Voids (%)	3.7	3.0	4.0	3.4 - 4.0
VMA (%)	15.2	14.4	15.5	13 min
VFA (%)	75.6	79.2	74.2	65 – 78
Dust/Binder	0.99	0.82	0.78	0.6 – 1.6
P_{ba} (%)	0.78	0.72	0.65	--
P_{be} (%)	5.06	5.01	5.14	--

Construction

The section of Route 322 being paved while NCAT was on-site began at the intersection with Montana SR 7 South. The HMA was placed in both lanes starting at the intersection with SR 7 South going to approximately 2.6 miles east of the intersection. The WMA began there after the 600 tons of HMA were placed on the morning of August 12. The WMA section paved while NCAT was on-site was in the eastbound lane only and terminated approximately 6.7 miles from the intersection of SR 7 South. Figure 75 shows the location of the test sections. The target thickness for both surface mixes was 1.5 inches. The surface mixes were placed as an overlay over an existing asphalt pavement layer. Both the HMA and WMA test sections were paved as the surface layer and were topped with a chip seal approximately 8 months after construction. It is typical for all pavements in this area to be topped with a chip seal within the first year.

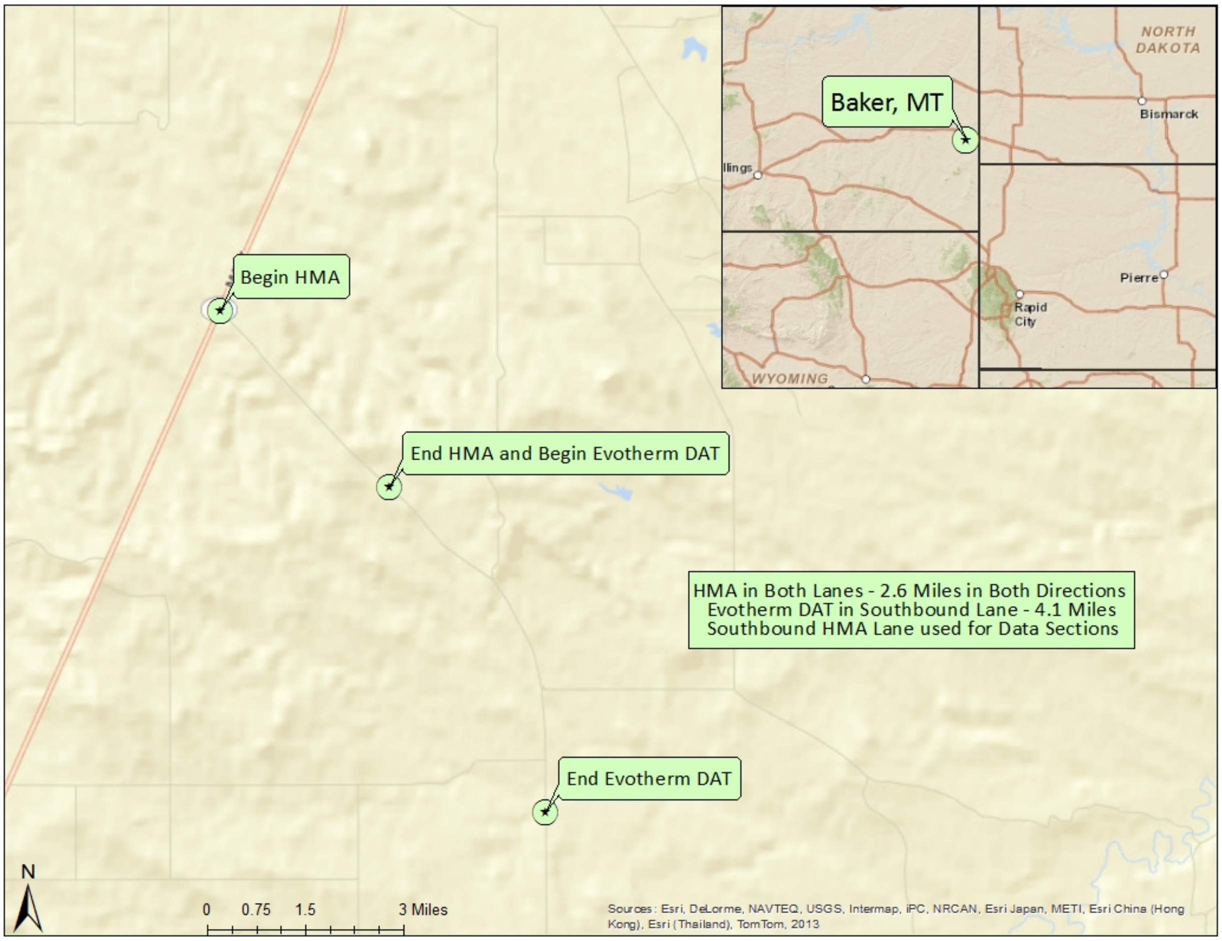


Figure 75 Location of Test Sections in Baker, Montana

Weather data were collected hourly at the paving location using a handheld weather station. There was no rain during the construction of either mix, and both the plant and paving locations were very dry.

The same three rollers were used to compact both mixes, and the rolling patterns were kept the same. The breakdown and intermediate rollers used were both Dynapac CC-772 steel wheel rollers operated in the vibratory mode. A Dynapac CC-552 was used as the finishing roller operated in the static mode.

Table 106 shows the ambient temperatures, wind speed, and humidity for both mixes produced.

Table 106 Weather Conditions during Construction in Baker, MT

Measurement	Statistic	HMA	Evotherm DAT
Ambient Temperature (°F)	Average	88.7	81.8
	Range	68.0 - 96.1	71.1 – 87.1
Wind Speed (mph)	Average	14.3	9.3
	Range	5.8 – 18.4	4.6 – 12.7
Humidity (%)	Average	23.3	43.8
	Range	14.0 – 42.0	34.0 - 68.0

Construction Core Testing

After construction, seven 4-inch (101.6-mm) cores were obtained from both sections. Core densities were determined in accordance with AASHTO T 166. Six of the cores from each mix were also tested for tensile strength according to ASTM D6931. Average test results are shown in Table 107.

Average core densities were almost identical for both mixes, as were the tensile strengths. The tensile strengths for both mixes seem a bit low, but this is more than likely due to the soft binder and no RAP contained in these mixes.

Table 107 Construction Cores Test Results from Baker, MT

Property	Statistic	HMA	Evotherm DAT
In-place Density (%)	Average	91.3	91.2
	Standard Deviation	1.1	1.7
Tensile Strength (psi)	Average	67.6	65.5
	Standard Deviation	7.2	7.9

Field Performance at 13-Month and 22-Month Project Inspections

A field-performance evaluation was conducted on September 7, 2011, after about 13 months of traffic were applied to the test sections. A second performance evaluation was performed on June 21, 2012 after about 22 months of traffic were applied. As stated earlier, this portion of Route 322 near Baker had been topped with a chip seal over the test sections as is typical for similar roads in this area. Data were collected on each section to document performance regarding rutting and cracking. Raveling could not be analyzed on these mixes because of the chip seal. Evaluation sections were selected as described for previous projects. For the HMA and Evotherm DAT sections, three 4-inch (101.6 mm) diameter cores were taken from the right wheelpath, and five 4-inch (101.6 mm) diameter cores were taken from in between the wheelpath. The chip seal was cut off the top of the test mixes. Then these cores were used to determine the in-place density after 13 months, indirect tensile strengths, theoretical maximum specific gravity, gradation and asphalt content.

The HMA section exhibited an average of 0.3 mm of rutting between the three random locations at the time of the first inspection. The WMA section had an average of 0.2 mm of rutting at the first inspection. At the time of the second inspection, the WMA had the same average rut depth, and the HMA section had increased slightly to 0.5 mm. Both sections performed very well in terms of rutting.

Each 200 ft. (61 m) evaluation section was carefully inspected for visual signs of cracking. None of the evaluation sections in either mix section had any visible cracking through the chip seal at the time of the first inspection. At the time of the second inspection, some slight cracking was found in both sections. In one of the HMA sections, a low-severity transverse crack was observed which ran across the entire roadway, suggesting that it was probably reflective or thermal cracking. However, it could not be determined if the mix was the cause of the cracking since the section was topped with the chip seal. In one of the WMA sections, two similar low-severity transverse cracks were observed to extend across the entire roadway. These cracks summed to a total of 12 ft. (3.7 m) for the HMA and 24 ft. (7.3 m) for the WMA. Figure 76 shows an example of the cracking observed in both mix sections. Figure 77 and Figure 78 show the surface of the HMA and WMA sections respectively. It can be seen that the sections appear identical due to the chip seal applied to both sections.

Figure 76 Example of Low-Severity Transverse Cracking at 22-Month Revisit in Baker, MT

Figure 77 HMA Control Section at 22-Month Revisit in Baker, MT

Figure 78 Evotherm DAT Section at 22-Month Revisit in Baker, MT

Core Testing

At the time of each project inspection, eight 4-inch (101.6 mm) cores were taken from each mix. A summary of the 13-month core testing compared to the construction data is shown in Table 108.

The gradations for both mixes were similar at each point in time. Although the dust contents appeared to decrease over time, this change is likely due to sampling and testing variability. The asphalt contents for both mixes from the one-year inspection were almost 1 percent higher than tested at construction. This was probably due to some asphalt from the chip seal remaining on cores after trimming. The asphalt contents at 22-months were similar for both mixes and a little closer to the as-constructed results. The 13-month and 22-month cores had slightly higher average densities as compared to the construction cores. The maximum specific gravities for both mixes were slightly lower than was tested at construction. This is probably because the chip seal binder was not completely removed from the samples, which caused the maximum specific gravities to decrease slightly. The tensile strengths for the one-year cores were slightly lower than the cores tested at construction. The average tensile strengths decreased by 8.5 psi and 14.0 psi for the HMA and WMA, respectively. The tensile strengths of the 22-month cores form the HMA and WMA sections were similar and higher, which is likely due to aging.

Table 109 shows the average densities and tensile strength results by location for both inspections. The average densities were higher in the wheelpaths for both sections as expected. At the time of the first inspection, the tensile strength of the WMA was lower in the right wheelpaths than between the wheelpaths. However, at the second inspection, the tensile strengths were slightly higher in the wheelpaths for both mixes. However, the difference is not considered significant.

Table 108 Test Results on Production Mix, 13-Month and 22-Month Cores from Baker, MT

Property	HMA	Evotherm DAT	HMA	Evotherm DAT	HMA	Evotherm DAT
	Production Mix (August 2010)		13-Month Cores (September 2011)		22-Month Cores (June 2012)	
Sieve Size	% Passing		% Passing		% Passing	
19.0 mm (3/4”)	100.0	100.0	100.0	100.0	100.0	100.0
12.5 mm (1/2”)	87.3	89.1	92.5	94.7	92.9	87.3
9.5 mm (3/8”)	75.5	75.2	81.5	85.6	82.9	78.0
4.75 mm (#4)	55.3	53.9	59.6	61.6	61.6	58.0
2.36 mm (#8)	33.8	32.9	36.1	37.2	38.2	37.4
1.18 mm (#16)	22.0	20.6	21.9	22.2	23.5	23.0
0.60 mm (#30)	14.5	13.4	14.7	14.6	15.5	15.4
0.30 mm (#50)	10.0	9.2	9.6	9.4	9.8	10.0
0.15 mm (#100)	6.6	6.2	6.2	5.9	5.8	5.8
0.075 mm (#200)	4.1	4.0	3.9	3.6	3.2	3.2
Asphalt Content (%)	5.69	5.76	6.52	6.79	6.06	6.12
G_{mm}	2.413	2.407	2.393	2.378	2.391	2.399
G_{mb}	2.218*	2.195*	2.240	2.236	2.240	2.236
In-place Density (%)	91.3*	91.2*	93.6	94.0	93.7	93.3
P_{ba} (%)	0.72	0.65	0.87	0.75	0.53	0.72
Tensile Strength (psi)	67.6*	65.5*	59.1	51.5	78.9	70.4

*Data comes from construction cores, not mix sampled during production as specified in column header.

Table 109 In-Place Density and Tensile Strengths by Location in Baker, MT

Location and Property	HMA	Evotherm DAT	HMA	Evotherm DAT
	13-Month		22-Month	
Between Wheelpaths Density (% of G_{mm})	93.5	93.5	93.1	92.5
In Right wheelpath Density (% of G_{mm})	93.8	95.0	94.7	94.5
Between Wheelpaths Tensile Strength (psi)	60.1	53.9	75.7	69.8
In Right wheelpath Tensile Strength (psi)	57.9	48.2	83.2	71.4

Performance Prediction

The initial AADTT for CR-322 near Baker, MT was 52 trucks per day with one lane in each direction. Montana DOT reported a growth rate of 2.6 percent. CR-322 is classified as a local route. Table 110 summarizes the pavement structure. Cores and ground penetrating radar indicated that the total asphalt thickness for the HMA was 0.5 in. (12.7 mm) thicker than the WMA section; the distribution of layer thicknesses varies as well.

Table 110 CR-322 Baker, MT Pavement Structures

Layer	WMA Thickness, in. [cm]	HMA Thickness, in. [cm]
WMA/HMA surface course	1.8[4.6]	1.6 [4.1]
Existing HMA – 12.5 NMAS with PG 64-28	2.2 [5.6]	1.8 [4.6]
Existing HMA – 12.5 NMAS with PG 64-28	1.9 [4.8]	1.7[4.3]
Existing HMA – 12.5 NMAS with PG 64-28	NA	1.3 [3.3]
AASHTO A-4 Subgrade	Semi-infinite	

Figure 79 shows a comparison of the predicted rutting for the WMA and HMA sections. The predicted total asphalt rutting after 20-years of service is practically identical for the WMA and HMA, 0.13 and 0.14 in. (3.3 and 3.6 mm) respectively). The predicted rutting for the WMA layer is actually slightly less than the HMA, 0.02 versus 0.03 in. (0.5 versus 0.8 mm) respectively).

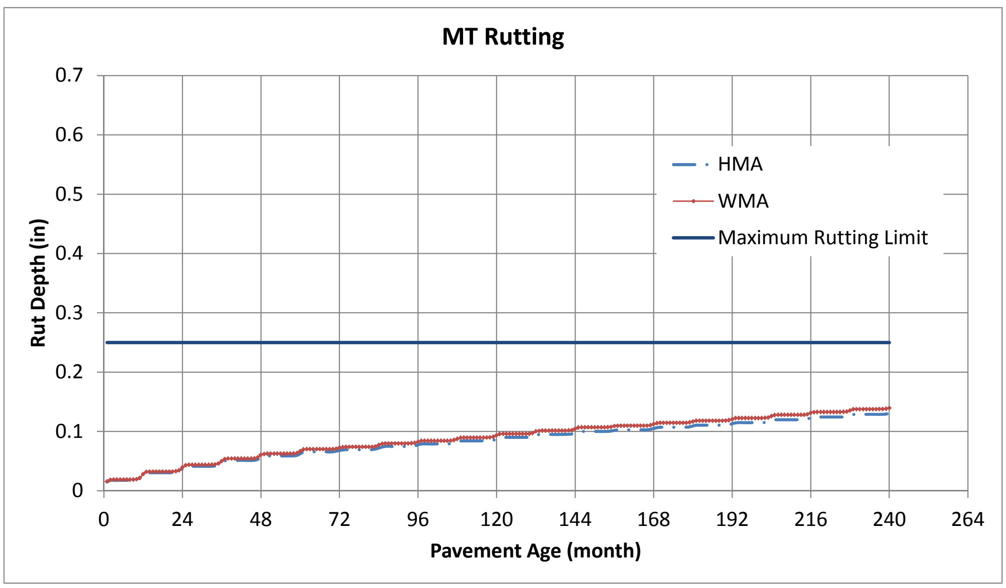


Figure 79 MEPDG Predicted Asphalt Rutting for CR-322 Baker, MT

Figure 80 compares the predicted longitudinal cracking for CR-322 over the design life. The MEPDG predicts more cracking for the WMA compared to the HMA, 1,030 versus 822 ft./mile (195 and 156 m/km) at 20-years of service. This may, in part, be due to the difference in pavement thickness. Level 1 thermal cracking analysis was performed for this project. Figure 81 shows a comparison of the predicted thermal cracking for the WMA and HMA. The HMA is predicted to exceed the 1000 ft./mile (189m/km) threshold one year earlier than the WMA (67 versus 78 months).

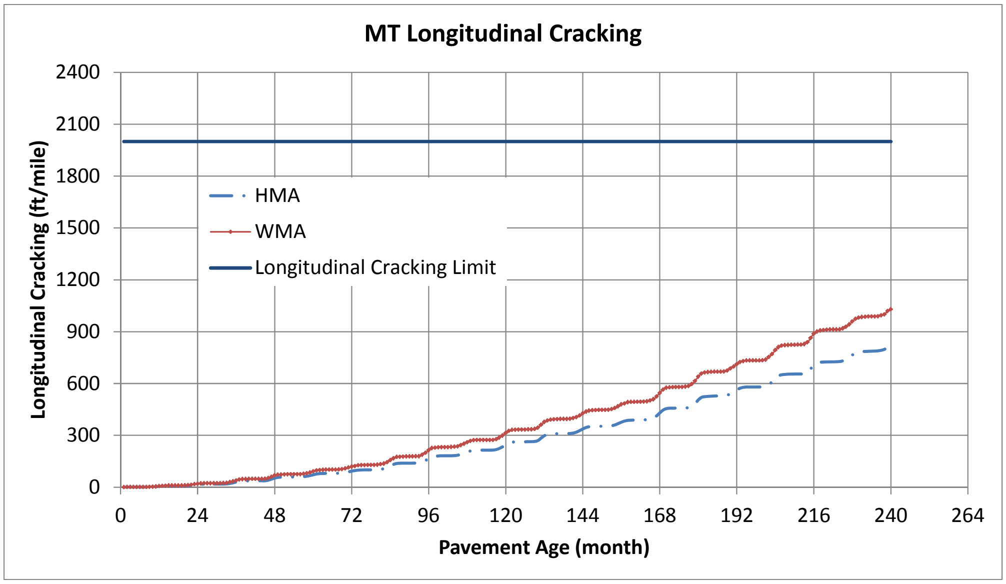


Figure 80 MEPDG Predicted Longitudinal Cracking for CR-322 Baker, MT

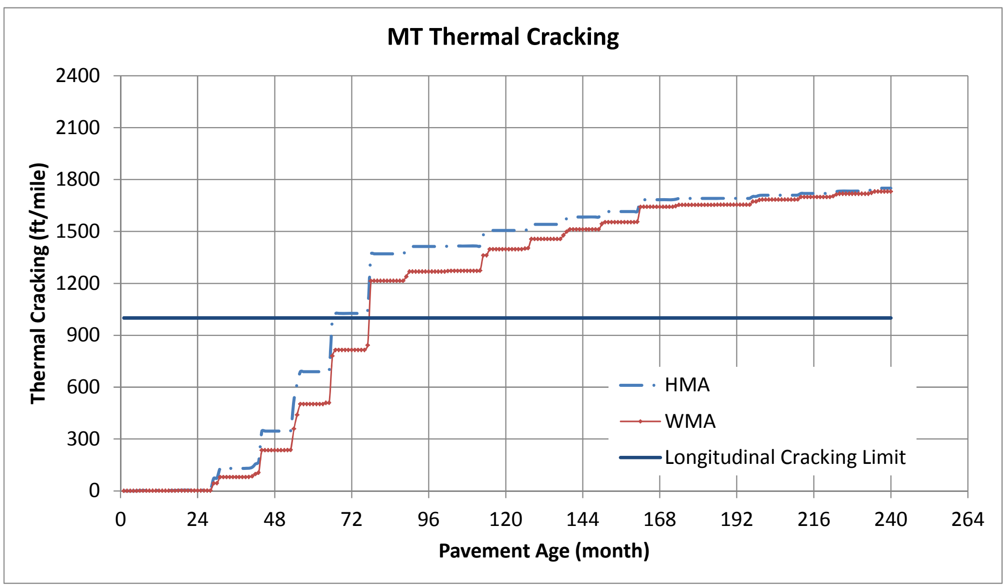


Figure 81 MEPDG Predicted Thermal Cracking for CR-322 Baker, MT

Munster, Indiana

A WMA trial project was constructed on Calumet Avenue in Munster, Indiana in September 2010. The contractor for this WMA trial was Walsh & Kelley, Inc., Griffith, IN. The project featured three different WMA technologies. The first WMA technology was the water foaming system manufactured by Gencor Industries, Inc. under the trade name Ultrafoam GX2, also known as The Green Machine. The second WMA technology used was the chemical additive Evotherm 3G developed by MeadWestvaco Asphalt Innovations. The last WMA technology was a wax product made by the Heritage Environmental Services, LLC. The HMA and all three

WMA technologies were placed on Calumet Avenue from the intersection of Main Street heading northbound for approximately one mile. There are four main travel lanes on this portion of roadway, each of which contains one of the trial mixes The estimated two-way AADT for this four-lane roadway was calculated to be 37,986 vehicles per day with 7.1 percent trucks. The production of the HMA and Ultrafoam GX2 took place on September 14 and 15, 2010 respectively, while the Evotherm 3G and Heritage wax were produced and placed on September 16, 2010.

The asphalt mixture used for this trial consisted of a coarse-graded 9.5 mm NMAS Superpave mix design with a compactive effort of 75 gyrations. The mix design used for the HMA was also used for all WMA technologies without any changes. All four mixtures contained limestone, slag sand, and 15 percent RAP. The RAP consisted of multiple-source millings that were fractionated into two stockpiles in order to have better control of the material. The material percentages used for mix design and production are shown in Table 111. A PG 64-22 asphalt binder supplied by British Petroleum was used as the virgin binder for all mixes. The JMF, optimum asphalt content, and specifications are shown in Table 112.

Table 111 Aggregate Percentages for Munster, IN

Aggregate Type	Mix Design (%)
11 Limestone	48
FM 21	10
Slag Sand	25
RAP	15
BH Dust	2

Table 112 Design Gradation, Asphalt Content, and Volumetrics for Mix Design for Munster, IN

Property	JMF	Specification
Sieve Size	% Passing	
12.5 mm (1/2")	100.0	100
9.5 mm (3/8")	92.0	90-100
4.75 mm (#4)	54.0	<90
2.36 mm (#8)	41.0	32-67
1.18 mm (#16)	30.0	--
0.60 mm (#30)	22.0	--
0.30 mm (#50)	15.0	--
0.15 (#100)	10.0	--
0.075 mm (#200)	6.0	2-10
AC (%)	5.50	--
Air Voids (%)	4.0	--
VMA (%)	15.4	--
VFA (%)	73.9	--
D/A Ratio	1.23	--
P_{be} (%)	4.87	--
P_{ba} (%)	0.66	--

Production

The first WMA process used for this field evaluation was the Ultrafoam GX2 system, which injects water into the virgin binder to create foaming that temporarily expands the asphalt volume. This allows for maximum coating of the aggregate as well as improved compactability at lower temperatures. For this field evaluation, water was injected at a rate of 2 percent by weight of virgin binder. The Ultrafoam GX2 system is shown in Figure 82.

Figure 82 Ultrafoam GX2 Foaming System used in Munster, IN

The next WMA process used on this field evaluation was Evotherm 3G. The Evotherm chemical was introduced via a mass-flow meter at the plant at a rate of 0.5 percent by weight of liquid binder. The final WMA technology used was Heritage organic wax additive. This material was terminal blended with the PG 64-22 liquid binder. Once mixed, the wax bumped the binder grade to PG 70-22.

Table 113 shows the production temperatures for all four mixes. The asphalt plant used to produce the asphalt mixtures was an Astec counter-flow drum mix plant. Figure 83 shows the asphalt plant used for this field trial.

Table 113 Production Temperatures in Munster, IN

Statistic	HMA	Ultrafoam GX2	Evotherm 3G	Heritage Wax
Average (°F)	300.4	276.5	255.6	267.5
St. Deviation (°F)	10.0	7.9	6.3	11.3
Maximum (°F)	320	288	267	277
Minimum (°F)	290	265	248	243

Figure 83 Counter-Flow Drum Plant in Griffith, IN

Volumetric Mix Properties

Samples of each mixture were obtained during production to compare moisture contents, percent coating, and volumetric properties between the HMA and WMA. Samples were taken from a mini-stockpile made each day specifically for sampling.

The average moisture contents were 0.26, 0.44, 0.47, and 0.52 percent for the HMA, Ultrafoam GX2, Evotherm 3G, and Heritage wax respectively. These moisture contents results are somewhat high for two reasons. First, it rained overnight prior to production of the mixes. Secondly, the limestone used is known to be highly absorptive, which means there was residual moisture in the aggregate that was not completely removed in the drier. It was expected that the WMA mixes might have slightly higher mix moisture contents due to the lower mix production temperatures, which could leave more residual moisture in the aggregate or RAP going through the plant as compared to the HMA mixture.

The percent of coated particles was 100.0, 99.0, 99.0 and 98.0 for the HMA, Ultrafoam GX2, Evotherm 3G, and Heritage wax mixes, respectively. This shows that even at lower production temperatures, the WMA technologies had coating characteristics similar to the HMA.

Specimens were compacted using 75 gyrations in the SGC at compaction temperatures of 285°F, 240°F, 230°F, and 240°F for the HMA, Ultrafoam GX2, Evotherm 3G, and Heritage wax mixes, respectively. These laboratory compaction temperatures were determined using the average temperature at the start of rolling during the first couple of hours of construction for each mixture. These volumetric samples were compacted on-site in the NCAT mobile laboratory so that the mixes would not have to be reheated. Average test results for the plant produced mixtures are summarized in Table 114.

The asphalt content results for all mixes were higher than the JMF values, with the HMA having the largest difference from the JMF (0.68 percent). All of the WMA technologies had asphalt contents within 0.5 percent of the JMF value. The gradations for all four mixes were within the specification limits. Most sieves were very close to the JMF gradation except for the #4 and #200 sieves. All four mixes were about 6 percent finer on the #4 sieve, and all mixes except for the Evotherm mix, contained about 1 percent more dust (P_{200}) than the JMF. The percent of absorbed asphalt (P_{ba}) was significantly higher for the four plant-produced mixes compared to the value computed from the JMF. This is most likely related to the maximum specific gravities (G_{mm}) for the four mixes being higher than the JMF value. The air void contents for each of the mixes were higher than the design value of 4.0 percent. However, the bulk specific gravity (G_{mb}) values were very similar to the JMF. Therefore, the differences in air voids can be attributed to the differences in maximum specific gravity values.

Table 114 Gradation, Asphalt Content, and Volumetrics for Plant-Produced Mix in Munster, IN

Property	JMF	HMA	Foam	Evo.	Wax	Spec
Sieve Size	% Passing					
12.5 mm (1/2”)	100.0	99.8	99.6	99.8	99.6	100
9.5 mm (3/8”)	92.0	94.0	93.5	93.8	94.2	90-100
4.75 mm (#4)	54.0	61.5	62.1	60.3	61.2	<90
2.36 mm (#8)	41.0	39.6	40.8	38.9	40.0	32-67
1.18 mm (#16)	30.0	28.6	28.6	26.7	28.1	--
0.60 mm (#30)	22.0	19.6	19.9	17.8	19.6	--
0.30 mm (#50)	15.0	13.5	13.7	11.5	13.4	--
0.15 mm (#100)	10.0	9.5	9.6	7.6	9.4	--
0.075 mm (#200)	6.0	6.9	7.0	5.6	7.0	2-10
Asphalt Content (%)	5.50	6.18	5.61	5.95	5.95	--
G_{mm}	2.499	2.526	2.525	2.517	2.531	--
G_{mb}	2.398	2.386	2.383	2.357	2.407	--
Air Voids (%)	4.0	5.6	5.6	6.4	4.9	--
P_{ba} (%)	0.66	1.58	1.18	1.27	1.51	--

Construction

The HMA and three WMA technologies were all placed on Calumet Avenue in Munster, Indiana from the intersection of Main Street to approximately one-mile north on Calumet Avenue. This portion of Calumet Avenue was approximately 6 miles from the plant, which was located in Griffith, Indiana. However, the travel time to the site was approximately 20 – 45 minutes due to the high volume of traffic in the area. The HMA and Ultrafoam GX2 foam mixes were placed in the southbound outside and northbound outside lanes respectively. The Evotherm and Heritage wax mixes were placed in the northbound inside and southbound inside lanes respectively. The four test mixes were placed as the surface (wearing) course and had a target thickness of 1.5 inches. All four lanes had been milled and then had a new intermediate asphalt pavement course paved before placement of the surface mixes. Figure 84 illustrates the location of the test sections.

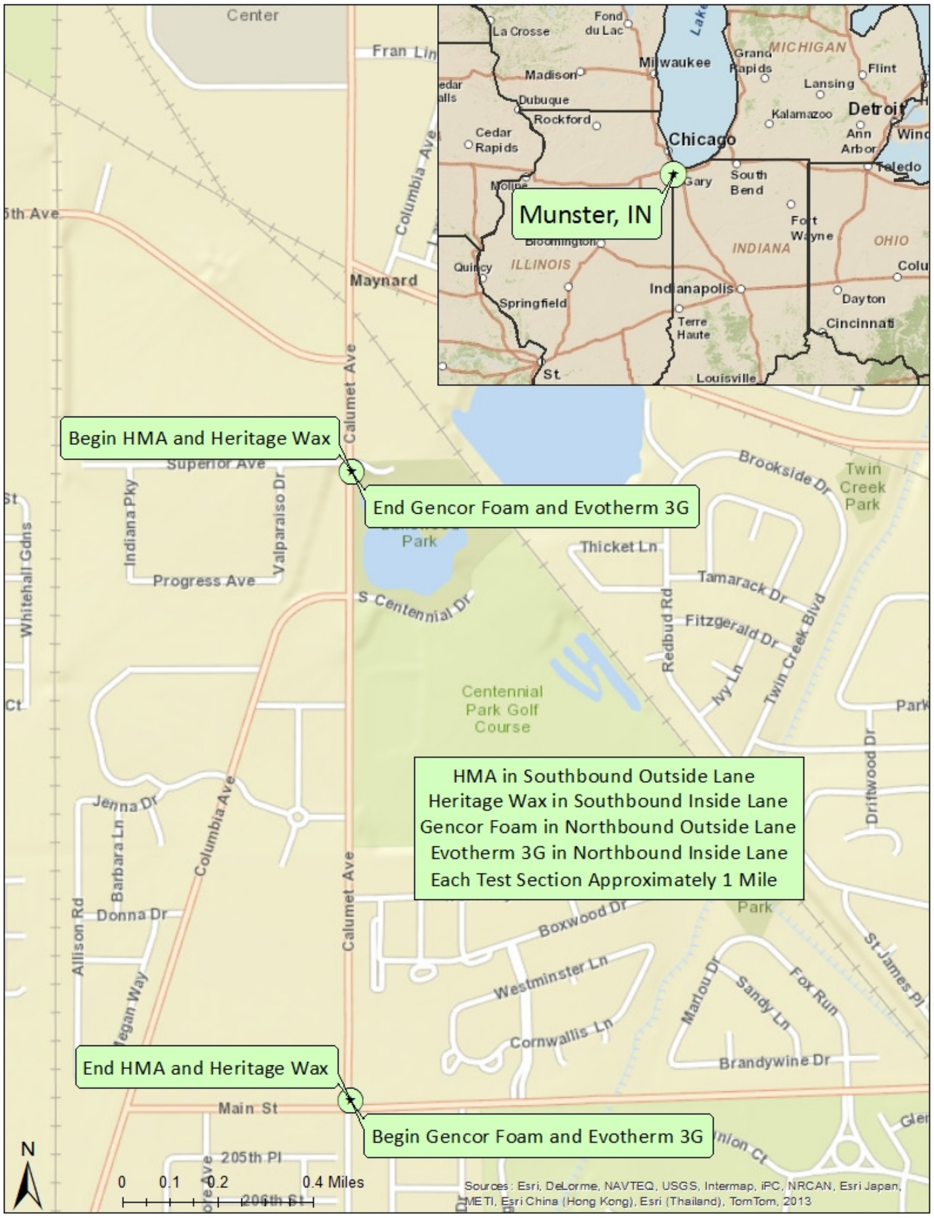


Figure 84 Location Test Sections in Munster, Indiana

The asphalt mixes were delivered using a cycle of nine tarped dump trucks that discharged the material directly into the paver. Figure 85 shows a truck dumping into the paver.

Figure 85 Truck Dumping into Caterpillar AP-1055D Paver

The temperature of the mix behind the paver was measured using a hand-held temperature gun and the PAVE-IR system. Table 115 shows the temperatures from behind the screed using both measuring techniques. Since the PAVE-IR system takes continuous readings some differences are expected as compared to the periodic temperature gun readings. When taking the temperature gun reading periodically, several readings were taken and the results averaged to give one temperature reading for that point in time.

Table 115 Temperatures behind the Screed in Munster, IN

Temperature (°F)	Measuring Device	HMA	Foam	Evotherm	Wax
Average	Temperature Gun	282.9	259.5	233.5	245.3
	PAVE-IR	249.0	222.0	210.0	235.0
Standard Deviation	Temperature Gun	6.2	7.0	4.2	11.1
	PAVE-IR	13.1	13.9	13.4	13.0
Maximum	Temperature Gun	291.3	266.0	239.3	259.3
	PAVE-IR	280.0	258.0	248.0	267.0
Minimum	Temperature Gun	272.3	247.7	226.3	224.0
	PAVE-IR	210.0	179.0	158.0	171.0

Weather data were collected hourly at the paving location using a handheld weather station. Ambient temperature, wind speed, and humidity was recorded are shown in Table 116.

Table 116 Weather Conditions during Construction in Munster, IN

Measurement	Statistic	HMA	Foam	Evotherm*	Wax*
Ambient Temperature (°F)	Average	81.4	75.5	72.5	72.5
	Range	72.3 – 87.1	59.9 – 90.1	70.2 – 75.1	70.2 – 75.1
Wind Speed (mph)	Average	2.0	4.4	3.8	3.8
	Range	0 – 2.7	1.5 – 9.0	2.2 – 4.7	2.2 – 4.7
Humidity (%)	Average	39.9	46.5	67.1	67.1
	Range	32.8 – 64.7	23.5 – 70.2	51.5 – 84.1	51.5 – 84.1

* The Evotherm and Wax sections were constructed on the same day.

All four mixes were compacted using two rollers, and the rolling pattern was approximately the same for all mixes. Both of these rollers were steel wheel rollers operated in the vibratory mode. The breakdown roller was a Hamm HD-110HV, and the finishing roller was a Hamm HD-14.

Construction Core Testing

Test results on the construction cores are shown in Table 117. The average core densities for the HMA and Heritage Wax were approximately 1.7 percent lower than the Ultrafoam GX2 foam and Evotherm sections. The tensile strengths for the three WMA mixes were similar, but were about 10-psi higher than the HMA.

Table 117 Construction Cores Test Results from Munster, IN

Property	Statistic	HMA	Foam	Evotherm	Wax
In-place Density (% of G_{mm})	Average	88.7	90.3	90.4	88.7
	Standard Deviation	1.5	1.6	2.2	2.9
Tensile Strength (psi)	Average	89.5	101.0	105.6	98.3
	Standard Deviation	14.8	15.1	12.0	18.6

Field Performance at 13-Month and 24-Month Project Inspections

Field-performance evaluations were conducted on October 18, 2011, after about 13 months of traffic, and on September 18, 2012 after about 24 months of traffic. Data were collected on each section to document performance regarding rutting, cracking, and raveling.

The rut depths were measured at the beginning of each 200 ft. (61 m) evaluation section with a straight edge and a wedge. No measurable rutting was detected in any of the test sections at the time of either inspection.

Each evaluation section was carefully examined in each inspection for visual signs of cracking. At the time of the first inspection, a 1ft. (0.3 m), low-severity (< 6 mm wide), transverse crack was observed in one of the HMA evaluation sections. At the second inspection, this crack had progressed to three feet in length, but was still low severity. Another 11 ft. (3.4 m) long crack was also observed in an HMA evaluation section at the time of the second inspection. This non-wheelpath, longitudinal crack was also low severity. The Ultrafoam GX2 foam section had four low-severity transverse cracks at the time of the first inspection. These four cracks totaled eight feet in length. There were also four longitudinal cracks in the foam sections, totaling a length of 11 ft. (3.4 m). All of these cracks were low severity and were not in the wheelpath. At the time of the second inspection, the total length of transverse cracking in the foam sections had progressed to 20 ft. (6.1m), with five cracks. The non-wheelpath longitudinal cracking had progresses to 97 ft. (29.6m) with a total of 11 cracks. All of these cracks were still low severity. Although the foam sections had a good deal more cracking as compared to the other mixes, none of the longitudinal cracks were in the wheelpath for either of the two mixes that had cracking, so it is thought that the cracks are probably not fatigue related. In addition, most of the cracks had been sealed in the foam section. According to the *Distress Identification Manual for the Long-Term Pavement Performance Program,* they are considered low severity since they are sealed. Figure 86 shows an example of a transverse crack that had been sealed. All three Evotherm 3G and Heritage wax sections exhibited no cracking at the time of either inspection. It should be noted that the two mixes that exhibited cracking (HMA and Ultrafoam GX2) were in the outside lanes, while the other two with no cracking were in the inside lanes. Figure 87 shows an example of the non-wheelpath longitudinal cracking observed at the time of the 24-month inspection.

Figure 86 Example of Low-Severity Transverse Crack in Munster, IN

Figure 87 Example of Low-Severity Non-Wheelpath Longitudinal Crack in Munster, IN

The surface textures of both the HMA and WMA test sections were measured using the sand patch test according to ASTM E965. The calculated mean texture depths for each mix are shown in Table 118.

Table 118 Mean Texture Depths for Munster, IN

Mix	13-Month Revisit		24-Month Revisit	
	Mean Texture Depth (mm)	Standard Deviation (mm)	Mean Texture Depth (mm)	Standard Deviation (mm)
HMA	0.60	0.07	0.58	0.06
Evotherm 3G	0.53	0.03	0.51	0.04
Ultrafoam GX2	0.52	0.01	0.52	0.03
Heritage Wax	0.55	0.07	0.56	0.05

These results show similar mean texture depths for all four mixes. The HMA had a slightly higher mean texture depth at both inspections which indicates a slightly greater amount of raveling than the WMA sections. The wax WMA had the second highest mean texture depth. Overall, the results of the sand patch tests indicate that all four mixes have performed well in terms of raveling and weathering. Figure 88 shows the surface of the Ultrafoam, Evotherm 3G, Heritage wax, and HMA sections from left to right.

Figure 88 Foam, Evotherm, Wax, and HMA Sections Respectively in Munster, IN

Core Testing

A summary of the core testing from the 13-month inspection compared to the production data is shown in Table 119. The asphalt contents of the HMA and Heritage Wax 13-month cores were substantially lower than the results from the production samples. The results of the 13-month cores are more consistent with the G_{mm} results and the slightly higher raveling in the HMA section. The cores had higher densities compared to the construction cores. This increase in density was expected due to traffic densification. The increase in density for the HMA was 4.2 percent compared to the construction cores, while the Evotherm 3G, Ultrafoam, and Heritage wax sections increased by 2.6, 3.7, and 4.2 percent, respectively. The maximum specific gravities for all four mixes were very similar to the values measured on the mix sampled at construction. The average tensile strengths of the 13-month inspection cores improved for all four mixes as compared to the cores tested at construction. This was probably due to the increase in densities and stiffening of the binder because of aging. The tensile strengths of the three WMA technologies were all higher than the HMA at both construction and the first inspection. The tensile strengths were similar and acceptable for all mixes at the first inspection.

Table 119 Test Results on Production Mix and 13-Month Cores from Munster, IN

Property	HMA	Foam	Evo.	Wax	HMA	Foam	Evo.	Wax
	Production Mix (September 2010)				13-Month Cores (October 2011)			
Sieve Size	% Passing				% Passing			
19.0 mm (3/4")	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5 mm (1/2")	99.8	99.6	99.8	99.6	99.8	99.9	99.7	99.9
9.5 mm (3/8")	94.0	93.5	93.8	94.2	94.4	94.5	94.2	93.6
4.75 mm (#4)	61.5	62.1	60.3	61.2	62.9	63.5	62.3	59.0
2.36 mm (#8)	39.6	40.8	38.9	40.0	41.1	42.5	41.0	38.9
1.18 mm (#16)	28.6	28.6	26.7	28.1	29.0	29.6	27.9	27.1
0.60 mm (#30)	19.6	19.9	17.8	19.6	21.3	21.7	20.0	19.7
0.30 mm (#50)	13.5	13.7	11.5	13.4	14.7	15.2	13.4	13.5
0.15 mm (#100)	9.5	9.6	7.6	9.4	10.3	10.7	9.1	9.4
0.075 mm (#200)	6.9	7.0	5.6	7.0	7.5	7.9	6.5	6.7
Asphalt Content (%)	6.18	5.61	5.95	5.95	5.34	5.55	5.71	5.42
Avg. Prod. Temp. (°F)	300.4	276.5	255.6	267.5	300.4	276.5	255.6	267.5
G_{mm}	2.526	2.525	2.517	2.531	2.542	2.545	2.533	2.537
G_{mb}	2.242*	2.279*	2.276*	2.244*	2.357	2.367	2.356	2.357
In-place Density (%)	88.7*	90.3*	90.4*	88.7*	92.9	93.0	93.0	92.9
P_{ba} (%)	1.58	1.18	1.27	1.51	1.29	1.48	1.39	1.26
Tensile Strength (psi)	89.5*	101.0*	105.6*	98.3*	104.6	108.8	119.3	120.0

*Data from construction cores, not mix sampled during production as identified by the column header.

The results from the 13-month and 24-month inspections are presented in Table 120. The gradations are similar for all four mixes. The average asphalt contents for the 24-month cores were slightly higher than the 13-month cores and generally more consistent with the results from the as-produced samples, but the differences are likely due to sampling and testing variability. The in-place densities for all four sections were very similar and had not changed significantly between inspections. The tensile strength increased for all four mixes between inspections. The strengths at both inspections were reasonable for all mixes.

Table 120 Tests Results on 13-Month and 24-Month Cores in Munster, IN

Property	HMA	Foam	Evo.	Wax	HMA	Foam	Evo.	Wax
	13-Month Cores (October 2011)				24-Month Cores (September 2012)			
Sieve Size	% Passing				% Passing			
19.0 mm (3/4”)	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5 mm (1/2”)	99.8	99.9	99.7	99.9	99.6	99.7	99.7	99.6
9.5 mm (3/8”)	94.4	94.5	94.2	93.6	95.6	93.9	94.9	94.9
4.75 mm (#4)	62.9	63.5	62.3	59.0	65.8	62.3	64.2	62.5
2.36 mm (#8)	41.1	42.5	41.0	38.9	42.2	41.5	42.6	41.6
1.18 mm (#16)	29.0	29.6	27.9	27.1	28.9	28.6	29.1	28.2
0.60 mm (#30)	21.3	21.7	20.0	19.7	20.7	20.5	20.7	20.0
0.30 mm (#50)	14.7	15.2	13.4	13.5	13.8	14.0	14.0	13.4
0.15 mm (#100)	10.3	10.7	9.1	9.4	9.3	9.6	9.5	9.0
0.075 mm (#200)	7.5	7.9	6.5	6.7	6.4	6.8	6.7	6.2
Asphalt Content (%)	5.34	5.55	5.71	5.42	5.95	5.62	5.82	5.81
Avg. Prod. Temp. (°F)	300.4	276.5	255.6	267.5	300.4	276.5	255.6	267.5
G_{mm}	2.542	2.245	2.533	2.537	2.533	2.542	2.537	2.535
G_{mb}	2.357	2.367	2.356	2.357	2.368	2.378	2.367	2.363
In-place Density (%)	92.9	93.0	93.0	92.9	93.5	93.5	93.3	93.2
P_{ba} (%)	1.29	1.48	1.39	1.26	1.55	1.48	1.53	1.49
Tensile Strength (psi)	104.6	108.8	119.3	120.0	123.8	143.2	129.7	131.5

Table 121 shows the average density and tensile strength results by location for the cores from both inspections. The average densities in the wheelpaths are very similar to the average densities measured in between the wheelpaths for all three WMA technologies. The HMA had about 3 percent higher density in the wheelpath at both inspections. For all four mixes, the average tensile strength between the wheelpaths was slightly greater than in the wheelpath.

Table 121 In-place Density and Tensile Strengths by Location in Munster, IN

Location and Property	HMA	Foam	Evo.	Wax	HMA	Foam	Evo.	Wax
	13-Month Cores				24-Month Cores			
Between Wheelpaths Density (% of Gmm)	91.1	93.5	93.2	93.0	91.8	93.6	93.6	93.4
In Right wheelpath Density (% of Gmm)	94.0	92.7	92.9	92.8	94.6	93.5	93.0	93.1
Between Wheelpaths Tensile Strength (psi)	108.6	116.1	129.1	135.8	128.3	170.2	156.6	150.5
In Right wheelpath Tensile Strength (psi)	101.9	103.9	112.7	109.5	120.8	125.3	111.8	118.8

Performance Predictions

The initial AADTT for Calumet Avenue, Munster, IN was 2,697 trucks per day with two lanes in each direction. A growth factor of 1.8 percent was calculated based on historical traffic data. Calumet Avenue/US-45 was classified as a principal arterial. For the MEPDG analysis, the same traffic was used for all sections even though the Evotherm and Heritage wax were placed in the passing lanes. Observations on site indicate that trucks used both lanes. Table 122 summarizes the pavement structure used for the analyses.

Table 122 Calumet Avenue Munster, IN Pavement Structure

Layer	Thickness, in. [cm]
WMA/HMA surface course	2.1 [5.3]
HMA – 12.5 mm NMAS with PG 64-22	1.8 [4.6]
Existing HMA – 19.0 mm NMAS with PG 64-22	4.0 [10.2]
AASHTO A-7-6 Subgrade	Semi-infinite

Figure 89 shows a comparison of the predicted rutting in all of the asphalt layers for the WMA and HMA sections. Figure 90 shows the predicted rutting in the surface layers only. The MEPDG predicts that the cumulative rutting in all of the asphalt layers will reach 0.25 in. (6.4 mm) after 70 months of services. The total cumulative rutting in the asphalt layers predicted after 20-years of service is 0.49 in. (12.4 mm) for the HMA and 0.50 in. (12.7 mm) for all of the WMA sections. Similarly, the predicted rutting in the surface layer is 0.10 in. (2.5 mm) for the HMA, Evotherm, and Heritage wax and 0.11 in (2.8mm) for the Foam section. Essentially, the predicted rutting performance for all of the mixes is the same.

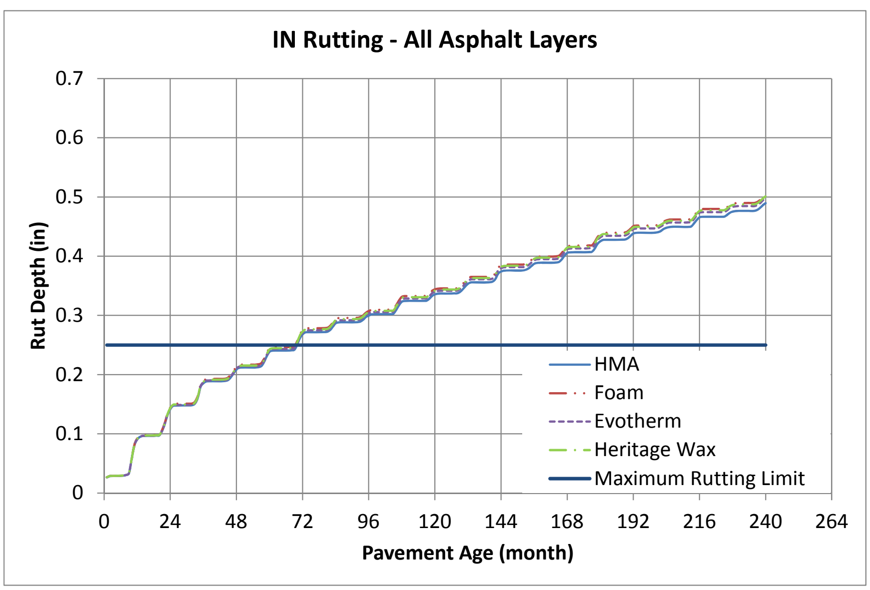


Figure 89 MEPDG Predicted Rutting in all Asphalt Layers for Calumet Ave., Munster, IN

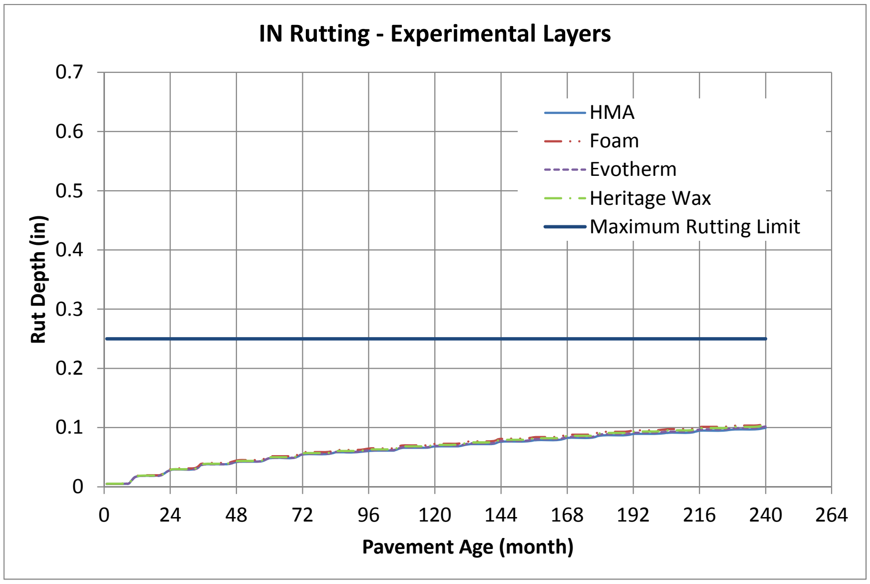


Figure 90 MEPDG Predicted Rutting in Experimental (Surface) Layers for Calumet Ave., Munster, IN

Figure 91 compares the predicted longitudinal cracking for Calumet Avenue/US-45 over the design life. The predicted top-down, longitudinal cracking exceeds the design limit of 2000 ft./mi. (379 m/km) for all of the sections. The Heritage Wax has the worst predicted performance, followed by the HMA, Gencor Foam, and Evotherm with cracking exceeding 2000 ft./mi. (379 m/km) predicted after 24, 34, 35, and 37 months, respectively.

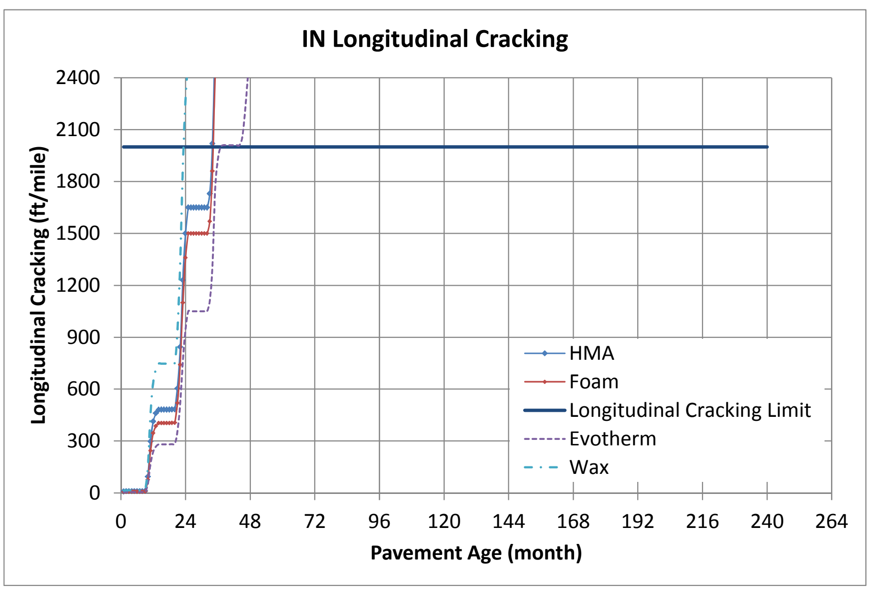
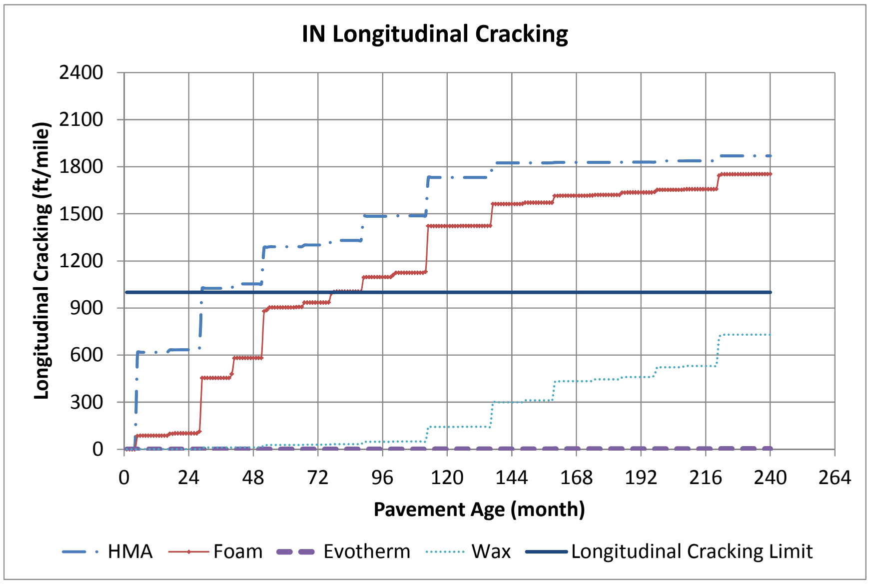


Figure 91 MEPDG Predicted Longitudinal Cracking for Calumet Ave., Munster, IN

Level I IDT thermal cracking inputs were available for the Munster, IN project. The predicted thermal cracking is presented in Figure 92. All of the WMA technologies performed better than the HMA. The Evotherm performed the best followed by the Wax and Foam mixtures. Interestingly, this corresponds to the measured production and placement temperatures (Table 115).



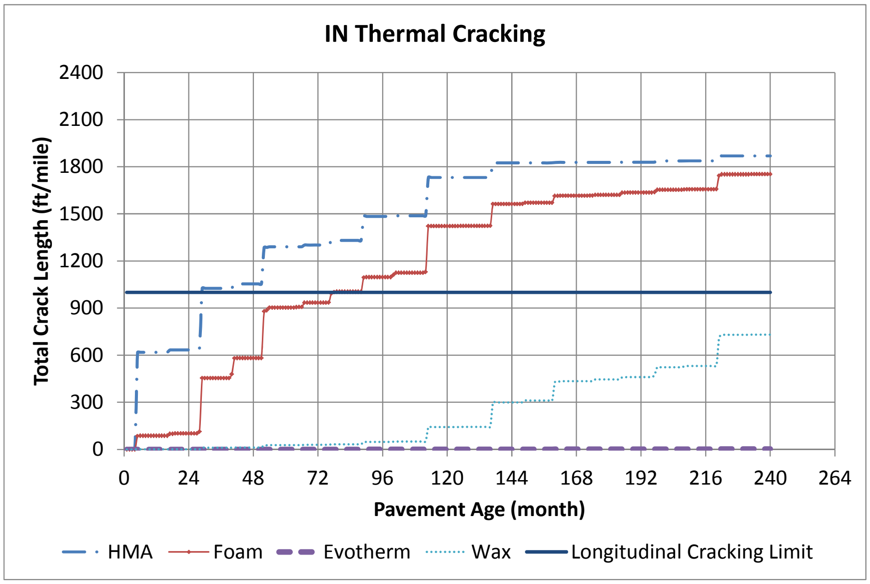


Figure 92 MEPDG Predicted Thermal Cracking for Calumet Ave., Munster, IN

Jefferson County, Florida

A WMA trial project was constructed on US-98 in Jefferson County, Florida southeast of Tallahassee in October 2010. The WMA technology used on this project was the water injection asphalt foaming system developed by Terex Roadbuilding. This WMA technology is referred to as the Terex WMA system. This section of US-98 has an estimated two-way AADT of 1,950 with 41 percent trucks. The production of the WMA and companion HMA control took place on October 6 and 7, 2010 with C.W. Roberts Contracting Inc., Tallahassee, FL as the contractor.

The asphalt mixture used for this trial consisted of a fine-graded 12.5 mm NMAS Superpave mix design, with a compactive effort of 75 gyrations. The mix design used for the HMA was also used for the WMA without any changes. The aggregate used for the design was a granite and sand blend including 20 percent crushed RAP. The material percentages used for mix design submittal and production are shown in Table 123. Both mixes used a polymer modified PG 76-22 asphalt binder. No anti-strip agent was used on this project for either mix. The laboratory and production JMFs, optimum asphalt contents, specifications, and allowable tolerances are shown in Table 124.

Table 123 Aggregate Percentages Used in Mix Design and Production in Jefferson County, Florida

Aggregate Type	Mix Design (%)	Production (%)
#78 Stone	24	24
#89 Stone	16	21
W-10 Screenings	20	23
M-10 Screenings	10	9
Local Sand	10	8
Crushed RAP	20	15

Table 124 Design Gradation, Asphalt Content, and Volumetrics for Mix Design in Jefferson County, Florida

Sieve Size	JMF	Control Points
	% Passing	
19.0 mm (3/4”)	100.0	100
12.5 mm (1/2")	100.0	90 – 100
9.5 mm (3/8")	89.0	--
4.75 mm (#4)	63.0	--
2.36 mm (#8)	46.0	28 – 58
1.18 mm (#16)	35.0	--
0.60 mm (#30)	27.0	--
0.30 mm (#50)	15.0	--
0.15 mm (#100)	8.0	--
0.075 mm (#200)	5.4	2 - 10
AC (%)	5.3	--
Air Voids (%)	4.0	--
VMA (%)	14.8	--
VFA (%)	72.9	--
D/A Ratio	1.19	--
P_{be} (%)	4.55	--
P_{ba} (%)	0.79	--

Production

The WMA was produced using Terex WMA system shown in Figure 88. The foaming allows for maximum coating of the aggregate as well as improved compactability at lower temperatures. For this field evaluation, water was injected at a rate of 2 percent by weight of virgin binder.

Figure 93 Terex WMA System used in Jefferson Co., FL

Table 125 shows the average production temperature for both mixes. The asphalt plant used to produce the asphalt mixes was a counter-flow Terex CMI drum mix plant that incorporated two asphalt storage silos. The plant used recycled waste oil for the burner fuel. Figure 94 shows the asphalt plant used for this field trial.

Table 125 Production Temperatures in Jefferson Co., FL

Temperatures (°F)	HMA	Terex Foam
Average	336.3	296.9
Standard Deviation	8.3	9.5
Maximum	348	311
Minimum	316	279

Figure 94 Inc. Terex CMI Plant in Jefferson Co., FL

Volumetric Mix Properties

Samples of both mixtures were obtained during production to compare moisture contents, percent coating, and volumetric properties between the HMA and WMA. Samples were taken from trucks leaving the plant.

The average moisture contents were 0.04 and 0.05 percent for the HMA and WMA, respectively. These results are both very low and virtually the same, which demonstrates that incomplete drying of the aggregate was not a concern for this WMA. The percent of coated particles was 98.0 and 99.0 percent for the HMA and WMA mixes respectively. Thus, the WMA and HMA exhibited similar coating characteristics and incomplete coating was not a concern for either mix.

Specimens were compacted using 75 gyrations in the SGC at compaction temperatures of 295°F for the HMA samples and 250°F for the WMA samples. These laboratory compaction temperatures were determined from the average compaction temperature observed on the test sections through the first couple of hours of construction for each mixture. These volumetric samples compacted on-site in the NCAT mobile laboratory so that the mixes would not have to be reheated. Average test results are summarized in Table 126.

Gradation and asphalt content results for the HMA were nearly identical to the JMF values. However, the air voids on the design verification samples were much lower than the target 4.0 percent. The bulk specific gravity of both of these samples was rechecked in order to verify the results. The average air void content for the WMA was much closer to the design target probably due to its slightly lower asphalt content and slightly lower dust content.

Table 126 Gradation, Asphalt Content, and Volumetrics for Plant-Produced Mix from Jefferson Co., FL

Property	JMF	HMA	Terex Foam	Control Points
Sieve Size	% Passing			
19.0 mm (3/4")	100.0	100.0	100.0	100
12.5 mm (1/2")	100.0	99.7	99.4	90 – 100
9.5 mm (3/8")	89.0	91.1	90.8	--
4.75 mm (#4)	63.0	63.8	63.0	--
2.36 mm (#8)	46.0	44.9	43.5	28 – 58
1.18 mm (#16)	35.0	33.8	32.5	--
0.60 mm (#30)	27.0	25.8	24.6	--
0.30 mm (#50)	15.0	15.3	13.9	--
0.15 mm (#100)	8.0	9.2	7.9	--
0.075 mm (#200)	5.4	5.5	4.8	2 - 10
Asphalt Content (%)	5.30	5.33	4.95	--
G_{mm}	2.545	2.542	2.556	--
G_{mb}	2.444	2.493	2.470	--
Air Voids (%)	4.0	1.9	3.4	--
P_{ba} (%)	0.79	0.76	0.74	--
P_{be} (%)	4.55	4.61	4.24	--

Construction

The segment of US-98 paved while the research team was on-site was about a 50-60 minute drive from the plant in Tallahassee. The WMA was placed in the eastbound lane while the HMA was placed in the westbound lane. Figure 95 illustrates the location of the test sections. Both the HMA and WMA test sections were paved as the surface (wearing) course and had a target thickness of 2.5 inches. The underlying layer was a new intermediate asphalt pavement course.

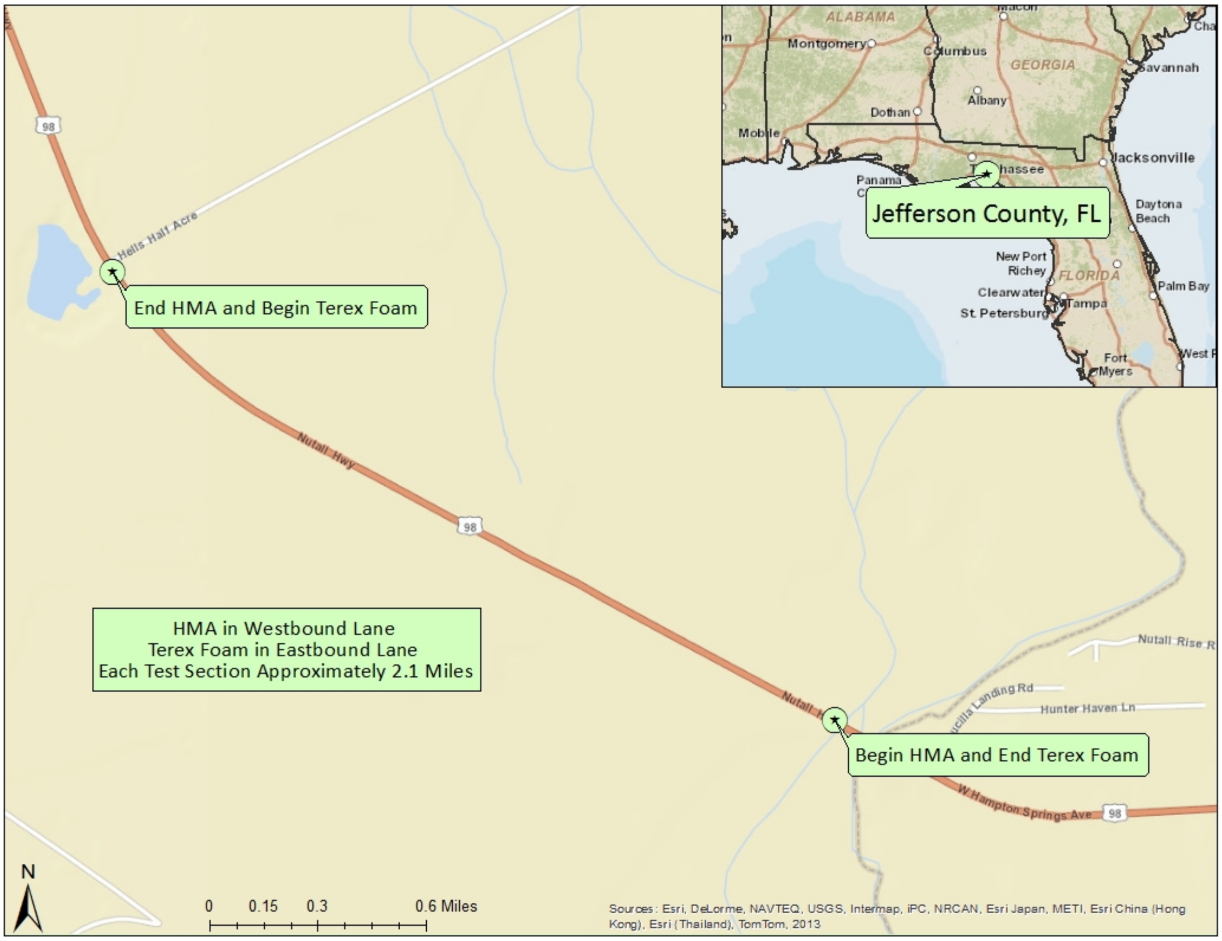


Figure 95 Location of Test Sections in Jefferson Co., Florida

The mixtures were delivered using tarped dump trucks. A cycle of 26-28 trucks was used to deliver the material to the roadway. The haul distance from the plant to the roadway was approximately 36 miles, which took the trucks about 50-60 minutes trucks to arrive. A RoadTec MTV-1000C MTV was used to transfer the mixtures from the delivery trucks to the paver. A Caterpillar AP-1055D was the paver used for both mixes. Figure 96 shows the MTV transfer mix from the dump truck into the paver.

Figure 96 MTV Transferring Mix into the Paver in Jefferson County, Florida

The temperature of the mix behind the paver was measured using a hand-held temperature gun and the PAVE-IR system. Table 127 shows the temperatures from behind the screed using both measuring techniques. Since the PAVE-IR system takes continuous readings throughout the paving operation some differences are expected as compared to the periodic temperature gun readings. It is likely that the hand-held temperature readings were not taken in some areas where the mix was cooler.

Table 127 Temperatures behind Screed in Tallahassee, FL

Temperature (°F)	Measuring Device	HMA	Terex Foam
Average	Temperature Gun	296.3	273.3
	PAVE-IR	268.4	247.0
Standard Deviation	Temperature Gun	9.0	10.0
	PAVE-IR	14.4	13.6
Maximum	Temperature Gun	312.3	287.7
	PAVE-IR	304.0	278.0
Minimum	Temperature Gun	273.3	249.3
	PAVE-IR	229.0	170.0

Weather data were collected hourly at the paving location using a handheld weather station. There was no rain during the construction of either mix. Table 128 shows the ambient temperatures, wind speed, and humidity for both mixes produced.

Table 128 Weather Conditions during Construction in Jefferson County, FL

Measurement	Statistic	HMA	Terex Foam
Ambient Temperature (°F)	Average	73.5	77.4
	Range	56.9 – 85.1	50.8 – 93.7
Wind Speed (mph)	Average	1.3	1.2
	Range	0 – 3.6	0.8 – 1.7
Humidity (%)	Average	52.2	48.7
	Range	34.6 – 78.5	23.0 – 92.7

The WMA was compacted using three rollers. Two Ingersoll Rand DD-110 steel wheel rollers compacted in echelon as the breakdown rollers. The two breakdown rollers were operated in the static mode. The finishing roller used for the WMA was also an Ingersoll Rand DD-110 steel wheel roller operated in the static mode. There was no "fixed" rolling pattern with the WMA. There seemed to be a "tender zone" and achieving the desired density level was a struggle.

The HMA was compacted using four rollers. The same breakdown and finishing rollers were used, but a fourth Ingersoll Rand PT-240R rubber tire roller was also used as the intermediate roller for most of the day. It was removed later in the day after the fourth sublot. The rolling pattern for the breakdown rollers was seven passes each in the static mode. The intermediate roller used a pattern of two passes on each side of the mat, then back up either the middle or the joint. The finishing roller used four passes each side, then back up either the middle or the joint

Construction Core Testing

A summary of test results from construction cores are shown in Table 129. Average core densities were similar for both mixes, at 93.1 percent of theoretical maximum density for the HMA and 92.1 percent for the WMA. The tensile strengths for both mixes were very good and were virtually the same for both mixes.

Table 129 Construction Cores Test Results from Jefferson Co., FL

Property	Statistic	HMA	Terex Foam
In-place Density (% of G_{mm})	Average	93.0	92.1
	Standard Deviation	1.1	1.1
Tensile Strength (psi)	Average	151.2	153.0
	Standard Deviation	10.2	16.7

Field Performance at 14-Month and 24-Month Project Inspections

Field performance evaluations were conducted on December 7, 2011, after about 14 months, and on September 12, 2012 after 24-months of traffic. Data were collected on each section to document performance regarding rutting, cracking, and raveling. Cores were also extracted to determine the in-place density, indirect tensile strengths, theoretical maximum specific gravity, gradation, and asphalt content.

The average rut depths are presented in Table 130. The HMA and WMA sections had average rut depths of 1.9 mm and 2.4 mm respectively at the time of the first inspection. At the time of the second inspection, the HMA had an average rut depth of 2.9 mm, and the WMA measures an average of 3.0 mm. The differences in rutting between the HMA and WMA were not practically significant and the rutting performance is considered excellent considering the high percentage of heavy truck traffic on this roadway.

Table 130 Rut Depths for Jefferson Co., FL

Mix	14-Month Inspection		24-Month Inspection	
	Avg. (mm)	Std. Dev. (mm)	Avg. (mm)	Std. Dev. (mm)
HMA	1.9	0.3	2.9	0.3
WMA	2.4	0.7	3.0	0.8

Each 200 ft. (61 m) evaluation section was carefully inspected for visual signs of cracking. No cracking was visible at the time of either inspection.

The surface textures of both the HMA and WMA test sections were measured using the sand patch test in accordance with ASTM E965. It was raining at the time of the first inspection, so the sand patch test could not be performed correctly on the in-place sections. Instead, the sand patch test was performed on the cores from the wheelpaths in each section. For the second inspection, the sand patch test was performed both in the field and on the cores from the wheelpath. The calculated mean texture depths for each mix are shown in Table 131.

Table 131 Mean Texture Depths for Jefferson Co., FL

	14-Month Inspection		24-Month Inspection			
	Measured in Laboratory on Cores from WP		Measured in Laboratory on Cores from WP		Measured in the Field in the WP	
Mix	Mean Texture Depth (mm)	Standard Deviation (mm)	Mean Texture Depth (mm)	Standard Deviation (mm)	Mean Texture Depth (mm)	Standard Deviation (mm)
HMA	0.44	0.11	0.45	0.05	0.61	0.02
Terex Foam	0.40	0.14	0.47	0.03	0.73	0.14

These results show similar mean texture depths for the two mixes. The WMA section performed slightly better in terms of raveling as compared to the HMA section. It can be seen that there is an offset between results from the field and results in the laboratory. Overall, the results of the sand patch test show that both mixes performed well in terms of raveling and weathering. Figure 97 shows an example of the surface of the Astec DBG and HMA sections at the time of the 24-month inspection.

Figure 97 WMA (left lane) and HMA Control Sections (right lane) in Jefferson Co., FL

Core Testing

A summary of the 14-month and 24-month core testing compared to the as-constructed results is shown in Table 132. The gradations and asphalt contents of both mixes were similar. The 14-month cores had slightly lower, but similar densities to cores obtained after construction. The 22-month cores were also similar to the as-constructed and 14-month cores, indicating that no densification has occurred for either mix. This is most likely due to the stiff binder specified for the project. The average tensile strengths increased by 47.3 and 35.3 psi for the HMA and WMA respectively. This increase can be attributed to stiffening of the binder due to aging. Overall, the tensile strengths for both mixes at the 14 and 24 month inspections are acceptable and expected for a stiff binder grade.

Table 132 Test Results from Production Mix, 14-Month Cores, and 24-Month Cores

Property	HMA	Terex Foam	HMA	Terex Foam	HMA	Terex Foam
	Production Mix (October 2010)		14-Month Cores (December 2011)		24-Month Cores (September 2012)	
Sieve Size	% Passing		% Passing		% Passing	
19.0 mm (3/4”)	100.0	100.0	100.0	100.0	100.0	100.0
12.5 mm (1/2”)	99.7	99.4	99.0	99.4	99.6	99.2
9.5 mm (3/8”)	91.1	90.8	92.5	92.2	92.8	93.3
4.75 mm (#4)	63.8	63.0	63.9	63.6	63.2	66.0
2.36 mm (#8)	44.9	43.5	45.2	45.1	44.8	46.8
1.18 mm (#16)	33.8	32.5	33.6	33.3	33.0	34.2
0.60 mm (#30)	25.8	24.6	26.2	25.9	25.7	26.5
0.30 mm (#50)	15.3	13.9	15.4	14.6	14.9	14.9
0.15 mm (#100)	9.2	7.9	9.2	8.5	8.8	8.7
0.075 mm (#200)	5.5	4.8	5.8	5.4	5.3	5.4
Asphalt Content (%)	5.33	4.95	4.82	4.99	4.87	5.13
G_{mm}	2.542	2.556	2.563	2.561	2.561	2.551
G_{mb}	2.366*	2.356*	2.373	2.352	2.343	2.343
In-place Density (%)	93.0*	92.1*	92.6	91.8	91.5	91.8
P_{ba} (%)	0.76	0.74	0.77	0.84	0.77	0.77
Tensile Strength (psi)	151.2*	153.0*	198.5	188.2	184.5	177.4

*Data from construction cores, not mix sampled during production as identified in column header.

Table 133 shows the average densities and tensile strength results by location for both inspections. At the first inspection, the average density of the HMA in the wheelpath was slightly higher than the density between the wheelpaths, but the difference is within the range expected for normal sampling and testing variability. For the WMA, the density in the right wheelpath at 14 months was slightly lower than the as-constructed cores, and the difference increased at 24 months. At the time of both inspections, the tensile strength values for both mixes were lower in the wheelpath as compared to the cores between the wheelpaths. The lower densities and tensile strengths in the wheelpaths do not follow the expected trends and may indicate the beginning of a moisture damage problem.

Table 133 In-Place Density and Tensile Strengths by Location in Jefferson County, FL

Property	Location of Cores	HMA	Terex Foam	HMA	Terex Foam
		14-Month Inspection		24-Month Inspection	
In-Place Density (% of G_{mm})	Between Wheelpaths	92.3	92.0	92.3	92.8
	In Right Wheelpath	93.0	91.6	90.4	90.9
Tensile Strength (psi)	Between Wheelpaths	207.5	208.7	223.5	227.1
	In Right Wheelpath	189.6	167.8	145.4	127.6

Performance Prediction

The initial AADTT for US-98 in Jefferson County, FL was 800 trucks per day with one lane in each direction. A traffic growth factor of 0.5% was calculated from recent historical data. US-98 was classified as a minor arterial. The five closest weather stations to the project site were missing data; therefore the MEPDG would not create a climate file from these sites. Attempts to edit the files were unsuccessful. Palatka, FL has similar average temperatures and rainfall. Data from surrounding stations was used to simulate Jefferson County's climate. Table 135 summarizes the pavement structure.

Table 135 US-98 Jefferson County, FL Pavement Structure

Layer	Thickness, in. [cm]
WMA/HMA surface course	1.5 [3.8]
Existing S-I HMA – 12.5 mm NMAS with PG 64-22	5.0 [12.7]
Existing Sand-Asphalt Hot Mix – 4.75 mm NMAS with PG 64-22	4.0 [10.2]
AASHTO A-3 Subgrade	Semi-infinite

Figure 94 shows a comparison of the predicted rutting for the WMA and HMA sections. The figure shows the subtotal of the predicted rutting for all of the asphalt layers and the predicted rutting for the experimental surface layers. The predicted rut depths for the test layer after 20-years of service were identical; 0.09 inches for both the WMA and HMA. Higher rutting, approximately 0.43 inches, was indicated for the combined asphalt layers.

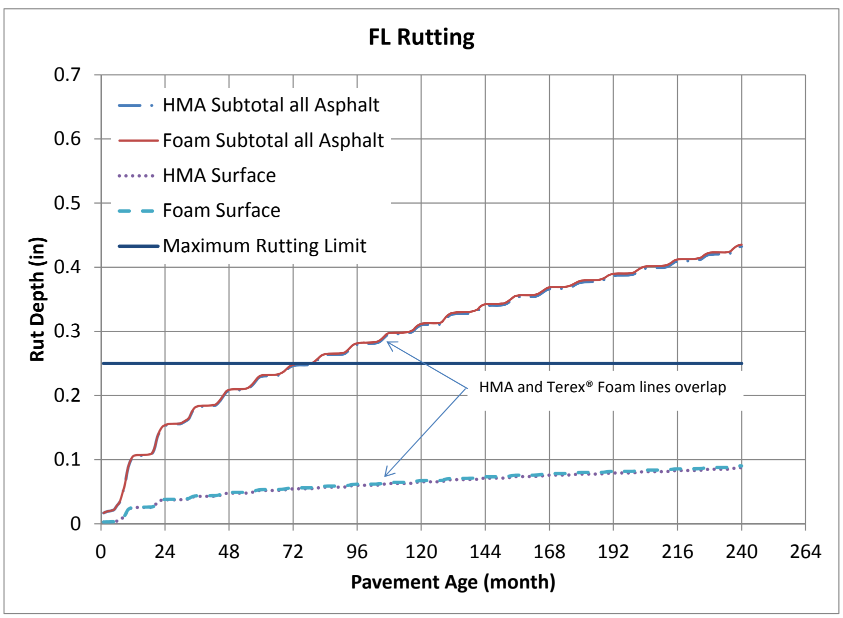


Figure 94 MEPDG Predicted Test Layer Asphalt Rutting for US-98 Jefferson County, FL

Figure 95 compares the predicted longitudinal cracking for US-98 over the design life. More longitudinal cracking is predicted for the WMA compared to the HMA (1,320 and 649 feet per mile, respectively). One possible explanation for the increased cracking predicted for the WMA is the difference in in-place air voids between the WMA and HMA. The Terex foam averaged 7.9 percent voids at the time of construction, whereas the HMA had an average of 7.0 percent voids.

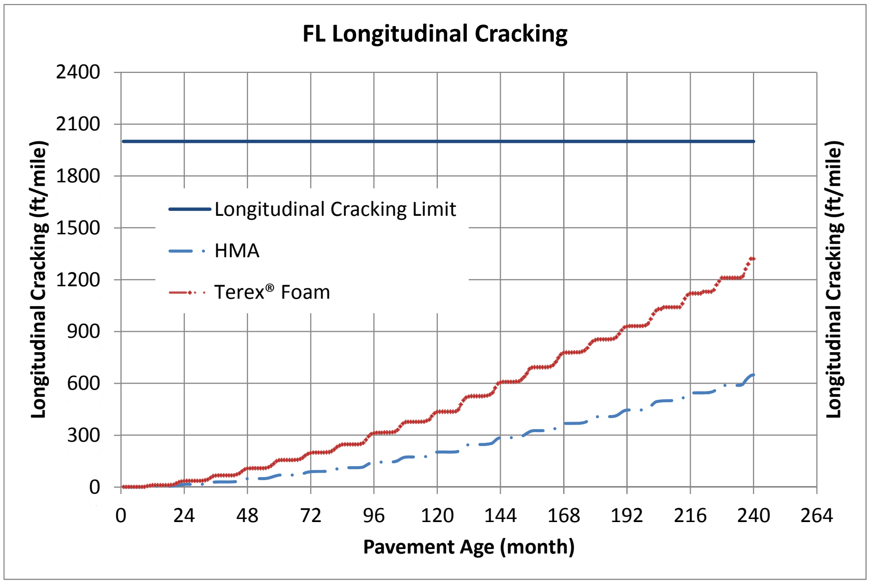


Figure 95 MEPDG Predicted Longitudinal Cracking for US-98 Jefferson County, FL

New York, New York

A WMA trial project was constructed on Little Neck Parkway in New York, NY in October 2010. Three WMA mixes and an HMA control mix were produced by a New York City (NYC) DOT owned plant and the project was constructed by a NYCDOT crew. The first WMA technology used on this project was the chemical additive Cecabase RT manufactured by the Arkema Group. The second WMA technology used was the additive BituTech PER produced by Engineered Additives, LLC. The third WMA technology was the additive SonneWarmix produced by SonneWarmix, Inc. The portion of Little Neck Parkway that contained the HMA and SonneWarmix had an approximate two-way AADT of 8,354 vehicles per day with 10.5 percent trucks. The portion of the roadway containing the Cecabase RT and BituTech PER had an approximate two-way AADT of 6,115 vehicles per day with 10.5 percent trucks. The production and construction of the Cecabase RT, HMA, SonneWarmix, and BituTech PER took place on October 19, 20, 21, and 22, 2010 respectively.

The asphalt mixture used for this trial consisted of a coarse-graded 12.5 mm NMAS Superpave mix design, with a compactive effort of 75 gyrations. The mix design used for the HMA was also used for the WMA technologies without any changes. The NYCDOT typically performs designs by the Marshall mix design method, but it was requested to provide a Superpave mix design for the purposes of this trial. The outside contractor hired to perform the design, constrained by the aggregates available and the DOT's material specifications, was only able to get as low as 91.1 percent passing the 9.5mm sieve instead of the required 89.9% to be a true 12.5mm NMAS mix. However, the gradation meets all other 12.5mm NMAS requirements.

All four mixtures contained 20 percent RAP. The RAP was a single-source milled material that was crushed off-site. The material percentages used for mix design and production are shown in Table 134. A PG 64-22 asphalt binder was used as the virgin binder for all mixes. The JMF, optimum asphalt contents, and specifications are shown in Table 135.

Table 134 Aggregate Percentages Used in Mix Design for New York, NY

Aggregate Type	%
⅜" by ¼" Coarse	55
Black Sand	25
Crushed RAP	20

Table 135 Design Gradation, Asphalt Content, and Volumetrics for Mix Design for New York, NY

Property	Design Values	JMF Targets	JMF Range	General Limits
Sieve Size	% Passing			
12.5 mm (1/2")	100.0	100.0	95-100	90-100
9.5 mm (3/8")	91.1	91.0	86-96	<90
4.75 mm (#4)	55.8	56.0	51-61	--
2.36 mm (#8)	34.5	34.0	31-39	31-58
1.18 mm (#16)	24.9	25.0	20-30	--
0.60 mm (#30)	18.5	19.0	14-24	--
0.30 mm (#50)	13.0	13.0	8-18	--
0.15 mm (#100)	8.9	9.0	4-14	--
0.075 mm (#200)	6.4	6.0	2-10	2-10
AC (%)	5.3	5.3	5.1-5.5	--
Air Voids (%)	3.51	--	--	--
VMA (%)	15.1	--	--	--
VFA (%)	76.7	--	--	--
D/A Ratio	1.37	--	--	--
P_{be} (%)	4.66	--	--	--
P_{ba} (%)	0.68	--	--	--

Production

All three WMA additives were terminal blended with the PG 64-22 binder and brought in for each day's production. The first WMA technology used on this project was the chemical additive Cecabase RT, a non-aqueous surfactant added to the binder at a rate of 0.4 percent by weight of total binder. HMA was produced on the second day. On the third day, the additive SonneWarmix was used at a rate of 0.7 percent by weight of total binder. On the fourth day of the project, the additive BituTech PER was used at a rate of 0.76 percent by weight of RAP. Table 136 shows the production temperatures for each mix.

Table 136 Production Temperatures in NYC, NY

Temperatures (°F)	HMA	BituTech PER	Cecabase RT	SonneWarmix
Average	344.2	279.0	246.9	262.3
St. Deviation	17.0	26.9	17.3	27.8
Maximum	368	360	271	330
Minimum	318	260	200	238

Volumetric Mix Properties

Samples of each mixture were obtained during production to determine moisture contents, percent coating, and volumetric properties for comparisons between the HMA and WMA mixes. Samples were taken from a mini-stockpile made each day specifically for sampling.

The average moisture contents were 0.13, 0.33, 0.37, and 0.43 percent for the HMA, BituTech PER, Cecabase RT, and SonneWarmix, respectively. The WMA moisture contents may have been higher than the HMA due to incomplete drying of the aggregate, RAP, or both. However, the moisture contents for the WMA mixes were all below the commonly specified limit of 0.5 percent.

The percentage of completely coated particles was then determined by a Ross count. The percent of coated particles was 100.0, 99.5, 100.0 and 99.5 percent for the HMA, BituTech PER, Cecabase RT, and SonneWarmix respectively, which indicates excellent coating for all of the mixes.

Specimens were compacted using 75 gyrations in the SGC at compaction temperatures of 300°F for the HMA and 225°F for all three WMA mixes. These laboratory compaction temperatures were determined from the average compaction temperature observed on the test sections through the first couple of hours of construction for each mixture. These volumetric samples were compacted on-site in the NCAT mobile laboratory so that the mixes would not have to be reheated. Average test results are summarized in Table 137.

The asphalt content of the HMA (5.38 percent) was very close to the target of 5.3 percent. However, the dust content was 1.0 percent low and the air void content was 1.9 percent above the design. The BituTech PER asphalt content was 0.18 percent above the JMF target and the dust content was closer to the JMF, but the air void content was 2.1 percent above the target of 3.5 percent. The Cecabase had the highest asphalt content and the highest dust content, which contributed to the air void content being 0.51 percent lower than the design. Finally, the SonneWarmix asphalt content hit the target asphalt content and was only 0.1 percent high on the dust content, but the air void content was 1.4 percent higher than the design. Except for the Cecabase-RT mix, the individual WMA mixes and the control HMA compare reasonably well.

Table 137 Gradation, Asphalt Content, and Volumetrics for Plant-Produced Mix in New York, NY

Property	JMF	HMA	BituTech PER	Ceca-base RT	Sonne-Warmix	JMF Range
Sieve Size	% Passing					
19.0 mm (3/4”)	100.0	100.0	100.0	100.0	100.0	100
12.5 mm (1/2”)	100.0	99.7	99.7	99.9	99.9	95-100
9.5 mm (3/8”)	91.0	92.1	94.5	94.9	94.7	86-96
4.75 mm (#4)	56.0	55.1	59.3	60.9	61.8	51-61
2.36 mm (#8)	34.0	33.8	34.7	36.2	36.5	31-39
1.18 mm (#16)	25.0	24.1	24.0	25.7	25.3	20-30
0.60 mm (#30)	19.0	17.4	17.2	18.9	18.2	14-24
0.30 mm (#50)	13.0	11.9	11.9	13.4	12.8	8-18
0.15 mm (#100)	9.0	7.7	8.0	9.2	8.8	4-14
0.075 mm (#200)	6.0	5.0	5.4	6.3	6.1	2-10
Asphalt Content (%)	5.30	5.38	5.48	5.66	5.30	--
G_{mm}	2.645	2.646	2.643	2.621	2.641	-
G_{mb}	2.552	2.505	2.496	2.544	2.512	-
Air Voids (%)	3.5	5.4	5.6	3.0	4.9	-
P_{ba} (%)	0.68	0.75	0.77	0.55	0.61	-
P_{be} (%)	4.66	4.67	4.75	5.15	4.72	-

Construction

The location of the field sections on Little Neck Parkway was approximately 12 miles from the plant. The travel time to the site ranged from 20 to 50 minutes depending on the time of day and traffic. The Cecabase RT was placed in both southbound lanes from the intersection of Union Turnpike to 21 feet south of the intersection of 82nd Avenue. The HMA was placed in the southbound lanes from the intersection of Hillside Avenue to in between the intersection of 87th Avenue and 87th Road. The SonneWarmix was placed in the two northbound lanes between 87th Drive and just before E. Williston Avenue. The BituTech PER was placed in the northbound lanes from Hillside Avenue to 82nd Avenue. All four mixes were paved as the surface (wearing) course and had a target thickness of 2.5 inches. The surface mixes were placed on a milled asphalt pavement surface that had some slight transverse cracking spread throughout the sections. Approximately 3.5 inches beneath the milled asphalt layers was a plain jointed concrete pavement. Figure 98 illustrates the location of the test sections.

Figure 98 Location of Test Sections in NYC, New York

The temperature of the mix behind the paver was measured using a hand-held temperature gun and the PAVE-IR system. **Table 138** shows the temperatures from behind the screed using both measuring techniques.

Table 138 Temperatures behind the Screed in New York, NY

Temperature (°F)	Measuring Device	HMA	BituTech PER	Cecabase RT	Sonne-Warmix
Average	Temperature Gun	299.2	234.2	220.9	228.5
	PAVE-IR	N/A	237.7	N/A	222.0
Standard Deviation	Temperature Gun	7.5	4.8	12.9	16.7
	PAVE-IR	N/A	14.6	N/A	7.1
Maximum	Temperature Gun	309.3	241.3	239.3	252.0
	PAVE-IR	N/A	316.0	N/A	252.0
Minimum	Temperature Gun	284.0	225.7	198.3	203.0
	PAVE-IR	N/A	195.0	N/A	178.0

Weather data was collected hourly at the paving location using a handheld weather station. Ambient temperature, wind speed, and humidity were recorded and are shown in Table 139. The only day that had rain was the first day during production of the Cecabase RT, during which trace amounts of rain fell in the area.

Three rollers were used to compact all four mixes. The breakdown roller was a Sakai SW-850 which operated in the vibratory mode. The intermediate roller was an Ingersoll Rand DD-110 which also operated in the vibratory mode. The finishing roller was a steel wheel Hyster C-350D which operated in the static mode. There was not a consistent rolling pattern for any of the mixes.

Table 139 Weather Conditions during Construction in New York, NY

Measurement	Statistic	HMA	BituTech PER	Cecabase RT	Sonne-Warmix
Ambient Temperature (°F)	Average	62.1	52.9	60.8	58.5
	Range	57.4 – 65.4	49.7 – 53.9	58.1 – 65.4	56.7 – 61.4
Wind Speed (mph)	Average	1.3	6.5	0.9	3.0
	Range	0 – 2.9	3.3 – 9.8	0.7 – 1.0	1.8 – 4.9
Humidity (%)	Average	51.3	46.1	66.9	72.9
	Range	39.6 – 65.8	43.1 – 54.2	59.4 – 71.3	59.5 – 76.8

Construction Core Testing

After construction of each mix, cores were obtained from all four sections. Core densities were determined in accordance with AASHTO T 166 and tensile strength according to ASTM D6931. Results are shown in Table 140. The densities for the Bitutech PER and Cecabase-RT mixes were similar; the densities for the HMA and SonneWarmix were lower. The tensile strengths for the Cecabase RT and SonneWarmix were slightly lower than the HMA and Bitutech PER.

Table 140 Construction Cores Test Results from New York, NY

Property	Statistic	HMA	BituTech PER	Cecabase RT	Sonne-Warmix
In-Place Density (% of G_{mm})	Average	90.8	92.4	92.1	89.9
	Standard Deviation	2.0	1.3	2.1	4.0
Tensile Strength (psi)	Average	103.4	98.9	93.3	91.8
	Standard Deviation	13.6	10.5	16.6	17.2

Field Performance at 15-Month and 26-Month Project Inspections

Field-performance evaluations were conducted on January 19, 2012, after about 15 months, and on December 12, 2012 after 26 months of traffic. Data were collected on each section to document performance regarding rutting, cracking, and raveling. Cores were taken to determine in-place densities, indirect tensile strengths, theoretical maximum specific gravity, gradations, and asphalt contents.

Table 141 shows the rut depths at the time of each inspection. These results are based on the measurements from the more severe of the two wheelpaths measured at each random location. The data show that none of the sections had rutted significantly at the time of the inspections.

Table 141 Rutting Measurements in New York, NY

Mix	15-Month Inspection		26-Month Inspection	
	Average Rut Depth (mm)	Standard Deviation (mm)	Average Rut Depth (mm)	Standard Deviation (mm)
HMA	1.0	0.9	1.9	1.2
BituTech PER	0.7	1.2	2.7	1.2
Cecabase RT	0.3	0.6	0.3	0.6
SonneWarmix	0.0	0.0	0.0	0.0

Each 200 ft. (61 m) evaluation section was carefully inspected for visual signs of cracking. At the time of the first inspection, only the Cecabase RT had any cracking. The Cecabase sections had a low severity, approximately nine-foot long transverse crack and two other one-foot cracks that appeared to due to underlying utility trenches. At the time of the second inspection, low severity cracks had appeared in all four mix sections, although all of the sections were still performing very well. Table 142 shows a summary of the cracking observed at the time of the second inspection.

Table 142 Observed Cracking in New York, NY at 26-Month Inspection

Mix Section	Severity	Wheelpath Longitudinal		Non-Wheelpath Longitudinal		Transverse	
		# of Cracks	Total Length, m	# of Cracks	Total Length, m	# of Cracks	Total Length, m
HMA Total	Low	1	0.3	1	3.0	5	5.5
	Mod.	0	0	0	0	0	0
	High	0	0	0	0	0	0
BituTech	Low	1	5.2	0	0	0	0
	Mod.	0	0	0	0	0	0
	High	0	0	0	0	0	0
Cecabase	Low	1	15.2	0	0	3	4.9
	Mod.	0	0	0	0	0	0
	High	0	0	0	0	0	0
SonneWarmix	Low	1	5.2	0	0	0	0
	Mod.	0	0	0	0	0	0
	High	0	0	0	0	0	0

During both inspections, the surface texture was measured using the sand patch test at the beginning of each evaluation section in the outside wheelpath. The calculated mean texture depths for each section are shown in Table 143 . It can be seen that the HMA had slightly higher mean texture depths than the WMA sections, indicating slightly more raveling compared to the three WMA mixes. However, the differences are probably not practically significant. Also, the surface texture results are similar for the 15-month and 26-month inspections which indicate that weathering of the pavements has stabilized. Figure 99 through Figure 102 show examples of the HMA, BituTech PER, Cecabase, and SonneWarmix sections, respectively.

Table 143 Mean Texture Depths for New York, NY

Mix	15-Month Inspection		26-Month Inspection	
	Mean Texture Depth (mm)	Standard Deviation (mm)	Mean Texture Depth (mm)	Standard Deviation (mm)
HMA	0.87	0.10	0.79	0.13
BituTech PER	0.67	0.09	0.70	0.05
Cecabase	0.64	0.22	0.60	0.08
SonneWarmix	0.65	0.02	0.56	0.06

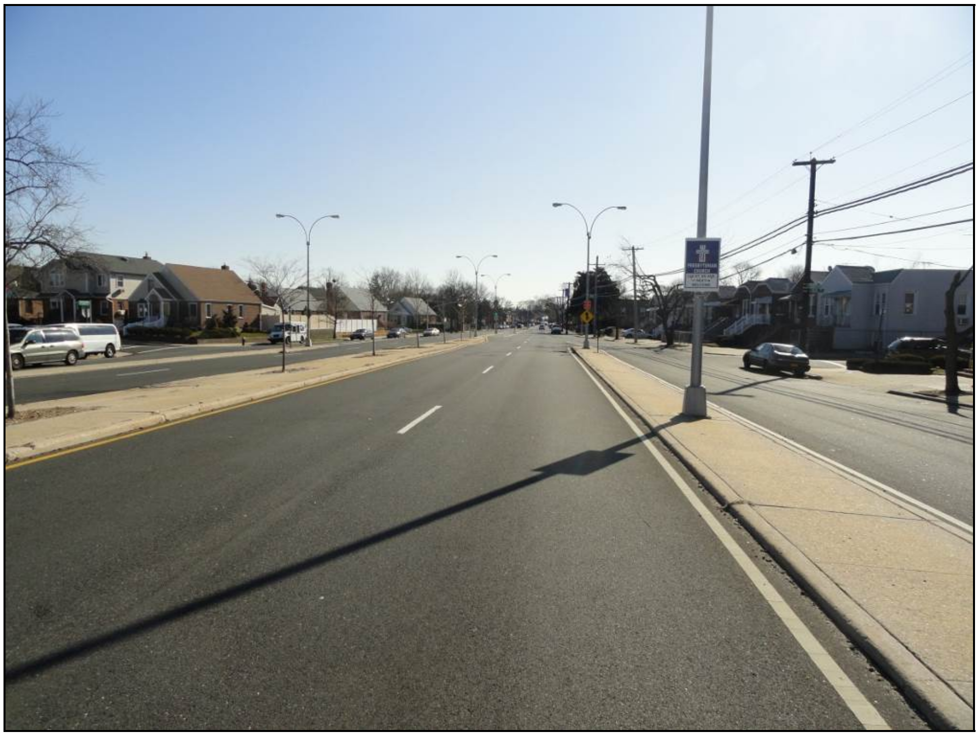


Figure 99 HMA Section in New York, NY

Figure 100 BituTech PER Section in New York, NY

Figure 101 Cecabase Section in New York, NY

Figure 102 SonneWarmix Section in New York, NY

Core Testing

At the time of each project inspection, seven 6-inch (150-mm) cores were taken from each mix section. A summary of the results from the 15-month inspection compared with the construction data is shown in Table 144.

The 15-month cores had higher densities compared to the construction cores due to traffic densification. The HMA density increased by 3.1 percent, while the BituTech PER, Cecabase RT, and SonneWarmix sections increased by 2.0, 1.3, and 2.4 percent, respectively. The tensile strengths were significantly lower compared to the cores taken right after construction. This can probably be attributed to the fact that four-inch cores were taken at construction, while six-inch cores were taken at the 15-month inspection. As explained in a previous section, four-inch cores typically yield higher tensile strengths compared to six-inch cores.

Table 144 Test Results on Production Mix and 15-Month Cores from New York, NY

Property	HMA	Bitu-Tech	Ceca-base	Sonne-War-mix	HMA	Bitu-Tech	Ceca-base	Sonne-War-mix
	Production Mix (October 2010)				15-Month Cores (January 2012)			
Sieve Size	% Passing				% Passing			
19.0 mm (3/4”)	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5 mm (1/2”)	99.7	99.7	99.9	99.9	100.0	99.6	99.7	99.9
9.5 mm (3/8”)	92.1	94.5	94.9	94.7	93.9	93.2	94.2	93.4
4.75 mm (#4)	55.1	59.3	60.9	61.8	63.2	59.6	60.9	59.1
2.36 mm (#8)	33.8	34.7	36.2	36.5	40.9	38.2	36.7	36.1
1.18 mm (#16)	24.1	24.0	25.7	25.3	27.6	26.1	24.8	25.2
0.60 mm (#30)	17.4	17.2	18.9	18.2	19.9	19.0	18.3	18.3
0.30 mm (#50)	11.9	11.9	13.4	12.8	13.3	13.1	12.5	12.4
0.15 mm (#100)	7.7	8.0	9.2	8.8	8.2	8.8	7.8	8.0
0.075 mm (#200)	5.0	5.4	6.3	6.1	5.1	6.1	4.8	5.2
Asphalt Content (%)	5.38	5.48	5.66	5.30	5.41	5.09	5.40	5.21
Avg. Prod. Temp. (°F)	344.2	279.0	246.9	262.3	344.2	279.0	246.9	262.3
G_{mm}	2.646	2.643	2.621	2.641	2.642	2.643	2.640	2.651
G_{mb}	2.404*	2.442*	2.415*	2.374*	2.482	2.494	2.466	2.447
In-place Density (%)	90.8*	92.4*	92.1*	89.9*	93.9	94.4	93.4	92.3
P_{ba} (%)	0.75	0.77	0.55	0.61	0.70	0.50	0.67	0.71
Tensile Strength (psi)	103.4*	98.9*	93.3*	91.8*	74.2	55.3	63.7	71.2

*Data from construction cores, not mix sampled during production as identified in column header.

The results from the 15-month and 26-month inspections are shown in Table 145. The cores from the second inspection exhibited slightly higher densities compared to the first inspection indicating further traffic densification between the first and second year. The densities were very similar for all four mixes. The average tensile strengths increased for all four mixes in the months between inspections due to binder stiffening and higher densities. The tensile strength of the HMA was significantly higher than the WMA sections.

Table 145 Test Results on 15-Month and 26-Month Cores from New York, NY

Property	HMA	Bitu-Tech	Ceca-base	Sonne-War-mix	HMA	Bitu-Tech	Ceca-base	Sonne-War-mix
	15-Month Cores (January 2012)				26-Month (December 2012)			
Sieve Size	% Passing				% Passing			
19.0 mm (3/4”)	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5 mm (1/2”)	100.0	99.6	99.7	99.9	99.7	99.7	100.0	99.8
9.5 mm (3/8”)	93.9	93.2	94.2	93.4	93.4	93.3	94.8	94.1
4.75 mm (#4)	63.2	59.6	60.9	59.1	61.2	58.9	63.6	61.7
2.36 mm (#8)	40.9	38.2	36.7	36.1	40.1	37.4	39.8	39.4
1.18 mm (#16)	27.6	26.1	24.8	25.2	27.7	25.9	27.5	27.2
0.60 mm (#30)	19.9	19.0	18.3	18.3	20.0	18.7	20.2	19.8
0.30 mm (#50)	13.3	13.1	12.5	12.4	13.3	12.6	13.9	13.4
0.15 mm (#100)	8.2	8.8	7.8	8.0	8.3	8.3	9.0	8.8
0.075 mm (#200)	5.1	6.1	4.8	5.2	5.0	5.4	5.8	5.8
Asphalt Content (%)	5.41	5.09	5.40	5.21	5.51	5.45	5.55	5.35
Avg. Prod. Temp. (°F)	344.2	279.0	246.9	262.3	344.2	279.0	246.9	262.3
G_{mm}	2.642	2.643	2.640	2.651	2.638	2.643	2.634	2.642
G_{mb}	2.482	2.494	2.466	2.447	2.502	2.524	2.491	2.502
In-place Density (%)	93.9	94.4	93.4	92.3	94.8	95.5	94.6	94.7
P_{ba} (%)	0.70	0.50	0.67	0.71	0.71	0.75	0.68	0.66
Tensile Strength (psi)	74.2	55.3	63.7	71.2	133.3	99.7	104.9	108.2

Table 146 shows the average density and tensile strength results by location for the cores at the time of both inspections. As expected, all four mixes had higher densities in the wheelpath compared to between the wheelpaths. The SonneWarmix section shows a large difference of 6.0 percent between the two locations at the time of the first inspection. However, the results seem to be more reasonable at the time of the second inspection. For most of the mix sections, the tensile strengths for the cores in the wheelpath are higher than the between wheelpath cores. This difference is likely due to the higher density of the wheelpath cores. The exception is the Cecabase RT mix, which had lower tensile strengths from wheelpath cores at both inspections.

Table 146 In-place Density and Tensile Strengths by Location from New York, NY

Location and Property	HMA	Bitu-Tech	Ceca-base	Sonne-Warmix	HMA	Bitu-Tech	Ceca-base	Sonne-Warmix
	15-Month Cores				26-Month Cores			
Between Wheelpaths Density (%)	93.4	93.8	93.1	89.8	94.2	94.8	94.2	93.4
In Right wheelpath Density (%)	94.7	95.1	93.8	95.7	95.7	96.5	95.0	96.5
Between Wheelpaths Tensile Strength (psi)	67.1	53.2	71.3	62.3	116.7	88.9	108.0	98.3
In Right wheelpath Tensile Strength (psi)	81.4	57.4	56.1	80.0	149.8	110.5	101.8	118.1

Performance Prediction

The test sections on Little Neck Parkway were divided by Hillside Avenue. Cecabase and BituTech PER were placed north of Hillside Avenue; HMA and SonneWarmix south of Hillside Avenue. The Cecabase and HMA were in the southbound lanes and the SonneWarmix and BituTech PER were in the northbound lanes. The initial AADTT north of Hillside Avenue was 643 trucks per day; south of Hillside Avenue it was 877 trucks per day. Little Neck Parkway is classified as a minor arterial. Table 147 summarizes the pavement structure. Thickness variations were noted in the cores, although the paver laid the same target thickness. An average thickness, which matched the target thickness, was used in the analysis.

Table 147 Little Neck Parkway New York City, NY Pavement Structure

Layer	Thickness, in. [cm]
WMA/HMA surface course	2.3 [5.8]
Type 6F RA Surface – 12.5 mm NMAS PG 64-22	1.9 [4.8]
Type 3 RA Binder – 19.0 mm NMAS PG 64-22	1.6 [4.1]
Plain Jointed Concrete Pavement	6.0 [15.2]
AASHTO A-3 Subgrade	Semi-infinite

Figure 103 shows a comparison of the predicted rutting for the WMA and HMA sections. The MEPDG predicts 0.12, 0.13, 0.15, and 0.10 in. (3, 3.3, 3.8, 2.5 mm) of rutting in the asphalt layers for the BituTech PER, Cecabase, SonneWarmix, and HMA, respectively after 20-years of service. As noted previously, the BituTech PER and Cecabase receive slightly less traffic than the other two mixes.

Figure 104 compares the predicted longitudinal cracking for Little Neck Parkway over the design life. Minimal longitudinal cracking is predicted. The maximum predicted longitudinal cracking is 2.89 ft./mi. (54.7 m/km) for the SonneWarmix after 20-years of service. IDT tests for low temperature cracking were not performed on the New York mixes, so thermal cracking predictions are not reported.

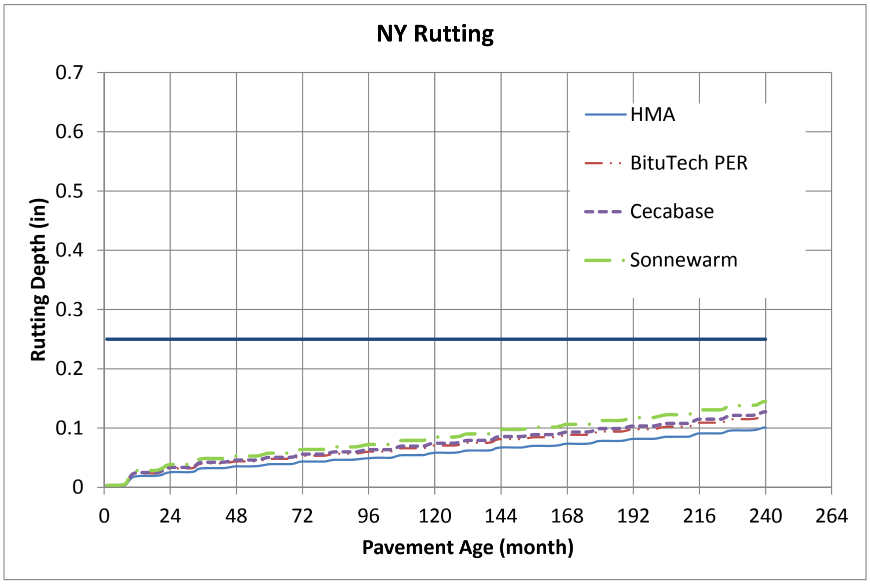


Figure 103 MEPDG Predicted Asphalt Rutting for Little Neck Parkway, New York, NY

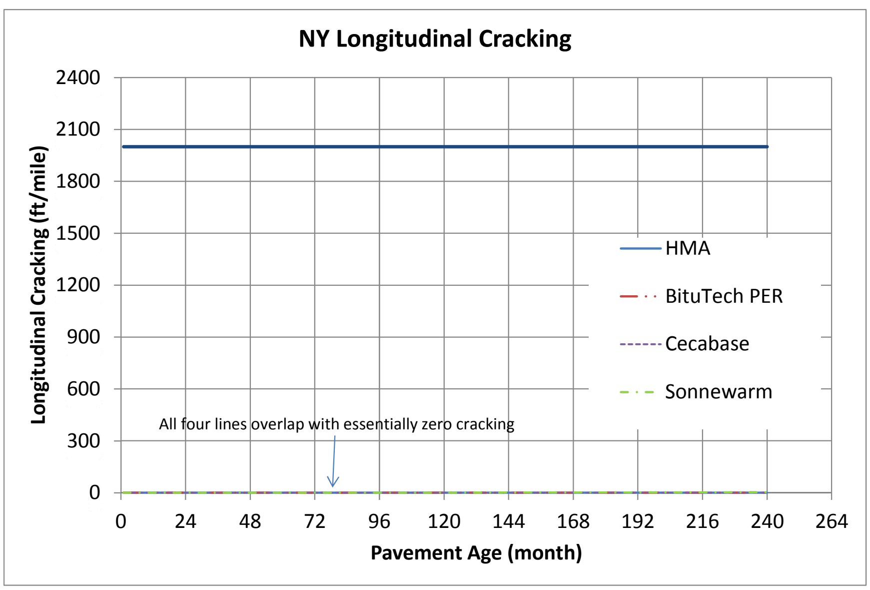


Figure 104 MEPDG Predicted Longitudinal Cracking for New York, NY

Casa Grande, Arizona

The final WMA project evaluated in this study was constructed on State Road (SR) 84 in Casa Grande, Arizona in December 2011. The contractor for this state-sponsored WMA trial was Southwest Asphalt, Tempe, AZ, a division of the Fisher Sand and Gravel Company. The WMA technology used on this project was Sasobit produced by the Sasol Wax North America Corporation. Two other WMA technologies (Evotherm 3G and Advera) were placed on this project before the NCAT team arrived. However, NCAT only documented the production and construction of the HMA and Sasobit sections due to project budget constraints. The WMA and HMA were produced and placed on SR-84 on the west side of Casa Grande, Arizona. The estimated two-way AADT for this two-lane roadway was approximately 3,800 vehicles per day with 12 percent trucks. The production of the Sasobit WMA and companion HMA control took place on December 6 and 7, 2011 respectively.

The asphalt mixture used for this trial consisted of a fine-graded 19.0 mm NMAS Marshall mix design, with a compactive effort of 75 blows. The mix design used for the HMA was also used for the WMA without any changes. Both mixtures contained crushed gravel, 11.9 percent RAP, and 1 percent portland cement as an anti-strip additive. The RAP consisted of millings from the project that was screened over a 1-1/2 inch sieve before entering the plant. The material percentages used for mix design and production are shown Table 148. A modified PG 70-10 asphalt binder supplied by Valero was used as the virgin binder for both mixes. The laboratory and production JMFs, optimum asphalt contents, specifications, and allowable tolerances are shown in **Table 149**.

Table 148 Aggregate Percentages Used in Mix Design in Casa Grande, AZ

Aggregate Type	Mix Design (%)
¾” Gravel	29.7
⅜” Gravel	15.8
Man Sand	9.9
Crushed Fines	31.7
RAP (Millings)	11.9
Type II Cement	1.0

Table 149 Design Gradation, Asphalt Content, and Volumetrics for Mix Design in Casa Grande, AZ

Property / Sieve Size	Design JMF	Production JMF	Mix Design Specification	Production Limits
25.0 mm (1”)	100	100	100	--
19.0 mm (3/4”)	97	97	90 - 100	--
12.5 mm (1/2")	92	92	--	--
9.5 mm (3/8")	75	75	62 - 77	69 - 81
6.35 mm (1/4”)	63	63	--	--
4.75 mm (#4)	55	55	--	--
2.36 mm (#8)	39	39	38 - 47	33 - 45
2.00 mm (#10)	34	34	--	--
1.18 mm (#16)	25	25	--	--
0.60 mm (#30)	15	15	--	--
0.425 mm (#40)	11	13	11 - 19	8 – 18*
0.30 mm (#50)	8	8	--	--
0.15 mm (#100)	5	5	--	--
0.075 mm (#200)	4.0	4.0	2.5 - 6.0	2.0 - 6.0
AC (%)	4.8	4.6	--	--
Air Voids (%)	5.7	5.7	--	--
VMA (%)	15.4	15.4	--	--
VFA (%)	63.2	63.2	--	--
D/A Ratio	0.94	0.94	--	--
P_{be} (%)	4.26	4.26	--	--
P_{ba} (%)	0.56	0.56	--	--

*Originally 6 - 16

Production

The WMA was produced using Sasobit blended on-site with the virgin binder in a tank typically used for blending ground tire rubber (GTR) at this particular plant. The tanks used for blending and storing the Sasobit binder are shown in Figure 105. For this field trial, the Sasobit was blended at a rate of 1.75 percent by weight of virgin binder to compensate for the RAP binder in order to reach a target rate of 1.5 percent by weight of total binder.

Figure 105 Tanks Used to Blend (left) and Store (right) Sasobit in Casa Grande, AZ

Production temperature for the HMA was approximately 319°F (159.4°C), and for the Sasobit mix, the production temperature was approximately 276°F (125.6°C). Table 150 shows the maximum, minimum, average, and standard deviation production temperatures for both the HMA and the Sasobit mixes.

Table 150 Production Temperatures in Casa Grande, AZ

Temperatures (°F)	HMA	Sasobit
Average	319.1	275.9
Standard Dev.	22.4	26.5
Maximum	356.0	336.0
Minimum	285.0	222.0

Volumetric Mix Properties

Samples of both mixtures were obtained during production to compare moisture contents, percent coating, volumetric properties between the HMA and WMA. Samples were taken from trucks leaving the plant.

The average moisture contents were 0.04 and 0.05 percent for the HMA and WMA, respectively. These results are low but reasonable considering the environment. Problems with incomplete drying of aggregates or RAP are not common in Arizona.

The percentages of completely coated particles were 96.2 and 96.3 percent for the HMA and Sasobit WMA mixtures respectively. This shows that the WMA and HMA exhibited similar coating characteristics.

Since the mix designs for this project were done by the Marshall mix design method, an equivalent gyration level was determined on-site in order make appropriately compacted SGC samples. This was accomplished by compacting samples at 50, 60, and 75 gyrations. The air voids determined from these samples were then plotted against gyration number to determine the gyration level equal to the target design air voids (5.2 percent). An air void target of 5.2 percent was used instead of the 5.7 percent from design because there was a consistent difference of about 1 percent air voids between the state QA and contractor's QC test results. The state was consistently getting around 4.7 percent air voids while the contractor was getting 5.7 percent. So 5.2 percent was used in order to split the difference. The equivalent SGC compactive effort was determined to be 67 gyrations. Figure 106 shows the plot used to determine this gyration level.

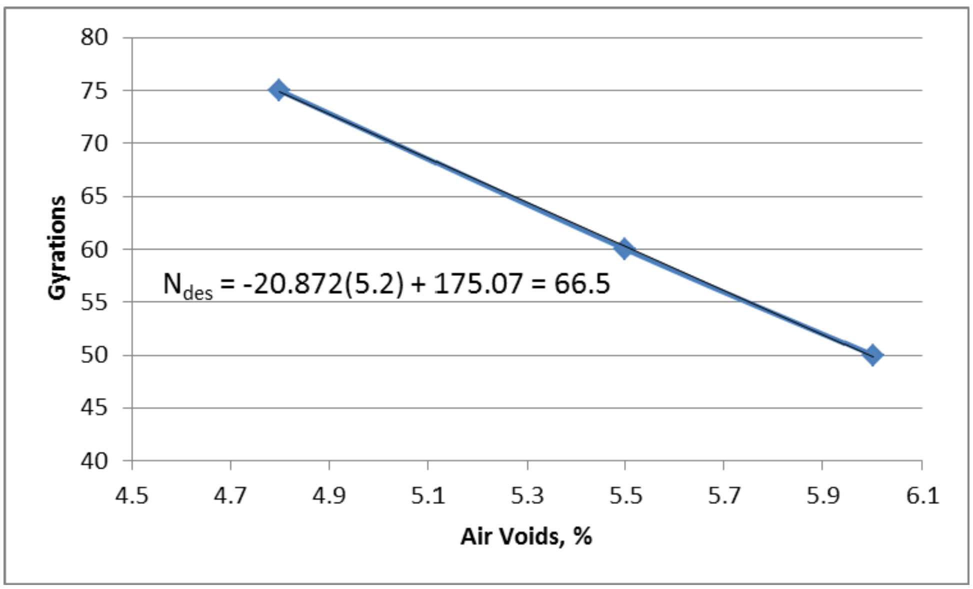


Figure 106 Determination of Equivalent Design Gyration Level for Casa Grande, AZ

Specimens were compacted using 67 gyrations in the SGC at compaction temperatures of 305°F for the HMA samples and 250°F for the WMA samples. These laboratory compaction temperatures were determined using the average compaction temperature observed on the test sections through the first couple of hours of construction for each mixture. These volumetric samples were compacted on-site in the NCAT mobile laboratory without reheating the mixes. Bulk specific gravities (G_{mb}) of the compacted specimens were determined in accordance with AASHTO T 166. The mixes were also brought back to the main NCAT laboratory where solvent extractions were conducted in accordance with AASHTO T 164. The gradation of the extracted aggregate was determined according to AASHTO T 30. Average test results are summarized in Table 151.

The asphalt contents for the HMA and WMA were very close to the JMF. The gradation for both mixes were somewhat finer than the production JMF, but were still within AZDOT's production limits. The percentages of absorbed asphalt were essentially equivalent for the two mixtures. The HMA had slightly lower air void contents than the WMA, which was not expected. Generally, due to increased compactability with WMA mixtures, WMA air voids are slightly lower than HMA when using the same design. However, some of the difference can probably be attributed to normal variability as well as the slightly lower asphalt content and percent passing the #200 sieve observed for the Sasobit mix.

Table 151 Gradation, Asphalt Content, and Volumetrics for Plant-Produced Mix in Casa Grande, AZ

Property	Production JMF	HMA	Sasobit WMA	Production Limits
Sieve Size	% Passing			
25.0 mm (1")	100.0	100.0	100.0	--
19.0 mm (3/4")	97.0	98.4	98.1	--
12.5 mm (1/2")	82.0	88.7	87.2	--
9.5 mm (3/8")	75.0	79.5	77.2	69-81
4.75 mm (#4)	55.0	57.3	55.3	--
2.36 mm (#8)	39.0	42.3	42.9	33-45
1.18 mm (#16)	25.0	29.5	29.2	--
0.60 mm (#30)	15.0	20.4	20.1	--
0.30 mm (#50)	8.0	12.4	12.0	--
0.15 mm (#100)	5.0	7.9	7.6	--
0.075 mm (#200)	4.0	5.6	5.4	2.0-6.0
Asphalt Content (%)	4.6	4.55	4.47	--
G_{mm}	2.467	2.482	2.484	--
G_{mb}	2.326	2.366	2.356	--
Air Voids (%)	5.2*	4.7	5.2	--
P_{ba} (%)	0.56	0.64	0.62	--

* 5.2% was the target air void content for the Superpave volumetric verification samples.

Construction

The HMA and WMA mixes were placed on the westbound and eastbound portions of SR-84 respectively. All paving was done heading eastbound. This portion of SR-84 was approximately 17 miles west of the plant location. Both mixes were placed over milled sections and incorporated a SS-1H tack coat applied at an application rate of 0.06 gal/yd^2. Figure 107 shows the placement of the test sections. Both the HMA and WMA test sections were paved as the surface (wearing) course and had a target thickness of 1.5 inches. Both surface mixes were placed on top of a milled section of asphalt pavement. Both mixes were topped with a chip seal approximately four months after construction. It is typical for all pavements in this area with similar traffic to be topped with a chip seal.

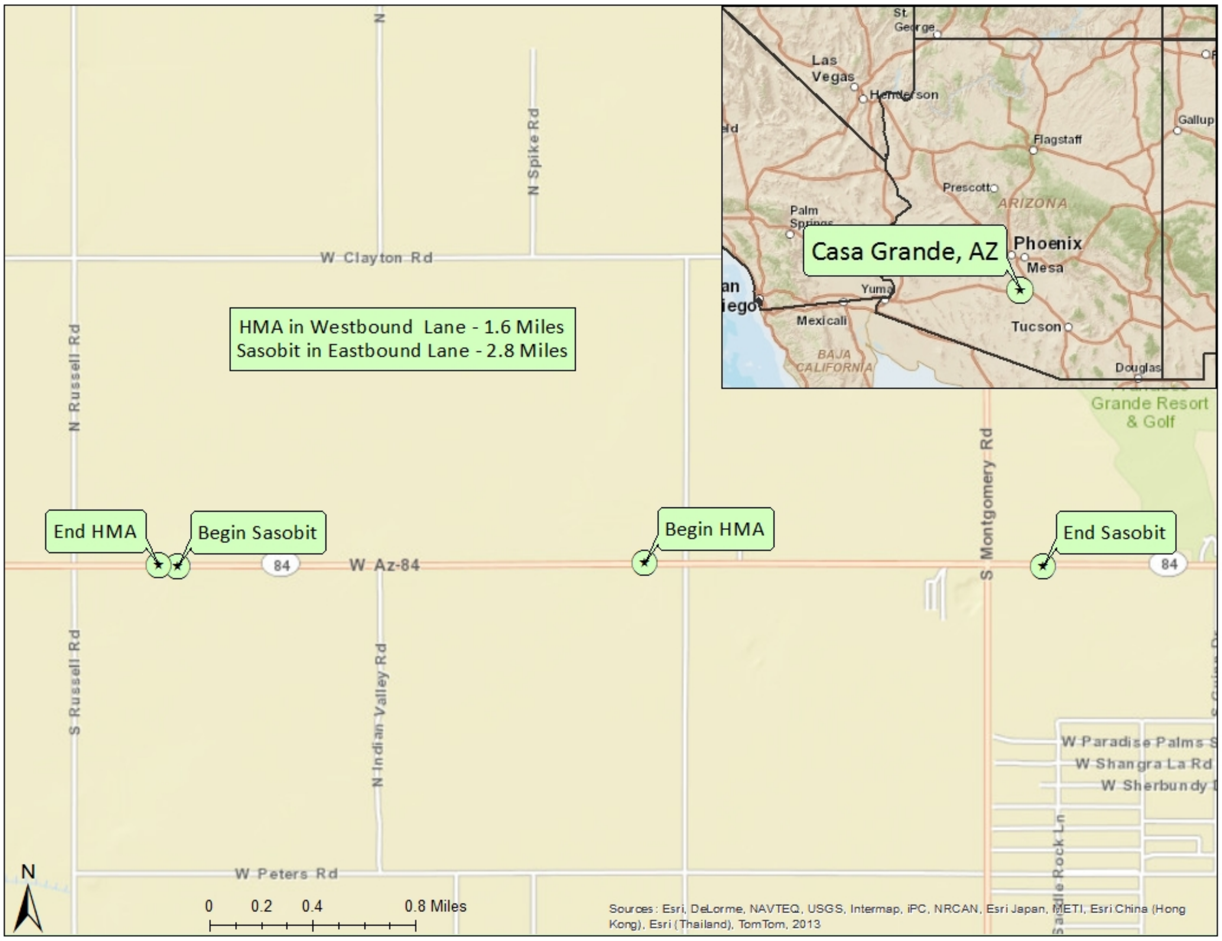


Figure 107 Locations of Test Sections in Casa Grande, Arizona

The temperature of the mix behind the paver was measured using a hand-held temperature gun and the PAVE-IR system. Two temperature readings were taken with the hand held temperature gun every 5-20 minutes, and the two readings were averaged to yield the temperature reading at that location and time. Table 152 shows the temperatures from behind the screed using both measuring techniques.

Table 152 Temperatures behind the Screed in Casa Grande, AZ

Temperature (°F)	Measuring Device	HMA	Sasobit
Average	Temperature Gun	299.7	254.3
	PAVE-IR	297.0	257.0
Standard Deviation	Temperature Gun	14.6	11.8
	PAVE-IR	20.4	212
Maximum	Temperature Gun	345.5	284.0
	PAVE-IR	340.0	330.0
Minimum	Temperature Gun	279.0	234.5
	PAVE-IR	220.0	210.0

Weather data was collected hourly at the paving location using a handheld weather station. The ambient temperature during the construction of the HMA ranged from 34.3°F to 61.0°F with an average temperature of 50.6°F.The average wind speed was 2.5 miles per hour (mph) and the average humidity was 43.2 percent. The ambient temperature during construction of the WMA ranged from 38.8°F to 62.5°F with an average ambient temperature of 50.5°F. The wind speed and humidity for the WMA construction were 3.5 mph and 48.4 percent, respectively. It was sunny with no rain during the paving of both mixes.

The HMA was compacted using three Ingersoll-Rand steel wheel rollers and one Ingersoll-Rand rubber tire roller for a portion of the day. Two steel wheel rollers were operated in tandem as the breakdown rollers with four vibratory passes (up and back twice) and then one static pass. The rubber tire roller was used as the intermediate roller performing four passes across the mat. Lastly, a third steel-wheel roller operating as the finishing roller made one vibratory pass and four static passes. The rubber tire roller began to pick up mix so it was removed from the paving train. The rolling pattern for the WMA was the same as for the HMA except the rubber tire roller was never used due to the problems of HMA sticking to the tires the previous day.

Construction Core Testing

The day after construction of each mix, seven 4-inch (101.6-mm) cores were obtained from each section (HMA and Sasobit) to determine in place densities and tensile strengths. Average test results are shown in Table 153.

The average core density for the WMA section was 1.8 percent higher than the HMA. This could have been due to increased compactability of the WMA or just normal variation. The tensile strengths for both mixes were reasonable with the Sasobit mix having approximately 17-psi higher tensile strength.

Table 153 Construction Cores Test Results from Casa Grande, AZ

Property	Statistic	HMA	Sasobit
In-place Density (% of G_{mm})	Average	90.6	92.4
	Standard Deviation	2.1	1.3
Tensile Strength (psi)	Average	118.0	135.9
	Standard Deviation	17.8	10.3

Field Performance at 9-Month Inspection

A field-performance evaluation was conducted on August 30, 2012. As stated earlier, this segment of SR-84 had been topped with a chip seal. Data were collected on each section to document rutting and cracking performance. Raveling could not be analyzed on these mixes because of the chip seal. In addition, three 4-inch (101.6 mm) diameter cores were taken from the outside wheelpath, and five 4-inch (101.6 mm) diameter cores were taken from in between the wheelpath. Four-inch (101.6 mm) diameter cores were taken to determine the in-place density, indirect tensile strengths, theoretical maximum specific gravity, gradation, asphalt content, and the true binder grade for each mix.

After nine months, the HMA had an average of 3.18-mm of rutting, while no rutting was observed in the WMA section. Both sections had performed well in terms of rutting after nine months. Each 200 ft. (61 m) evaluation section was carefully inspected for visual signs of cracking. No cracking was evident for either mix through the chip seal at the time of the nine-month inspection.

Core Testing

At the time of the nine-month project inspection, eight 4-inch (101.6 mm) cores were taken from each mix section. The densities of these cores were measured using AASHTO T 166 after the chip seal was removed. Seven of the cores were then tested for tensile strength using ASTM D6931. These seven samples were then combined and the cut faces were removed. This mix was split into two samples that were used to determine the maximum specific gravity according to AASHTO T 209. A summary of the core testing is shown in Table 154

The gradations were similar for both mixes at the time of the inspection and were similar to the gradations from production. The asphalt contents of the nine-month cores were higher for the HMA compared to the as-constructed mix samples. This is likely due to some binder from the chip seal being absorbed by the mix. The in-place densities were similar for both mixes at the time of the inspection and both had increased since construction, as expected. The tensile strength of the Sasobit WMA was higher than the HMA at the time of construction. Sasobit typically stiffens the asphalt binder which may explain the higher tensile strength. After nine-months the tensile strengths had nearly doubled for both mixes. This increase can likely be attributed to rapid binder aging in the desert climate.

Table 154 Test Results on Production Mix and Nine-Month Cores from Casa Grande, AZ

Property	HMA	Sasobit WMA	HMA	Sasobit WMA
	Production Mix (December 2011)		Nine-Month Cores (August 2012)	
Sieve Size	% Passing		% Passing	
25.0 mm (1”)	100.0	100.0	100.0	100.0
19.0 mm (3/4”)	98.4	98.1	98.8	98.1
12.5 mm (1/2”)	88.7	87.2	90.6	88.4
9.5 mm (3/8”)	79.5	77.2	81.5	78.7
4.75 mm (#4)	57.3	55.3	61.0	56.4
2.36 mm (#8)	42.3	42.9	45.9	41.3
1.18 mm (#16)	29.5	29.2	32.3	28.7
0.60 mm (#30)	20.4	20.1	22.2	20.0
0.30 mm (#50)	12.4	12.0	13.3	12.3
0.15 mm (#100)	7.9	7.6	8.2	7.6
0.075 mm (#200)	5.6	5.4	5.6	5.2
Asphalt Content (%)	4.55	4.47	5.02	4.65
G_{mm}	2.482	2.484	2.458	2.458
G_{mb}	2.250*	2.295*	2.304	2.323
In-place Density (%)	90.6*	92.4*	93.8	94.5
P_{ba} (%)	0.64	0.62	0.51	0.27
Tensile Strength (psi)	118.0*	135.9*	237.8	248.7

*Data from construction cores, not mix sampled during production as identified in column header.

Table 155 shows the average densities and tensile strengths by location for the nine-month inspection cores. The in-place densities for both mixes were slightly higher in the wheelpaths than in between, as expected. Also, the tensile strengths were slightly lower between the wheelpaths, but the difference is minimal.

Table 155 In-place Densities and Tensile Strengths by Location in Casa Grande, AZ

Location and Property	HMA	Sasobit
	Nine-Month Cores	
Between Wheelpaths Density (% of G_{mm})	93.3	94.1
In Right Wheelpath Density (% of G_{mm})	94.6	95.1
Between Wheelpaths Tensile Strength (psi)	231.6	239.8
In Right Wheelpath Tensile Strength (psi)	246.1	260.6

Performance Prediction

The initial AADTT for SR-84 in Case Grande, AZ was 456 trucks per day with one lane in each direction. A traffic growth rate of 4.8 percent was calculated from Arizona DOT's ESAL estimation for the project. SR-84 was classified as a minor arterial. Table 156 summarizes the pavement structure.

Table 156 SR-84 Casa Grande, AZ Pavement Structure

Layer	Thickness, in. [cm]
WMA/HMA Surface Course	2.1 [5.3]
Existing 3/4- inch HMA – 19.0 mm NMAS with PG 70-10	2.9 [7.4]
Uncrushed Gravel	9.0 [22.9]
AASHTO A-7-5 Subgrade	Semi-infinite

Figure 108 shows a comparison of the predicted rutting for the WMA and HMA sections. The MEPDG predicts that for the total asphalt section both the HMA and WMA will reach 0.25 inches of rutting at 187 months of service. The total predicted asphalt rutting after 20-years of service is 0.30 in. (7.6 mm) for both the WMA and HMA. The predicted rutting for the surface layers after 20 years is only 0.08 in. (2 mm).

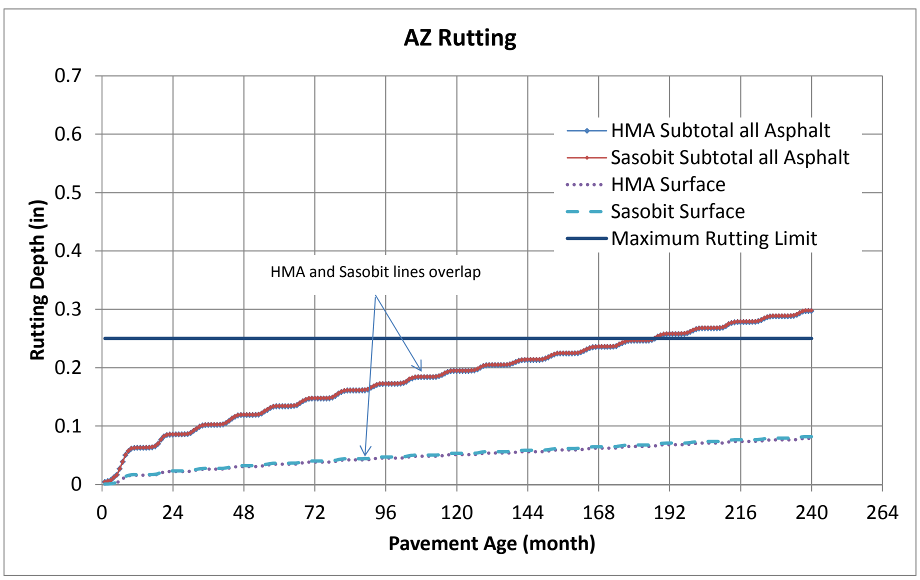


Figure 108 MEPDG Predicted Rutting SR-84, Casa Grande, AZ

Figure 109 shows a comparison of the predicted longitudinal, top-down cracking for Casa Grande, AZ. Both the WMA and HMA exceeded the recommended maximum limit for top-down cracking, the HMA after 161 months and the WMA after 223 months. The total predicted cracking after 20-years of services is 3,830 and 2,290 ft./mi. (726 and 444 m/km), respectively for the HMA and WMA.

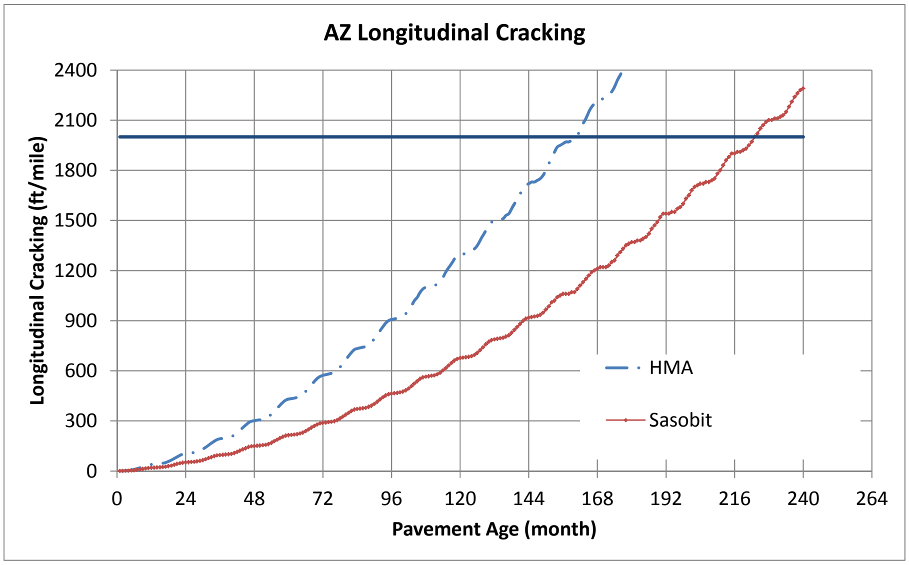


Figure 109 MEPDG Predicted Longitudinal Cracking for SR-84, Casa Grande, AZ

Comparison of Observed and Predicted Performance of WMA and HMA for New Projects

When evaluating new technologies, it is desirable to compare the long-term performance of the new and existing technologies. Since desired pavement performance is in the range of 12 to 20 years, it is generally impractical to base comparisons on the long-term performance of field tests sections. Accelerated loading facilities, performance prediction tests and performance prediction models may be used to evaluate expected long-term performance. These results must always be tempered with field performance experience. The following section compares the observed and the predicted performance from the MEPDG of the new projects' HMA and WMA for up to two-years (12 and 24 month revisits) after construction. Comparisons are then made between the predicted performance of HMA and WMA for 12 and 20 years after construction. Thus, a total of four prediction intervals: 12 months, 24 months, 12 years, and 20 years are presented. Predicted rutting, longitudinal, top-down cracking, and thermal cracking are evaluated. Thermal cracking is only evaluated for projects with Level I IDT inputs at temperatures accepted by the MEPDG (Rapid River, Michigan was excluded due to lower IDT test temperatures).

Rutting

The MEPDG predicts rutting of each asphalt layer, provides a sub-total of expected rutting for the asphalt layers, predicts the rutting of the base and subgrade layers, and provides the total expected pavement rutting. The observed field performance over the short-term was compared to the subtotal of predicted rutting for all of the asphalt layers. The predicted and observed data for the subtotal of all asphalt layers are summarized in Table 157.

Table 157 Observed and Predicted Rut Depths (mm) – Subtotal of all Asphalt Layers

Project	Mix	Approximately 12 Months		Approximately 24 Months		12 Years	20 Years
		Observed	Predicted	Observed	Predicted	Predicted	
Walla Walla, WA	HMA	1.0	3.0	4.6	4.7	9.9	13.5
	Maxam	0.0	3.3	0.3	5.0	10.6	14.3
Centreville, VA	HMA	0.0	1.8	3.2	1.9	4.5	6.0
	Astec DBG	0.0	1.8	2.7	2.0	4.5	6.0
Rapid River, MI	HMA	0.0	0.6	0.0	0.7	1.6	2.1
	Advera	0.0	0.2	0.0	0.4	1.0	1.3
	Evotherm	0.0	0.6	0.0	0.7	1.6	2.1
Baker, MT	HMA	0.4	0.8	0.5	0.8	2.5	3.3
	Evotherm	0.2	0.8	0.2	0.8	2.7	3.5
Munster, IN	HMA	0.0	2.4	0.0	3.6	9.5	12.4
	Evotherm	0.0	2.4	0.0	3.6	9.6	12.6
	Gencor Foam	0.0	2.4	0.0	3.7	9.8	12.8
	Wax	0.0	2.4	0.0	3.6	9.7	12.7
Jefferson County, FL	HMA	1.9	2.7	2.9	3.9	8.6	11.0
	Terex Foam	2.4	2.7	3.0	3.9	8.7	11.1
NYC, NY	HMA	1.0	0.5	1.9	0.5	1.7	2.6
	Bitutech	0.7	0.6	2.7	1.0	2.1	3.1
	Cecabase	0.3	0.6	0.3	1.1	2.2	3.2
	SonneWarmix	0.0	0.7	0.0	1.0	2.5	3.7
Casa Grande, AZ	HMA	3.2	1.4	NA	2.2	0.5	7.5
	Sasobit	0.0	1.5	NA	2.2	0.5	7.6

Figure 110 shows a comparison of the observed and predicted rutting. The predicted rutting was selected for the same months in which the field inspection occurred. The data that approximates both the 12 and 24 month field visits is shown in Table 158. The MEPDG generally over predicts the observed rut depths, more so for the WMA although the linear regression between predicted and observed rut depth is very poor.

Table 158 Predicted Rut Depths (mm) – Experimental (Surface) Layer

Project	Mix	Approx. 12 Months	Approx. 24 Months	12 Years	20 Years
Walla Walla, WA	HMA	0.9	1.4	3.2	4.4
	Aquablack	1.2	1.8	4.0	5.4
Centreville, VA	HMA	0.5	0.5	1.2	1.6
	Astec DBG	0.5	0.5	1.2	1.5
Rapid River, MI	HMA	0.6	0.7	1.6	2.1
	Advera	0.2	0.4	1.0	1.3
	Evotherm	0.6	0.7	1.6	2.1
Baker, MT	HMA	0.2	0.2	0.5	0.7
	Evotherm	0.1	0.1	0.4	0.5
Munster, IN	HMA	0.5	0.7	1.9	2.5
	Evotherm	0.5	0.7	2.0	2.6
	Gencor Foam	0.5	0.8	2.1	2.7
	Wax	0.5	0.7	2.0	2.6
Jefferson County, FL	HMA	0.6	0.9	1.8	2.2
	Terex Foam	0.7	1.0	1.9	2.3
NYC, NY	HMA	0.4	0.6	1.4	2.1
	Bitutech	0.5	0.7	1.8	2.7
	Cecabase	0.6	0.8	1.9	2.8
	SonneWarmix	0.7	0.9	2.2	3.2
Casa Grande, AZ	HMA	0.4	0.6	1.4	2.0
	Sasobit	0.4	0.6	1.5	2.1

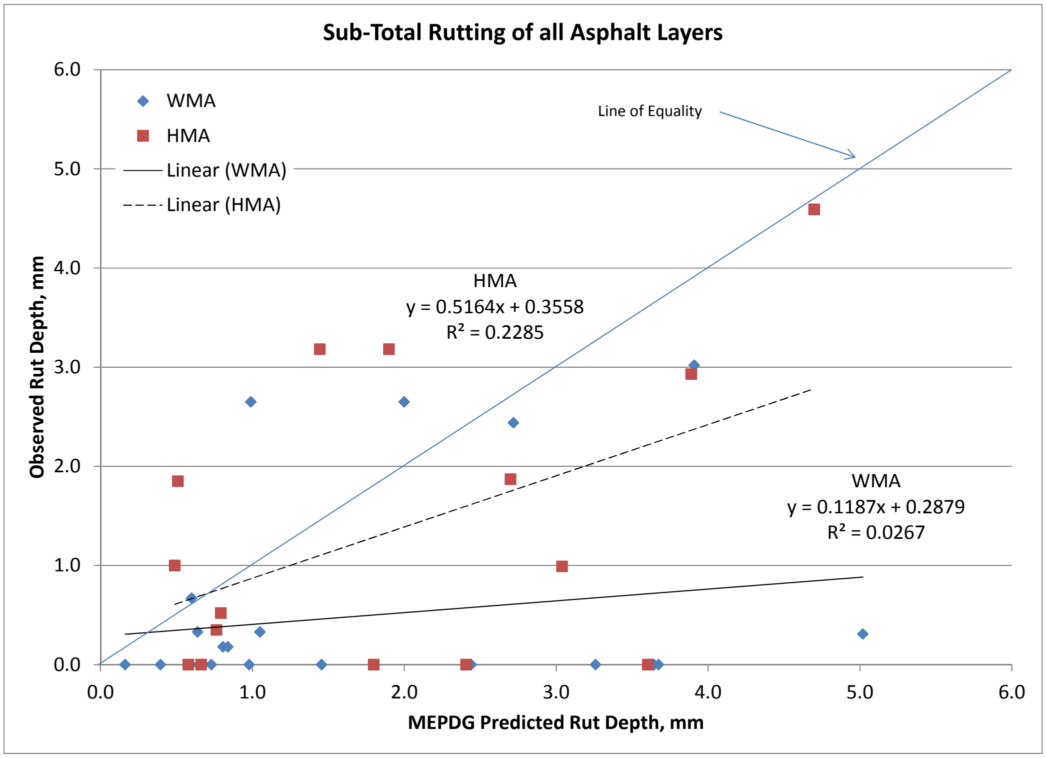


Figure 110 Predicted vs. Observed Rut Depths for New Projects WMA and HMA

Two-sample, paired *t*-tests were performed between the predicted WMA and HMA rut depths at both 12 and 24 months. The comparison was performed for both the subtotal of all asphalt layers and the experimental (surface) layers. The results are summarized in Table 159. Numerically, the mean rut depth for the WMA mixes is always greater; however, that difference is very small, approximately 0.2 mm. At 95 percent confidence, the paired *t*-tests indicate that the 12 and 20 year rut depth predictions are the same. Although it is a poor correlation, Figure 110 indicates that the MEPDG over-prediction of rutting is greater for WMA compared with HMA. Overall, however, the performance predictions indicate WMA should perform as well as HMA in terms of rutting.

Table 159 Summary of Statistical Analyses to Compare Predicted Rutting

Layer(s)	Prediction Interval, years	Mix	Mean Rut Depth, mm	Variance	*t*-test *p-value* (two-tail)
Subtotal all Asphalt Layers	12	HMA	4.84	15.0	0.08
		WMA	5.03	15.6	
	24	HMA	6.96	22.4	0.06
		WMA	7.23	23.2	
Experimental (Surface) Layer	12	HMA	1.65	0.36	0.16
		WMA	1.80	0.67	
	24	HMA	2.22	0.65	0.14
		WMA	2.45	1.31	

Longitudinal, Top-Down Cracking

The MEPDG predicts longitudinal top-down and bottom-up fatigue cracking. Because the experimental mixes were surface mixes, bottom-up fatigue cracking predictions are not presented. Bottom-up fatigue cracking predictions would be influenced more by the supporting pavement layers. The observed field performance over the short-term was compared to the predicted longitudinal, top-down cracking. The observed cracking in the three 200 ft. (61 m) long monitoring sections were normalized to feet per mile. The predicted and observed data are summarized in Table 160.

Table 160 Observed and Predicted Longitudinal, Top-Down Cracking (ft./mile)

Project	Mix	Approximately 12 Months		Approximately 24 Months		12 Years	20 Years
		Observed Normalized	Predicted	Observed Normalized	Predicted	Predicted	
Walla Walla, WA	HMA	0	0	0	1	13	35
	Aquablack	0	1	0	2	23	62
Centreville, VA	HMA	0	1	0	1	9	21
	Astec DBG	0	0	0	0	4	10
Rapid River, MI	HMA	0	8	4	14	266	550
	Advera	4	2	4	4	66	139
	Evotherm	18	8	18	12	214	434
Baker, MT	HMA	0	6	0	11	337	822
	Evotherm	0	8	0	15	428	1,030
Munster, IN	HMA	0	461	97	1,500	8,010	9,290
	Evotherm	0	268	0	949	7,160	8,810
	Foam	97	386	678	1,360	7,940	9,270
	Wax	0	716	0	2,280	9,020	9,850
Jefferson County, FL	HMA	0	4	0	15	285	649
	Terex	0	10	0	34	605	1,320
NYC, NY	HMA	0	0	97	0	0	0
	Bitutech	0	0	150	0	0	0
	Cecabase	0	0	440	0	0	1
	SonneWarmix	0	0	308	0	1	3
Casa Grande, AZ	HMA	0	26	NA	104	1,720	3,820
	Sasobit	0	13	NA	51	918	2,290

Figure 111 shows a comparison of the observed and predicted cracking. The data that approximates both the 12 and 24 month field visits is shown. The MEPDG generally overestimates the predicted cracking. Similar to the rutting prediction, the relationship between the observed and predicted cracking is poorer for the WMA compared to the HMA.

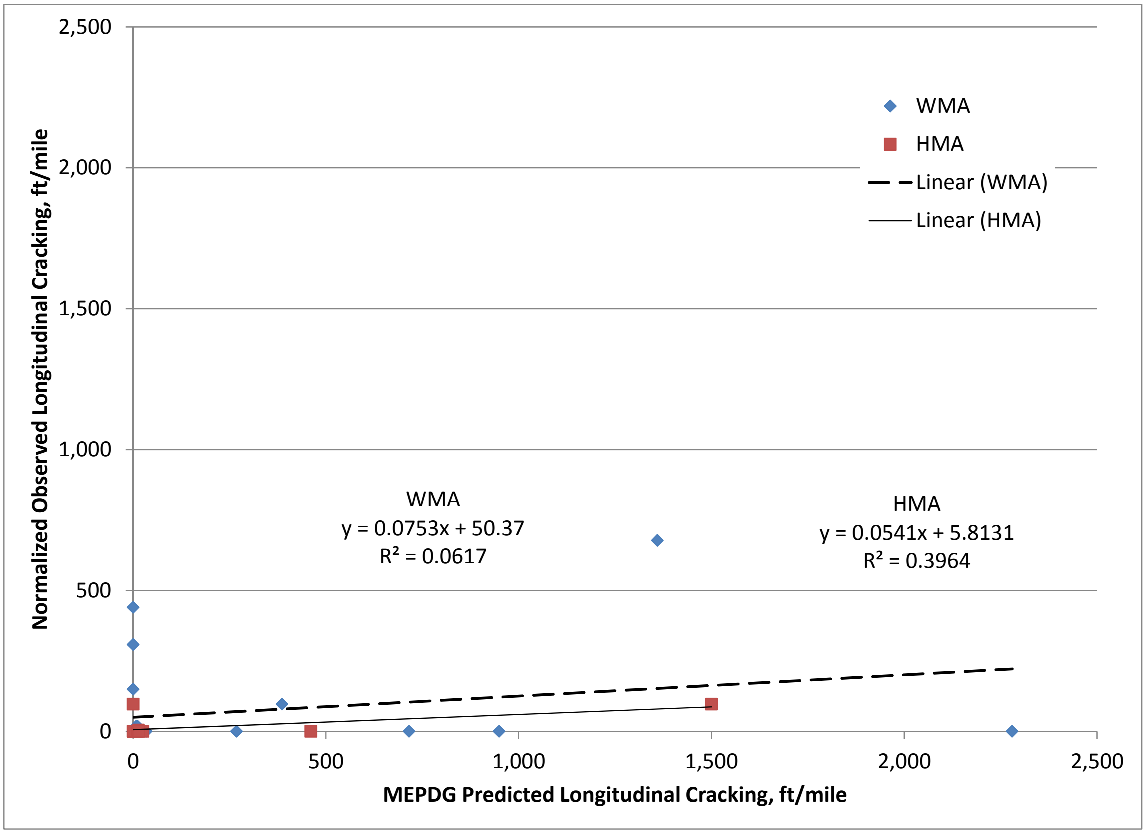


Figure 111 Predicted vs. Observed Top-Down, Longitudinal Cracking for New Projects

Two-sample, paired *t*-tests were performed between the predicted WMA and HMA top-down, longitudinal cracking at both 12 and 24 months. The results are summarized in Table 161. Numerically, the predicted HMA cracking is greater than the predicted WMA cracking in 6 of 13 cases and identical in 2 of 13 cases. The mean predicted cracking for the WMA mixes is always less. At 95 percent confidence, the paired *t*-tests indicate that the 12 and 20 year top-down cracking predictions are the same. The performance predictions indicate WMA should perform as well as HMA in terms of top-down cracking.

Table 161 Summary of Statistical Analyses to Compare Predicted Top-Down Cracking

Prediction Interval, years	Mix	Mean Cracking, ft/mile	Variance	*t*-test *p-value* (two-tail)
12	HMA	2,071	11,667,256	0.75
	WMA	2,029	11,965,250	
24	HMA	2,640	15,385,014	0.58
	WMA	2,556	15,329,011	

Thermal Cracking

Thermal cracking comparisons are only presented for projects with Level I IDT data compatible with the MEPDG. The Michigan IDT tests were conducted at lower temperatures due to the binder grade so that data could not be used in the MEPDG. Table 162 presents the predicted thermal cracking after 12 and 20 years of service. Table 163 presents the statistical comparison. In all cases the thermal cracking predicted for the WMA was less than or equal to the thermal cracking predicted for the HMA. Paired, two-sample *t*-tests indicate no significant difference between the predicted WMA and HMA cracking at 95 percent confidence. Based on the performance predictions, the WMA would generally be expected to perform better than the HMA. From the Indiana data, the Heritage wax does not seem to have a detrimental effect on low temperature performance.

Table 162 Predicted Thermal Cracking, ft/mile

Project	Mix	12 Year	20 Year
Walla Walla, WA	HMA	0	0
	Aquablack	0	0
Centreville, VA	HMA	0	0
	Astec DBG	0	0
Baker, MT	HMA	1,584	1,750
	Evotherm DAT	1,512	1,731
Griffith, IN	HMA	1,825	1,869
	Evotherm	1	3
	Gencor Foam	1,563	1,752
	Heritage wax	299	731

Table 163 Summary of Statistical Analyses to Compare Predicted Thermal Cracking

Prediction Interval, years	Mix	Mean Cracking, ft./mile	Variance	*t*-test *p-value* (two-tail)
12	HMA	1,176	838,811	0.13
	WMA	562	583,605	
24	HMA	1,226	904,292	0.17
	WMA	703	727,396	

Summary of Performance Prediction Comparisons

Comparisons were made between the short-term observed and predicted performance for the HMA and WMA in the new projects. The MEPDG generally over-predicted rutting and longitudinal cracking. The predictions for the HMA showed a slightly better correlation with the observed data. Comparisons of the predicted rutting after 12 and 20-years of service suggest that HMA will perform slightly better than WMA, on the order of 0.2 mm less rutting. The difference is not statistically or practically significant. In 6 of 13 cases for both the 12 and 20 year prediction, less top-down, longitudinal cracking is predicted for the WMA; in 2 of 13 cases the predictions are identical. The predicted top-down cracking is not significantly different between WMA and HMA. Level I IDT data was used in the MEPDG for four project sites. No thermal cracking was predicted after 20 years of service for two of the sites. For the remaining two sites (one multi-technology), the predicted thermal cracking for the WMA was also less than for the HMA. The differences, however, were not statistically significant. Overall, the performance prediction indicate that WMA should perform as well as HMA, and possibly better, in terms of cracking. Slightly more rutting might be expected, but this increase is practically and statistically insignificant.

Practical Guidelines for Production and Placement of WMA

Best practices for production and placement of WMA are not very different from those that have long been advocated for HMA. This section highlights best practices and documented benefits of WMA and areas of potential concern observed during the construction of the field tests sections. In some cases, interested readers are directed to other sources for potential solutions. There is no single best practice to address every situation. Instead, a variety of practices are offered for the user to consider.

Stockpile Moisture Content

Minimizing stockpile moisture contents is a best practice for both WMA and HMA. An early concern with WMA was incomplete drying of the aggregate at reduced production temperatures. However, moisture contents measured on numerous plant-produced HMA and WMA mix samples in this study have shown that incomplete drying of aggregates during WMA production is not a problem. Nonetheless, reducing stockpile moisture contents is beneficial in energy saving for asphalt mixture production. An industry rule-of-thumb is that fuel usage decreases ten percent for every one percent decrease in stockpile moisture content. Reducing stockpile moisture contents saves fuel, even with WMA.

The aggregates used on the Baker, MT project had average moisture contents that were 1.9 percent lower than the average for the other seven projects, resulting in an average fuel savings of 0.052 MMBtu/ton per percent moisture content compared to HMA produced at the same temperature. This savings actually exceeded the 10 percent rule-of-thumb.

Fine aggregate and RAP stockpiles tend to have higher moisture contents than coarse aggregate stockpiles do. Therefore, these stockpiles should be addressed first. There are a number of ways to reduce stockpile moisture content, such as placing stockpiles on surfaces sloped away from the plant and loading from the high side or covering stockpiles *(10)*.

Maintaining Adequate Baghouse Temperatures

One potential challenge in the production of WMA can be keeping baghouse temperatures high enough to prevent condensation. Condensation causes two problems: corrosion of the baghouse and the formation of “mud” (damp baghouse fines). In well-maintained baghouses, inlet temperatures should be above 220°F (104°C) for low-sulfur fuels and 240 to 250°F (116 to 121°C) for high-sulfur fuels, such as reclaimed oils. High-sulfur fuels produce acidic gases that attack steel if they condense on cooler surfaces such as baghouse tube sheets. The critical temperature, however, is the dew point of the exhaust stream. This is the temperature at which water vapor in the exhaust stream will condense into liquid water. The typical dew point for asphalt plant exhaust streams ranges from approximately 170 to 180°F.

Ideally, it is desirable to transfer as much heat as possible from the burner exhaust stream to the aggregate, resulting in lower baghouse and stack temperatures. Low baghouse temperatures are less likely with parallel-flow plants than with more efficient counter-flow plants. Typically, exhaust gases for parallel-flow drum plants range from 20°F (11°C) cooler to 50°F (28°C) hotter than mix discharge temperatures.

Mix, baghouse inlet (where available), and stack (baghouse outlet) temperatures were recorded at approximately 15 minute intervals during the production of the mixes for the “new” projects in this study. The average and minimum mix and stack temperatures are reported for each mix in Table 164. Also noted is the plant configuration and fuel type. With the exception of independent checks of mix temperature, the research team did not check the accuracy of the plant temperature measurements.

Table 164 Average and Minimum Mix and Stack Temperatures

Project, Plant Type, Fuel	Mix Section	Avg. Mix Temp., °F	Min. Mix Temp., °F	Avg. Stack Temp., °F	Min. Stack Temp., °F
Walla Walla, WA PF Drum Natural Gas	HMA	325	312	339	330
	Terex Foam	285	274	295	266
Centreville, VA Double-Barrel Natural Gas	HMA	318	294	218	213
	Astec DBG	288	280	192	180
Rapid River, MI PF Drum Reclaimed motor oil	HMA	302	273	310	269
	Advera WMX	269	254	278	247
	Evotherm 3G	271	257	284	272
Baker, MT PF Drum Liquid propane	HMA	299	293	249	216
	Evotherm DAT	252	242	238	217
Munster, IN CF Drum Natural gas	HMA	300	290	241	231
	Gencor Foam	277	265	233	226
	Evotherm 3G	255	248	218	213
	Heritage Wax	268	243	225	220
Jefferson County, FL CF Drum Reclaimed motor oil	HMA	334	316	174	159
	Terex Foam	297	279	175	156
NYC, NY Batch/mini-drum Natural gas	HMA	344	318	332	306
	Cecabase RT	245	200	251	235
	SonneWarmix	270	238	231	204
	BituTech PER	279	260	238	209
Casa Grande, AZ PF Drum Reclaimed motor oil	HMA	319	285	212	183
	Sasobit	276	222	181	148

Average stack temperatures were greater than 180°F for nineteen of twenty-one mixes. The exceptions were the WMA and HMA from Florida, the WMA from the Centreville, VA, and Casa Grande, AZ. The minimum stack temperatures for these mixes was less than or equal to 180°F. The Florida plant and the Arizona plant used recycled fuel which can have high sulfur contents. Although there were no reports of baghouse "mudding" during the trial sections, all of the production runs were relatively short.

Young *(27)* provides several best practices for minimizing condensation in the baghouse and preventing damage from corrosion when running at normal HMA production temperatures. These best practices are even more important when running WMA on a regular basis.

- Seal air leaks, particularly the seals on the baghouse doors and around dryer breeching.

Air leaks cause two problems: first, the introduction of cooler ambient air can reduce the overall temperature of the exhaust stream, leading to condensation; second, air leaks waste fan capacity, thereby lowering the maximum production rate.

- Preheat the baghouse for 15 to 20 minutes to heat the steel housing completely. Experience has shown that it is also beneficial to start WMA production at a slightly higher temperature.
- Inspect the fines return lines more frequently to ensure that no buildup due to moisture occurs. Typically, fines at lower temperatures are more susceptible to moisture, affecting flow back into the mix.
- Condensation may only occur in a limited portion of the baghouse, such as the windward side. In this case, periodic painting of the interior surfaces can minimize corrosion and insulation of exterior surfaces can reduce heat loss.

The minimum exhaust temperature necessary to avoid problems with condensation and returning baghouse fines will vary from plant to plant and from mix to mix. Cold weather and high aggregate moisture can be a dangerous combination when it comes to condensation and dust problems. Tight, well-maintained plants can be more sensitive to condensation due to higher moisture concentrations in the exhaust gas. Several strategies suitable for increasing baghouse temperatures are outlined in Warm-Mix Asphalt: Best Practices, 3rd edition *(10)*. Some are quick to implement while others are inexpensive. Some options require equipment upgrades that offer more benefits than simply raising stack temperatures.

Burner Performance

An improperly tuned burner can increase fuel usage and result in mix contamination. An expert on the project team conducted burner tuning for the NCHRP 9-47A team before each of the multi-technology projects (MI, IN, and NY). One plant had a 24.8 percent reduction in fuel usage for HMA after burner tuning. One symptom of improper burner adjustment and maintenance is unburned fuel. Unburned liquid fuels can contaminate the mix, leading to a binder which is less stiff than desired. The potential for mix damage from uncombusted fuel is probably greater for WMA than for HMA, because unburned fuel is more likely to vaporize at HMA temperatures. Uncombusted fuel was observed in a few early WMA trial projects before this study was initiated. WMA contaminated with fuel oil can be detected by a brown coloration of the coated aggregate. Performance testing of fuel contaminated mixes will also yield increased rutting susceptibility and lower dynamic modulus (stiffness) values. If fuels are not combusted, stack emissions tests will also indicate elevated levels of carbon monoxide (CO) and total hydrocarbons (THC).

Most burners have one modulating actuator motor with mechanical linkage driving dampers and fuel valves. The challenge with a mechanical linkage is making sure that the air to fuel ratio is optimal through the full operating range. Some contractors have reported difficulties adjusting burners to sufficiently low levels to reach the desired production temperatures for WMA. This problem has generally been exacerbated when the plant runs at a very slow production rate for a small WMA trial. At normal production rates, most burners should be able to produce the lower temperatures required for WMA. In any case, a contractor attempting their first WMA trial should have an experienced burner technician inspect the burner and aid with adjustments.

There can be a number of causes for uncombusted fuel with both WMA and HMA. Clogged burner nozzles and fuel filters are always good places to start looking. When burning heavy or reclaimed fuel oil, maintaining the fuel preheater temperatures to obtain a suitable viscosity for fuel atomization and accelerated pump wear are frequent problem areas.

Producing Mixes with RAP and RAS

The addition of even a relatively small percentage of RAP to WMA can greatly aid in drying the virgin aggregate and increasing the baghouse temperature with no detrimental consequences. For a discharge temperature of 220°F, the virgin aggregate must be superheated to a temperature of 280°F for a batch plant running a mixture with 10 percent RAP with a moisture content of 3 percent *(27)*. Superheating the virgin aggregate will increase the likelihood that the internal moisture in the virgin aggregate is removed. Superheating the virgin aggregate will also increase the temperature of the exhaust gases going to the baghouse. Thus the addition of a small amount of RAP helps to satisfy both needs. The mix designs for seven of eight NCHRP Project 9-47A field trials included at least 12 percent RAP; the Baker, MT project used a virgin mix.

On the performance side, one purported benefit of WMA is reduced aging of the binder. Performance grading of binder recovered from the NCHRP Project 9-47A fields sections generally supports this. Nine of fourteen WMA mixes had low temperature true grades which were lower than the corresponding HMA control mixes. The five remaining WMAs had low temperature true grades within 0.6°C of the HMA control. Only one WMA had a recovered high-temperature true grade higher than its corresponding HMA (VA, 1.2°C). The addition of RAP to WMA production should also increase the early-life composite stiffness of the mixture, helping to counteract any concerns over the impact of reduced aging on high-temperature performance.

Placement Changes

Several contractors have commented that equipment remains cleaner with less asphalt buildup when placing WMA. In a few instances, material flow issues have been observed at asphalt plants and when dumping into transfer vehicles or pavers, most likely due to the reduced temperatures. Observed differences included:

- Sluggish flow of mix into vertical bucket elevator – resolved by slight increase in mix temperature (pre-NCHRP Project 9-47A project),
- Sticking of silo gate, and
- Trucks need to raise bed higher to break the load when dumping.

Hand work can be difficult at reduced temperatures, particularly in urban environments where more hand work is required for manholes, storm water inlet grates, valves, and so forth. The New York, NY project required a significant amount of hand work by the paving crew. Figure 112 shows the hand work associated with one typical intersection including a storm water inlet just outside the bottom of the picture. The crew reported a significant improvement in workability with a 25°F increase in average production temperature between the Cecabase RT and SonneWarmix and BituTech PER. Thus, WMA can be used where handwork is required, even with 20 percent RAP, but care must be used to select appropriate production temperatures.

Figure 112 Typical Handwork in Urban Paving Project

Compaction

WMA technologies are compaction aids. However, the compaction benefits may be offset by lower production and compaction temperatures. In general, for the lower WMA production temperatures measured in this study, there was not a reduction in the required compaction effort in the field compared to HMA. In nine of thirteen cases, the WMA achieved the same or better in-place density as the corresponding HMA during construction. For the four cases where the WMA in-place densities were lower, the average difference was within one percent and *t*-tests confirmed the averages were not statistically different with 95 percent confidence. Thus, there appears to be a tradeoff between reduction in production temperature and reduction in compaction effort. Compaction should be monitored using a non-destructive device, calibrated to cores, to ensure that adequate density is consistently being achieved.

The WMA on the Jefferson County, FL project exhibited a tender zone at intermediate compaction temperatures. Jim Warren of the Asphalt Contractors Association of Florida commented that the use of polymer modified PG 76-22 had largely eliminated the tender zone in Florida.

CHAPTER 4

ENGINEERING PROPERTIES OF HMA AND WMA

Statistical analyses were conducted to assess whether or not differences exist between WMA and HMA for the binder properties, mix characteristics, in-place properties, and laboratory-measured engineering properties. For projects with one WMA and an HMA control, F-tests and *t*-tests were used to compare the characteristics and properties that have replicate data with a 90% confidence interval (α=0.10). F-tests were used to compare variances of the properties; *t*-tests were used to compare means of the properties. For projects with more than one WMA technology, an Analysis of Variance (ANOVA) was used to detect statistical differences among the results. Some test results, such as TSR, do not have replicate data since they are computed from average tensile strength results. Comparisons of such properties for WMA and HMA were made using paired t-tests with the results from all projects.

For the mix properties, statistical analysis results were used to compare WMA and HMA sections in terms of equal, lower or higher performance. Equal performance indicates that no statistical differences were found in the results and lower or higher performance indicates there were differences between them.

BINDER PROPERTIES

The performance grades of the recovered asphalt binders were determined in accordance with AASHTO M 320 and AASHTO R 29 for all the mixes of each project under study. For the new projects asphalt binders were recovered from mixes sampled during construction and cores from inspections at approximately one and two years after construction. For the existing projects, asphalt binders were recovered from cores obtained from one inspection only, the age of these cores varies depending on the project (range between 30 and 65 months).

Table 165 to Table 172 present the true grade and performance grade of the extracted binders for all the mixes of each new project. The results are as follows:

Walla, Walla, WA (Table 165): The performance grades were the same for both WMA and HMA recovered binders at three different ages (production, 13 months, and 27 months). The high performance grade for both binders (HMA and WMA) were 1 grade lower at 13 months and 27 months than the high performance grade at production.

Centreville, VA (Table 166)*:* The performance grades were the same for HMA and WMA binders for the production mix and 27 month cores. For the 13 month cores, the high performance grade of the WMA-Astec DBG binder was one grade lower than the HMA binder. The low performance grades at 13 months were the same for both binders. It is also observed that the high performance grades for WMA and HMA binders were one grade lower at 24 months compared to the production mix, which is not expected since the binders should show a stiffer behavior.

Rapid River, MI (Table 167)*:*

- ✓ Production: The performance grades were the same for HMA and WMA binders.
- ✓ 13 months: The performance grades were the same for the WMA-Evotherm and HMA binders. The high and low performance grades of the WMA-Advera binders were one grade higher than the HMA binder.
- ✓ 22 months: The high performance grades were the same for the HMA and WMA-Advera binders, but the WMA-Evotherm binder was one grade lower than the HMA binder. The low performance grades were the same for all binders.

Baker, MT (Table 168)*:* The performance grades were the same for binders recovered from WMA and HMA at two different ages, production and 13 months. At 22 months, the WMA-Evotherm DAT binder was one grade lower at the high temperature grade and the low temperature grade was the same for both recovered binders.

Munster, IN (Table 169)*:*

- ✓ *Production:* The high performance grades were the same for HMA and all of the WMA recovered binders. The low performance grades were one grade higher for the WMA-Evotherm 3G and WMA-Gencor foam binders compared to the HMA binder.
- ✓ *13 months:* The high performance grades were the same for the HMA and two of the WMA binders, Evotherm 3G and Heritage wax. The WMA-Gencor foam was one grade lower. The low performance grades were the same for the HMA and WMA-Gencor foam binders, but they were one grade lower for the other two WMA binders, Evotherm 3G and Heritage wax.
- ✓ *24 months:* The high performance grades were the same for the HMA recovered binder and the recovered binder of two WMA mixes, Evotherm 3G and Gencor foam; the WMA-Heritage Wax binder was one grade higher than the HMA binder.

Jefferson Co. FL (Table 170)*:* The performance grades of HMA and WMA recovered binders were the same at construction. At 14 months the high performance grades were the same for both binders, but the low temperature grade was one grade (actually just 1.4 degrees) lower for the WMA-Terex foam binder. At 24 months, the high performance grade of the WMA-Terex foam binder was one grade lower than the HMA binder; the low temperature grade of the WMA-Terex foam binder was one grade higher than the HMA binder.

NYC, NY (Table 171):

- ✓ Production: The high performance grades of the recovered binders were the same for the HMA and WMA-SonneWarmix. For the other two WMA binders, Cecabase and BituTech PER, the high performance grades were one grade lower than the HMA binder. The low performance grades of the three WMA binders were one grade lower than the HMA binder.
- ✓ 13 months: The performance grades were the same for HMA and the WMA binders.
- ✓ 24 months: The high performance grades were the same for HMA binder and two WMA binders, Cecabase and SonneWarmix, the BituTech PER-WMA binder was one grade higher than the HMA binder. The low performance grades were the same for all the binders (HMA and WMA).

Casa Grande, AZ (Table 172)*:* The performance grades of the recovered binders were the same for the HMA and WMA-Sasobit for the construction mixes. At 13 months the high performance grade of the WMA-Sasobit binder was one grade higher than the HMA binder, the low performance grade was the same for both binders.

It can be observed that the performance grades for the HMA and WMA binders were the same for most of the projects at different ages with a few exceptions. But in all of these cases, the difference in binder grades was only one grade (up or down). It is also noticed that short-term field aging does not seems to have an effect on the performance grading obtained. For the cases were a difference was observed, the binder grades were changed only one grade (up or down), indicating little or no in-service aging of the binders. It seems likely that the pressure aging vessel (PAV) conditioning of the binders as part of the binder grading process may have masked some of the effects of plant and short-term aging of the binders.

Table 165 True and Performance Binder Grades at Different Ages-Walla, Walla, WA

Age	Grade	HMA	Aquablack
Production Mix	High Temp. Grade (°C)	77.9	75.3
	Low Temp Grade (°C)	-26.0	-27.3
	Performance Grade	76-22	76-22
13 months	High Temp. Grade (°C)	73.7	74.7
	Low Temp Grade (°C)	-27.2	-27.3
	Performance Grade	70-22	70-22
27 Months	High Temp. Grade (°C)	74.2	76.3
	Low Temp Grade (°C)	-26.2	-24.4
	Performance Grade	70-22	70-22

Table 166 True and Performance Binder Grades at Different Ages-Centreville, VA

Age	Grade	HMA	Astec DBG
Production Mix	High Temp. Grade (°C)	88.3	89.5
	Low Temp Grade (°C)	-20.1	-21.9
	Performance Grade	88-16	88-16
15 months	High Temp. Grade (°C)	92.3	83.7
	Low Temp Grade (°C)	-18.0	-22.2
	Performance Grade	88-16	82-22
24 Months	High Temp. Grade (°C)	83.5	84.6
	Low Temp Grade (°C)	-24.8	-22.7
	Performance Grade	82-22	82-22

Table 167 True and Performance Binder Grades at Different Ages-Rapid River, MI

Age	Grade	HMA	Evotherm	Advera
Production Mix	High Temp. Grade (°C)	59.0	58.1	59.7
	Low Temp Grade (°C)	-35.2	-34.8	-35.2
	Performance Grade	58-34	58-34	58-34
13 months	High Temp. Grade (°C)	57.2	55.7	60.2
	Low Temp Grade (°C)	-35.2	-34.6	-33.4
	Performance Grade	52-34	52-34	58-28
22 Months	High Temp. Grade (°C)	61.0	57.3	59.4
	Low Temp Grade (°C)	-34.5	-34.5	-34.5
	Performance Grade	58-34	52-34	58-34

Table 168 True and Performance Binder Grades at Different Ages-Baker, MT

Age	Grade	HMA	Evotherm DAT
Production Mix	High Temp. Grade (°C)	65.3	65.2
	Low Temp Grade (°C)	-31.2	-30.8
	Performance Grade	64-28	64-28
13 months	High Temp. Grade (°C)	66.5	65.4
	Low Temp Grade (°C)	-30.7	-33.0
	Performance Grade	64-28	64-28
22 Months	High Temp. Grade (°C)	66.5	62.6
	Low Temp Grade (°C)	-33.7	-32.5
	Performance Grade	64-28	58-28

Table 169 True and Performance Binder Grades at Different Ages-Munster, IN

Age	Grade	HMA	Evotherm 3G	Gencor Foam	Heritage Wax
Production Mix	High Temp. Grade (°C)	74.6	71.9	70.4	72.5
	Low Temp Grade (°C)	-21.0	-23.2	-22.8	-20.4
	Performance Grade	70-16	70-22	70-22	70-16
13 months	High Temp. Grade (°C)	72.1	71.0	68.9	70.0
	Low Temp Grade (°C)	-22.7	-21.5	-24.0	-21.6
	Performance Grade	70-22	70-16	64-22	70-16
24 Months	High Temp. Grade (°C)	75.0	71.5	73.7	76.9
	Low Temp Grade (°C)	-22.9	-23.6	-23.3	-18.5
	Performance Grade	70-22	70-22	70-22	76-16

Table 170 True and Performance Binder Grades at Different Ages-Jefferson Co. FL

Age	Grade	HMA	Terex
Production Mix	High Temp. Grade (°C)	92.5	90.4
	Low Temp Grade (°C)	-17.8	-17.2
	Performance Grade	88-16	88-16
14 months	High Temp. Grade (°C)	93.9	90.9
	Low Temp Grade (°C)	-15.3	-16.7
	Performance Grade	88-10	88-16
24 Months	High Temp. Grade (°C)	97.6	91
	Low Temp Grade (°C)	-12.2	-17.9
	Performance Grade	94-10	88-16

Table 171 True and Performance Binder Grades at Different Ages-NYC, NY

Age	Grade	HMA	Cecabase	Sonne-Warmix	BituTech PER
Production Mix	High Temp. Grade (°C)	74.6	68.9	70.1	69.3
	Low Temp Grade (°C)	-21.4	-26.2	-24.7	-24.9
	Performance Grade	70-16	64-22	70-22	64-22
15 months	High Temp. Grade (°C)	68.6	69.2	68.7	69.1
	Low Temp Grade (°C)	-23.1	-25.1	-24.9	-26.5
	Performance Grade	64-22	64-22	64-22	64-22
26 Months	High Temp. Grade (°C)	71.9	72.8	72.2	76.3
	Low Temp Grade (°C)	-23.8	-24.4	-25.1	-22.8
	Performance Grade	70-22	70-22	70-22	76-22

Table 172 True and Performance Binder Grades at Different Ages-Casa Grande, AZ

Age	Grade	HMA	Sasobit
Production Mix	High Temp. Grade (°C)	80	78
	Low Temp Grade (°C)	-14.3	-13.7
	Performance Grade	76-10	76-10
13 months	High Temp. Grade (°C)	74.4	78.6
	Low Temp Grade (°C)	-14.1	-15.1
	Performance Grade	70-10	76-10

Table 173 Temperature Difference-High and Low True Grade (WMA-HMA) at Different Ages

Location	WMA	Construction		1 Year Cores		2 Year Cores	
		High T_c	Low T_c	High T_c	Low T_c	High T_c	Low T_c
Walla Walla, WA	Aquablack	-2.6	-1.9	1	-0.1	2.1	1.8
Centerville, VA	Astec DBG	1.2	-1.8	-8.6	-4.2	1.1	2.1
Rapid River, MI	Evotherm 3G	-0.9	0.4	-1.5	0.7	-3.7	0
	Advera	0.7	0	3	1.9	-1.6	0
Baker, MT	Evotherm DAT	-0.1	0.4	-1.1	-2.3	-3.9	1.2
Munster, IN	Evotherm 3G	-2.7	-2.2	-1.1	1.2	-3.5	-0.7
	Gencor Ultrafoam	-4.2	-1.8	-2.3	-1.3	-1.3	-0.4
	Heritage Wax	-2.1	0.6	-2.1	1.1	1.9	4.4
Jefferson CO, FL	Terex CMI Foam	-2.1	0.6	-3	-1.4	-6.6	-5.7
New York, NY	Cecabase	-5.7	-4.8	0.6	-2	0.9	-0.6
	SonneWarmix	-4.5	-3.3	0.1	-1.8	0.3	-1.3
	BituTech PER	-5.3	-3.5	0.5	-3.4	4.4	1
Casa Grande, AZ	Sasobit	-2.0	0.6	4.2	-1.0	-	-
Average, WMA-HMA		-2.3	-1.3	-0.8	-1	-0.8	0.2
Maximum T_c Difference, WMA-HMA		-5.7	-4.8	-8.6	-4.2	-6.6	-5.7
Minimum T_c Difference, WMA-HMA		1.2	0.6	4.2	1.9	4.4	4.4

Table 173 shows the differences for the high and low true grades between WMA-HMA for the recovered binder at three ages; construction, 1st inspection (~13 months), and 2nd inspection (~24 months). From this table, the following can be observed:

- ✓ At construction:
  - o High true grade temperature difference: The average difference for all projects was -2.3°C, which indicates that WMA production temperatures typically result in slightly less aging of asphalt binders.

- Low true grade temperature difference: The average difference for all projects was -1.3 °C, which indicates that slightly less plant-related aging of the binders occurs at lower production temperatures.

✓ 1st Inspection Cores:

- High true grade temperature difference: The average difference for all projects was -0.8°C, which indicates that WMA typically results in slightly lower high critical temperature, but this difference is less than 1°C
- Low Temperature Difference: The average difference for all projects was -1°C, which indicates that WMA sections could have a very slight improvement in low temperature cracking in the first year of service.

✓ 2nd Inspection Cores:

- High Temperature Difference: The average difference for all projects was -0.8°C, which indicates that WMA pavements have a lightly lower high critical temperature compared to HMA.
- Low Temperature Difference: The average difference for all projects was 0.2°C, which is probably insignificant in practical terms.

Overall, the high and low true grades for the WMA and HMA binders at different ages are very similar, with the largest difference at time of construction. It is also noticed that the differences obtained for the high and low true grades seem to decrease with time: -2.3, -0.8 and -0.8°C (high critical temperature differences) and -1.3, -1, and 0.2°C (low critical temperature differences) at construction, first inspection, and second inspection respectively.

Table 174 presents the true grades and performance grades of the recovered binders from cores obtained for all the mixes of each existing project. The results are as follows:

St. Louis, MO: The inspection for this project was conducted 65 months after construction. The high performance grade of the HMA recovered binder was one grade higher than the grades of the binders of the three WMA technologies: Sasobit, Evotherm and Aspha-min. The low performance grades of the HMA recovered binder and two WMA binders: Evotherm and Aspha-min were the same; WMA-Sasobit was one grade higher than the HMA recovered binder.

Iron Mountain, MI: The inspection of this project was conducted 57 months after construction. The high performance grade of the HMA recovered binder was two grades lower than the grade of the WMA-Sasobit binder, which indicates a significant increase in the WMA-Sasobit binder stiffness. The low performance grade of the HMA binder was one grade lower than the WMA-Sasobit binder.

Silverthorne, CO: This project sections were inspected 38 months after construction. The high performance grades of the recovered binders from the HMA and the two WMA mixes, Advera and Evotherm, were the same; the high binder grade of the WMA-Sasobit was one grade higher than the HMA. The low performance grades of all recovered HMA and WMA binders were the same.

Franklin, TN: This project sections were inspected 41 months after construction. For two of these sections, WMA-Advera and WMA-Evotherm, it was not possible to obtain the low performance grades due to insufficient recovered binder. The high performance grades of the HMA and two WMA binders, WMA-Advera and WMA-Astec DBG, were the same; the WMA-Evotherm grade was one grade higher than the HMA binder. The low performance grades of the binders recovered the HMA and the WMA-Astec DBG were the same.

Graham, TX: The inspection of this project's sections was conducted 30 months after construction. The performance grades of both recovered binders, HMA and WMA-Astec DBG, were the same.

George, WA: This project was inspected 60 months after construction. The high performance grade of the binder recovered from the HMA was one grade higher than the WMA-Sasobit binder; the low performance grades were the same for both binders.

In summary, the high performance grades of binders recovered from HMA and WMA were the same for many of the projects. In most cases where differences in binder grade were evident, the difference was only one grade (up or down). For the Iron Mountain, MI project, the high performance grade was two grades higher for the WMA–Sasobit binder.

In general, with the exception of the Iron Mountain, MI project, the WMA technologies do not seem to have a negative effect on the binder's low and high performance grades. For the Iron Mountain, MI project, the Sasobit additive made the binder stiffer.

Table 174 True and Performance Binder Grades at Existing Projects (1 inspection only)

Project	Mix	High Temp. Grade (°C)	Low Temp Grade (°C)	PG Grade
St. Louis, MO (65 months)	HMA	85.4	-17.0	82-16
	Sasobit	79.5	-14.8	76-10
	Evotherm	77.2	-21.9	76-16
	Aspha-min	77.8	-19.7	76-16
Iron Mountain, MI (57 months)	HMA	61.2	-35.4	58-34
	Sasobit	70.2	-29.0	70-28
Silverthorne, CO (38 months)	HMA	59.2	-32.1	58-28
	Advera	60.6	-30.7	58-28
	Sasobit	66.0	-29.0	64-28
	Evotherm	59.9	-30.9	58-28
Franklin, TN (41 Months)	HMA	84.5	-16.0	82-16
	Advera	87.0	N/A	82-N/A
	Astec DBG	82.6	-17.6	82-16
	Evotherm	91.6	N/A	88-N/A
	Sasobit	87.5	-11.6	82-10
Graham, TX (30 months)	HMA	83.2	-19.0	82-16
	Astec DBG	82.7	-19.4	82-16
George, WA (60 Months)	HMA	82.6	-26.9	82-22
	Sasobit	80.6	-27.0	76-22

MIXTURE PROPERTIES

Mix Moisture Contents

AASHTO T 329 was used to determine the moisture content of loose plant-produced mix sampled at the time of construction for the new projects. The results are shown in Table 175. It can be seen that most mixes had low moisture contents (> 0.5%). WMA mixes generally had slightly higher moisture contents than their corresponding HMA mixes, but the differences are probably not significant. WMA using water foaming process appear to have similar moisture contents to other WMA technologies.

Table 175 Field Mix Moistures at Construction from New Projects

Project Location	WMA Technologies	Sample 1	Sample 2	Average
Walla Walla, WA	HMA	0.06	0.08	0.07
	Aquablack	0.22	0.23	0.23
Centreville, VA	HMA	0.06	0.02	0.04
	Astec DBG	0.12	0.17	0.15
Baker, MT	HMA	0.20	0.15	0.18
	Evotherm DAT	0.13	0.04	0.09
Jefferson Co., FL	HMA	0.04	0.04	0.04
	Terex foam	0.04	0.05	0.05
Casa Grande, AZ	HMA	0.06	0.03	0.05
	Sasobit	0.04	0.07	0.06
Rapid River , MI	HMA	0.09	0.05	0.07
	Advera	0.01	0.06	0.04
	Evotherm 3G	0.09	0.05	0.07
Munster, IN	HMA	0.25	0.27	0.26
	Evotherm	0.45	0.49	0.47
	Gencor foam	0.44	N/A	0.44
	Heritage wax	0.53	0.51	0.52
New York, NY	HMA	0.14	0.12	0.13
	BituTech PER	0.33	0.33	0.33
	Cecabase	0.31	0.43	0.37
	SonneWarmix	0.52	0.34	0.43

Densities

Densities of WMA and HMA pavements were assessed using field cores after compaction, and cores obtained during the first and second inspections. As described in the experimental plan, cores after compaction and the second inspection were only available from the new projects with the exception of two existing projects, George, WA and Iron Mountain, MI, for which densities from field cores after compaction were also available.

Densities from Field Cores after Compaction

A summary of the statistical analysis of in-place densities of cores taken after compaction are shown in Table 176. The *p*-values indicate the probability that the variances or means are not different for HMA and WMA on each respective project. These results show that variances were not statistically different except for the New York, NY project. Results for in-place relative densities on this project had a standard deviation of as low as 1.33% for the BituTech PER WMA and as high as 4.0% for the SonneWarmix WMA.

The *t*-test and Dunnett's test *p*-values shown in Table 176 indicate that none of the densities except for the Casa Grande, AZ project (Sasobit) were statistically different between WMA sections and corresponding HMA sections. This finding is counter to the often claimed benefit that WMA will improve compaction and density levels. For the Casa Grande, AZ project, higher density was achieved for the WMA section. Figure 113 summarizes the comparison of means graphically in terms of equal, higher, or lower values using the statistical analysis presented in Table 176.

Post-construction, in-place density results were not available for HMA sections on the projects in St. Louis, MO or Graham, TX. Only average density results were reported (no replicate data) for projects in Silverthorne, CO and Franklin, TN. Therefore, statistical comparisons were not possible for these projects.

Table 176 Summary of Statistical Analyses of Post-Construction In-Place Density

Single WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (% of Gmm)	F test	Avg. (% of Gmm)	*t*-test
			p-value		*p-value*
Walla Walla, WA	HMA	0.7	0.854	94.7	0.525
	Aquablack	0.7		94.4	
Centreville, VA	HMA	1.7	0.379	89.1	0.320
	Astec DBG	1.2		89.9	
Baker, MT	HMA	1.6	0.822	91.3	0.854
	Evotherm DAT	1.7		91.2	
Jefferson Co., FL	HMA	1.1	0.991	93.0	0.117
	Terex foam	1.1		92.1	
Casa Grande, AZ	HMA	2.1	0.25	90.6	0.081
	Sasobit	1.3		92.4	
George, WA	HMA	1.6	0.226	93.6	0.810
	Sasobit	1.4		93.7	
Iron Mtn., MI	HMA	1.1	0.621	94.6	0.580
	Sasobit	0.8		94.3	

Multiple WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (% of Gmm)	Bartlett's test for equal variance	Avg. (% of Gmm)	Dunnett's test of mean versus control
			p-value		*p-value*
Rapid River, MI	HMA	1.1	0.369	94.1	
	Advera	0.6		95.0	0.154
	Evotherm 3G	0.9		94.3	0.901
Munster, IN	HMA	1.5	0.370	88.7	
	Evotherm	1.6		90.3	0.352
	Gencor foam	2.2		90.4	0.417
	Heritage wax	2.9		88.7	1.000
New York, NY	HMA	2.0	0.061	90.9	
	BituTech PER	1.3		92.4	0.551
	Cecabase	2.1		92.2	0.669
	SonneWarmix	4.0		89.9	0.830

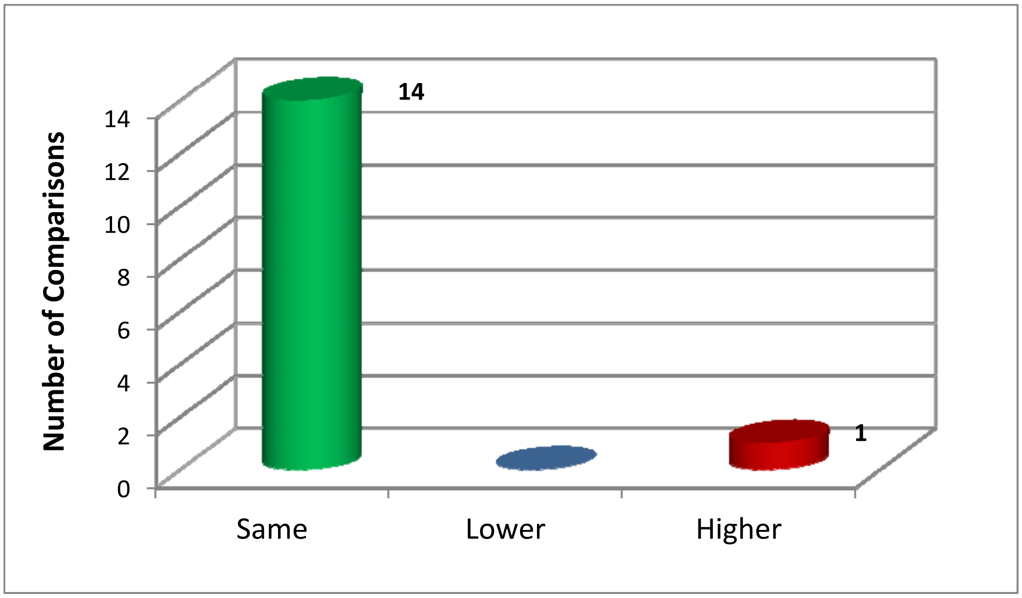


Figure 113 Comparison of WMA vs. HMA Post Construction Densities

Densities of Cores from the First Inspection (~1year)

A summary of analysis of densities of cores taken after approximately 1 year for the new projects is presented in Table 177. Three projects had statistical differences when variances were compared. These projects are Centreville, VA, Casa Grande, AZ and New York, NY. The *t*-test and Dunnett's test *p*-values show that the in place densities for the WMA mixes from Walla Walla, WA, Centreville, VA, Jefferson Co., FL and Rapid River, MI were different than their respective HMA mixes. For these four projects, the WMA densities were lower than for the corresponding HMA. Comparisons of WMA and HMA mixes in terms of equal, higher, or lower densities after about one year are presented in Figure 114 using the statistical analysis presented in Table 177. This comparison indicates that about 40 percent of the WMA sections had lower densities than their corresponding HMA sections after one year. The differences in the in-place densities after trafficking may be due to the HMA and WMA sections being placed in different lanes for some projects.

Table 177 Summary of Statistical Analyses of Densities of Cores from First Inspection (New Projects)

Single WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (% of Gmm)	F test	Avg. (% of Gmm)	*t*-test
			p-value		*p-value*
Walla Walla, WA	HMA	0.4	0.840	95.9	0.003
	Aquablack	0.4		95.2	
Centreville, VA	HMA	0.4	0.049	94.4	0.055
	Astec DBG	1.0		93.5	
Baker, MT	HMA	0.5	0.106	93.6	0.263
	Evotherm DAT	0.9		94.0	
Jefferson Co., FL	HMA	0.6	0.649	92.6	0.026
	Terex foam	0.5		91.8	
Casa Grande, AZ	HMA	1.4	0.046	93.8	0.174
	Sasobit	0.6		94.5	
Multiple WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (% of Gmm)	Bartlett's test for equal variance	Avg. (% of Gmm)	Dunnett's test of mean versus control
			p-value		*p-value*
Rapid River, MI	HMA	0.4	0.089	97.6	
	Advera	0.7		96.5	0.002
	Evotherm 3G	0.3		96.9	0.037
Munster, IN	HMA	1.7	0.122	92.9	
	Evotherm	0.7		93.0	0.990
	Gencor foam	0.9		93.0	0.990
	Heritage wax	0.5		92.9	0.999
New York, NY	HMA	1.4	0.012	93.9	
	BituTech PER	1.2		94.4	0.979
	Cecabase	2.2		93.4	0.962
	SonneWarmix	4.1		92.3	0.489

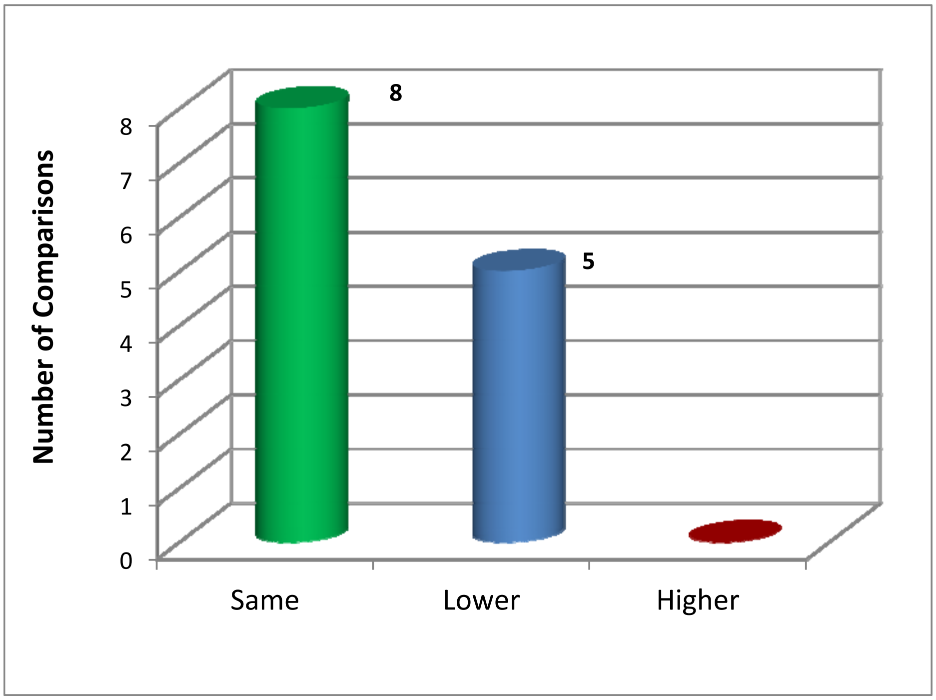


Figure 114 Comparison of WMA vs. HMA Densities –First Inspection

Densities of Cores from the Second Inspection (2-2.5 Years)

A summary of the statistical analysis of densities of cores taken after approximately 2 to 2.5 years is presented in Table 178. The majority of the results presented in these tables correspond to cores obtained in the second inspections of the new projects. Two projects had statistical differences when variances were compared: Graham, TX and Rapid Rivers, MI. The *t*-test and Dunnett's test *p*-values show that the in place densities for the Walla, Walla, WA, Graham, TX and Silverthorne, CO (Advera and Sasobit) were different than their respective HMA sections. For two of these projects (Walla Walla, WA and Graham, TX) the WMA sections had statistically lower densities. On the other hand, the results for Silverthorne, CO show that the Advera and Sasobit had statistically higher densities compared to the control mix. Figure 115 presents the results in Table 178 in terms of statistically equal, higher of lower density results.

Table 178 Summary of Statistical Analyses of Densities for Cores 2-2.5 Years Old

Single WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (% of Gmm)	F test *p-value*	Avg. (% of Gmm)	*t*-test *p-value*
Walla Walla, WA	HMA	0.4	0.239	96.3	0.007
	Aquablack	0.2		95.7	
Centreville, VA	HMA	0.7	0.636	93.8	0.402
	Astec DBG	0.9		93.4	
Baker, MT	HMA	0.9	0.670	93.7	0.409
	Evotherm DAT	1.1		93.3	
Jefferson Co., FL	HMA	1.1	0.987	91.5	0.612
	Terex foam	1.1		91.8	
Graham, TX	HMA	1.0	0.004	96.0	0.001
	Astec DBG	0.2		94.3	

Multiple WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (% of Gmm)	Bartlett's test for equal variance *p-value*	Avg. (% of Gmm)	Dunnett's test of mean versus control *p-value*
Rapid River, MI	HMA	1.0	0.083	96.6	
	Advera	0.4		97.0	0.496
	Evotherm 3G	0.5		96.0	0.244
Munster, IN	HMA	1.7	0.153	93.5	
	Evotherm	0.7		93.3	0.967
	Gencor foam	0.7		93.5	1.000
	Heritage wax	0.6		93.2	0.950
New York, NY	HMA	1.1	0.369	94.8	
	BituTech PER	1.0		95.5	0.709
	Cecabase	0.8		94.7	0.965
	SonneWarmix	1.2		94.6	0.995
Silverthorne, CO	HMA	0.2	0.500	97.0	
	Advera	0.3		97.8	0.001
	Evotherm DAT	0.3		97.2	0.375
	Sasobit	0.3		97.5	0.018

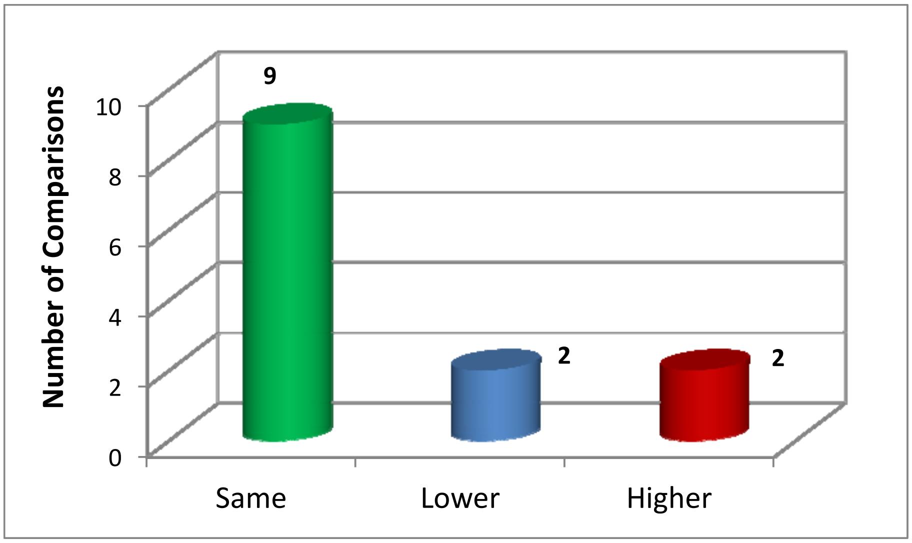


Figure 115 Comparison of WMA vs. HMA Densities –Cores 2 to 2.5 years

Densities for Projects more than 3 Years Old

A summary of the statistical analysis of densities from cores more than 3 years old is presented in Table 179. All results presented in this table correspond to "existing" projects. Only mixes from Silverthorne, CO were statistically different when variances were compared. The *t*-test and Dunnett's test *p*-values show that the in-place densities were statistically different for Iron Mtn., MI, George, WA, St. Louis, MO (Sasobit only), and Silverthorne, CO sections (Advera and Sasobit only). For the Iron Mtn., MI, St. Louis MO, and Silverthorne, CO (Sasobit only), the densities of the WMA sections were statistically lower than the companion control HMA. For George, WA, and Silverthorne, CO (Advera), the WMA section densities were statistically higher than the companion control HMA. These results are also presented in Figure 116.

Table 179 Summary of Statistical Analyses of Densities for Cores more than 3 Years Old

Single WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (% of Gmm)	F test *p-value*	Avg. (% of Gmm)	*t*-test *p-value*
Iron Mtn.., MI	HMA	0.2	0.429	97.3	0.000
	Sasobit	0.3		95.5	
George, WA	HMA	0.5	0.476	95.7	0.042
	Sasobit	0.6		96.3	

Multiple WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (% of Gmm)	Bartlett's test for equal variance *p-value*	Avg. (% of Gmm)	Dunnett's test of mean versus control *p-value*
St. Louis, MO	HMA	0.9	0.325	95.6	
	Aspha-min	1.5		95.3	0.920
	Evotherm ET	1.2		96.4	0.340
	Sasobit	0.8		94.1	0.038
Silverthorne, CO	HMA	0.6	0.028	97.3	
	Advera	0.3		98.1	0.008
	Evotherm DAT	0.2		97.0	0.278
	Sasobit	0.5		96.5	0.005
Franklin, TN2	HMA	1.9	0.389	88.9	
	Astec DBG	1.9		88.9	1.000
	Evotherm DAT	1.1		88.0	0.557

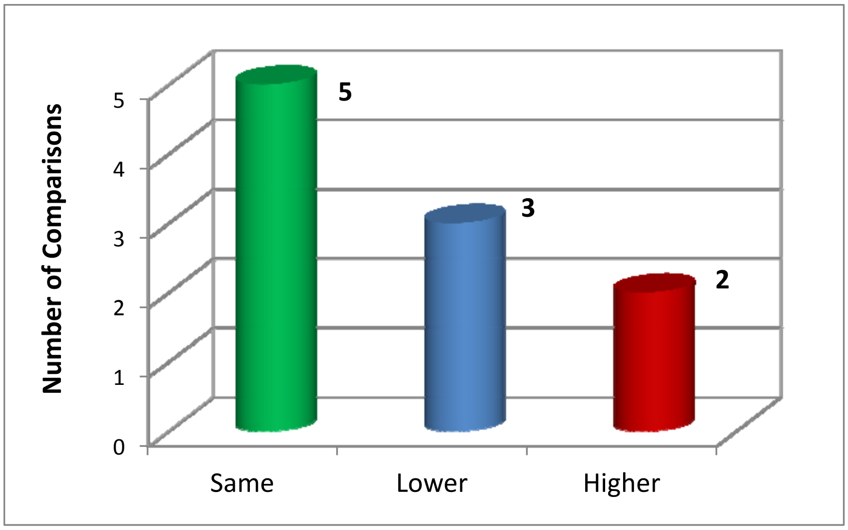


Figure 116 Comparison of WMA vs. HMA Densities –Cores more than 3 years

Binder Absorption

As part of the volumetric properties determination, the binder absorption was calculated for the plant produced mixes, and for mixtures from one and two year cores. The plant produced mixes were sampled and tested without reheating. Table 180 summarizes the asphalt absorption results for all the new projects.

Table 180 Binder Absorption for the Plant Mix, One and Two Year Cores

Project	WMA Technology	Binder Absorption (%)					
		Plant Mix		1-Year Cores		2-Years Cores	
		WMA	HMA	WMA	HMA	WMA	HMA
Walla Walla, WA	Aquablack foam	0.63	1.15	1.40	1.40	1.28	1.03
Centreville, VA	Astec DBG	0.92	0.88	0.91	0.61	0.61	0.78
Rapid River, MI	Evotherm 3G	0.66	0.59	1.01	0.88	0.91	0.78
	Advera	0.73	0.59	1.04	0.88	0.97	0.78
Baker, MT	Evotherm DAT	0.65	0.72	0.75	0.87	0.72	0.53
Munster, IN	Heritage Wax	1.51	1.58	1.26	1.29	1.49	1.55
	Gencor Foam	1.18	1.58	1.48	1.29	1.48	1.55
	Evotherm 3G	1.27	1.58	1.39	1.29	1.53	1.55
New York, NY	BituTech PER	0.77	0.75	0.50	0.70	0.75	0.71
	Cecabase	0.55	0.75	0.67	0.70	0.68	0.71
	SonneWarmix	0.61	0.75	0.71	0.70	0.66	0.71
Jefferson Co., FL	Terex CMI Foam	0.74	0.76	0.84	0.77	0.77	0.77
Casa Grande, AZ	Sasobit	0.62	0.64	0.27	0.51	NA	NA
Average Difference (WMA-HMA)		-0.12		0.03		0.03	
Difference Range		(-0.52, 0.07)		(-0.24, 0.3)		(-0.17, 0.25)	

For the plant produced mixes, binder absorptions of WMA averaged 0.12% less than for comparable HMA produced with the same aggregate blend. The differences in absorptions ranged from 0.07% greater to 0.52% less. Further analysis of the differences in asphalt absorption between WMA and HMA did not indicate that mix production temperature had a clear effect. It is likely that differences in asphalt absorption would be affected by interactions of storage time, temperature, aggregate characteristics, and binder properties.

For the one-year cores, binder absorption averaged 0.03% higher for WMA compared to HMA. The differences in calculated asphalt absorption ranged from 0.3% higher to 0.24% lower, and seven of the thirteen comparisons differed by more than 0.1%.

For the two-year cores, the average asphalt absorption difference was also 0.03%. The differences between WMA and HMA absorptions ranged from 0.25% higher to 0.17% lower. The differences in absorptions exceeded 0.1% in five of the twelve comparisons.

Since there are no replicates for binder absorption, comparison for WMA and HMA results were made using paired *t*-tests for all projects. For the field mix cores, the *p*-value is 0.041, which indicates that binder absorption of HMA and WMA is statistically different. On the other hand, for the 1- and 2-years cores, the *p*-values were 0.554 and 0.387, which indicates that their absorption values are not different.

Overall, for some mixes, there is less asphalt absorption for WMA compared to HMA for samples taken at production. However, there is no strong evidence that the asphalt absorption difference is practically significant over time. None of the mixes that had differences in absorption values of more than 0.1% at the time of construction also had similar differences after one year or two years. This suggests that the binder content of WMA mixes should not be reduced to account for reduced absorption.

Dynamic Modulus

Dynamic Modulus (E*) testing was performed in order to quantify the stiffness of the asphalt mixtures over a wide range of temperatures and frequencies. The E* tests were conducted on the field-produced mixes using an IPC Global Asphalt Mixture Performance Tester (AMPT) with a confining pressure of 20 psi. The E* samples were prepared in accordance with AASHTO PP 60-09. Triplicate samples were tested from each mix. The temperatures and frequencies used for testing these mixes were those recommended in AASHTO PP 61-10. For this methodology, the high test temperature is dependent on the high performance grade of the base binder utilized in the mix being tested. Table 181 shows the temperatures and frequencies used, and Table 182 shows the selection criteria for the high testing temperature. Samples were compacted hot in the field for the projects in Munster, IN, Jefferson Co., FL, New York, NY and Casa Grande, AZ. The samples for the other four projects were compacted in NCAT's main laboratory from reheated mix.

Table 181 Temperatures and Frequencies used for Dynamic Modulus Testing

Test Temperature (^oC)	Loading Frequencies (Hz)
4.0	10, 1, 0.1
20.0	10, 1, 0.1
High Testing Temperature	10, 1, 0.1, 0.01

Table 182 High Test Temperature for Dynamic Modulus Testing

High Performance Grade of Base Binder	High Test Temperature (^oC)
PG 58-XX and lower	35
PG 64-XX and PG 70-XX	40
PG 76-XX and higher	45

Master Curves

Data analysis for the E* tests were conducted per the methodology in AASHTO PP 61-10. Dynamic modulus master curves were generated for each of the mixes by project (WMA technologies and HMA control). The reference temperature for the master curves was 70°F (21.1°C). Figure 117 through Figure 124 present the master curves for each project on a logarithmic scale.

The three projects that appear to have differences in E* mastercurves for the HMA and WMA were Walla, Walla, WA, Baker, MT and New York, NY. E* mastercurves for the other projects appear to be very similar for HMA and WMA.

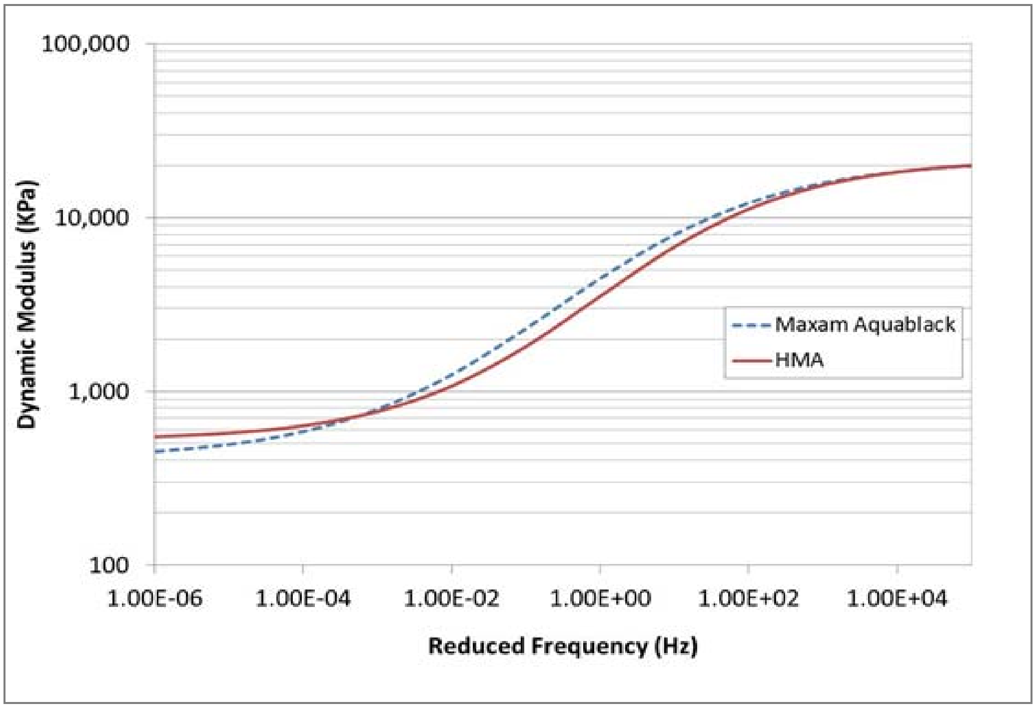


Figure 117 Dynamic Modulus Master Curves for Walla Walla, WA

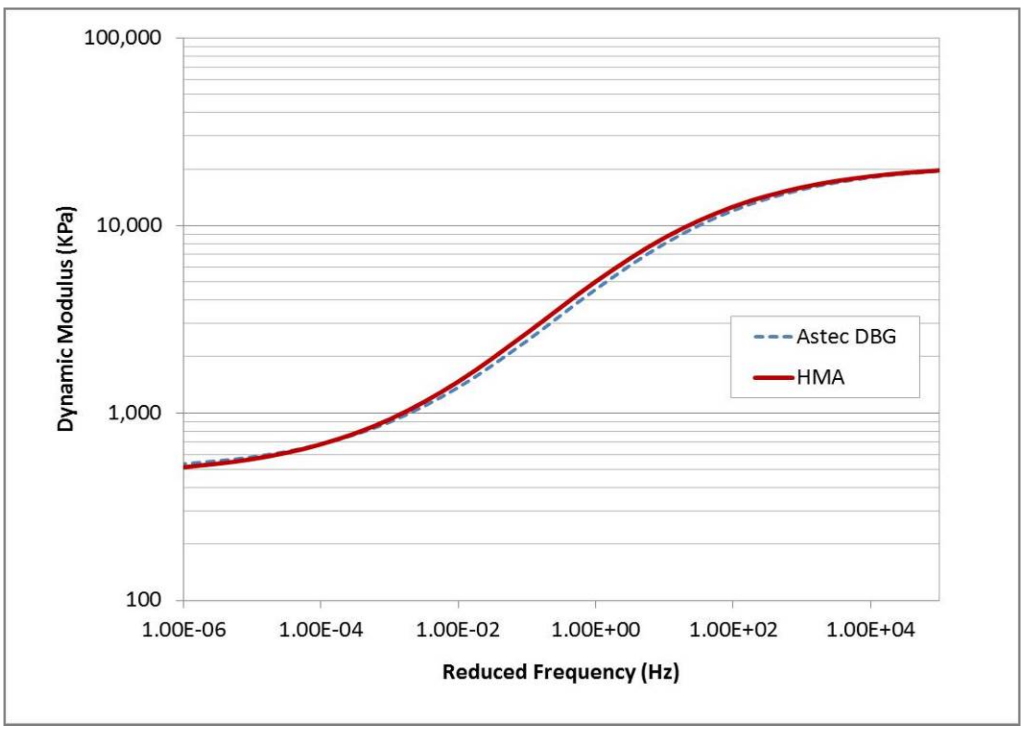


Figure 118 Dynamic Modulus Master Curves for Centreville, VA

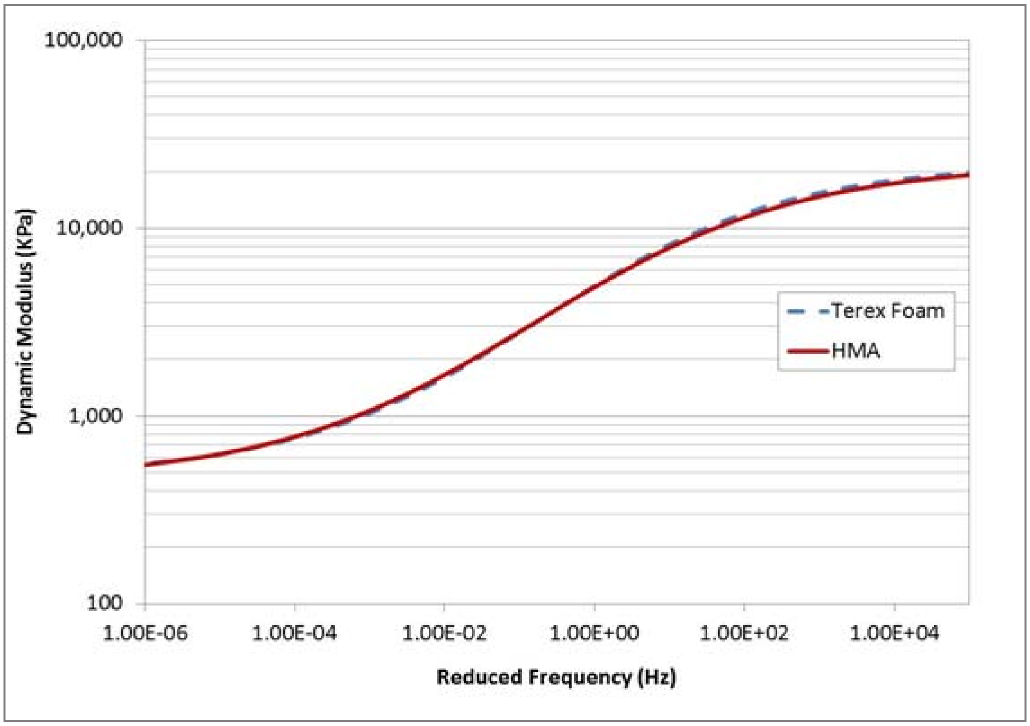


Figure 119 Dynamic Modulus Master Curves for Jefferson Co., FL

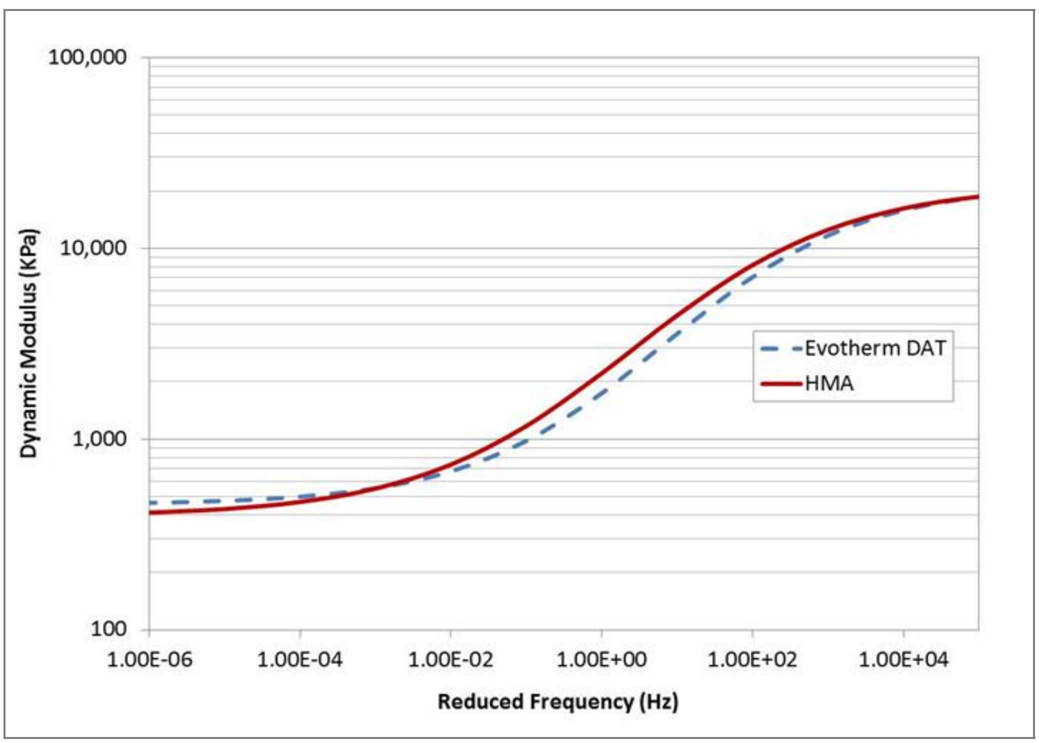


Figure 120 Dynamic Modulus Master Curves for Baker, MT

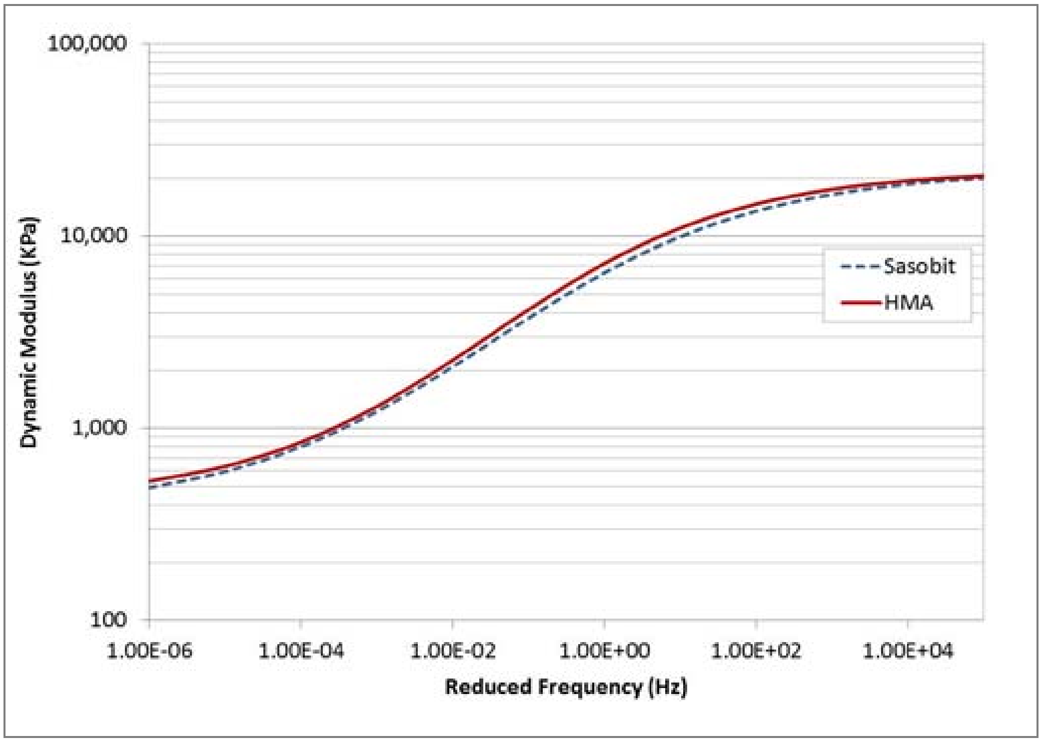


Figure 121 Dynamic Modulus Master Curves Casa Grande, AZ

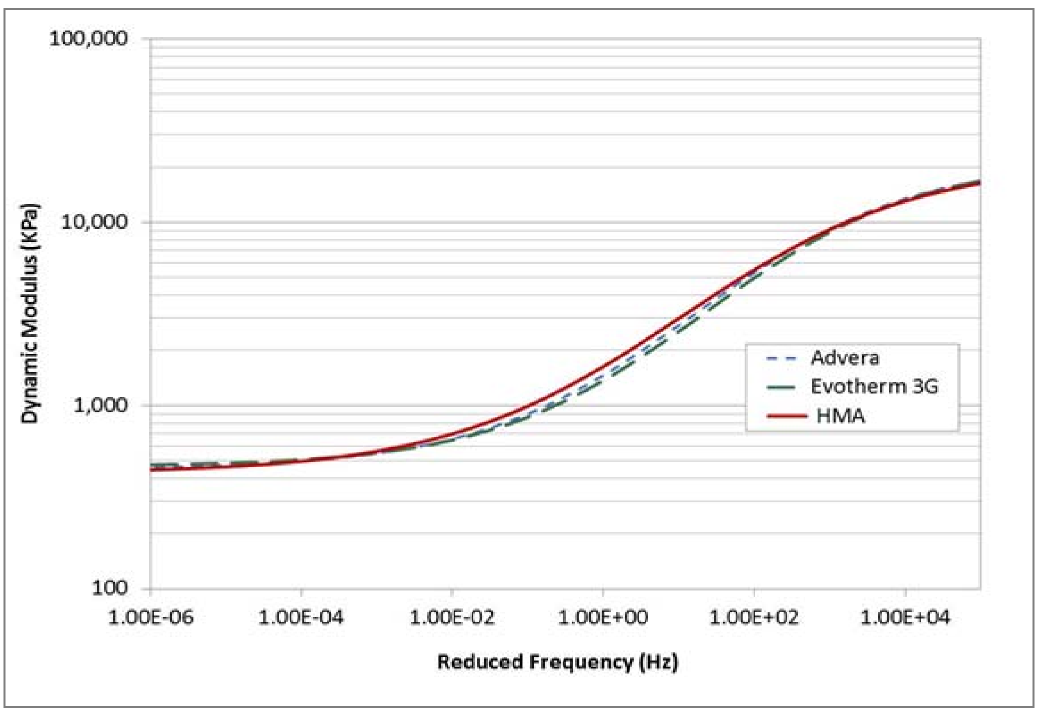


Figure 122 Dynamic Modulus Master Curves for Rapid River, MI

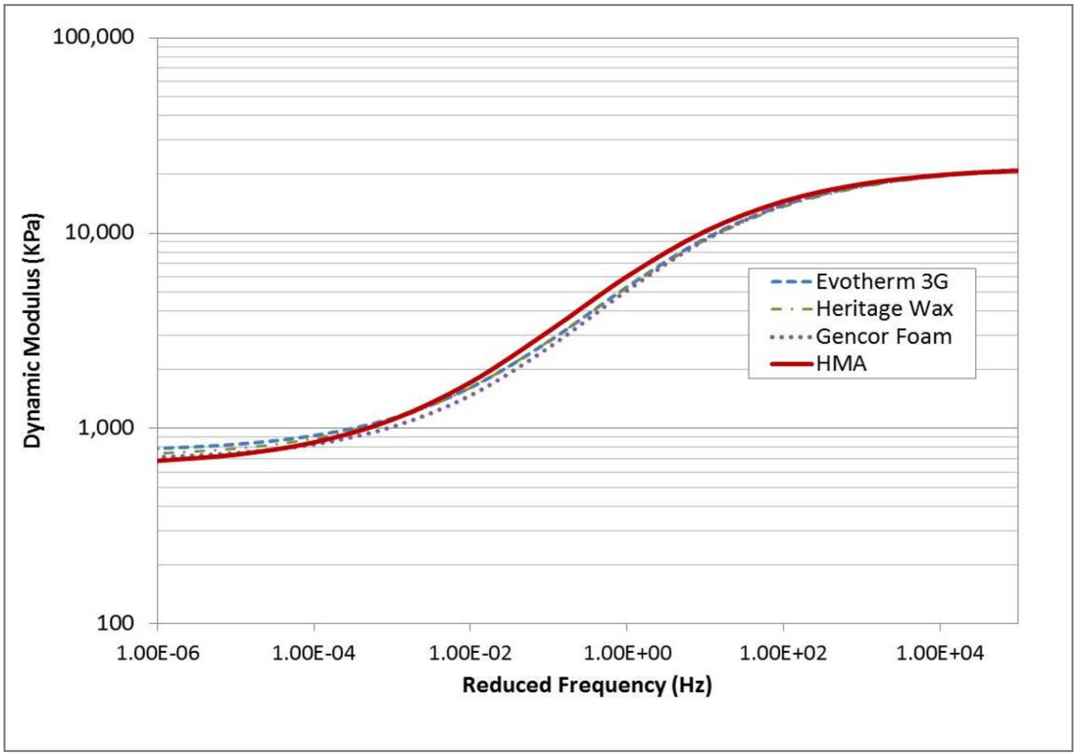


Figure 123 Dynamic Modulus Master Curves for Munster, IN

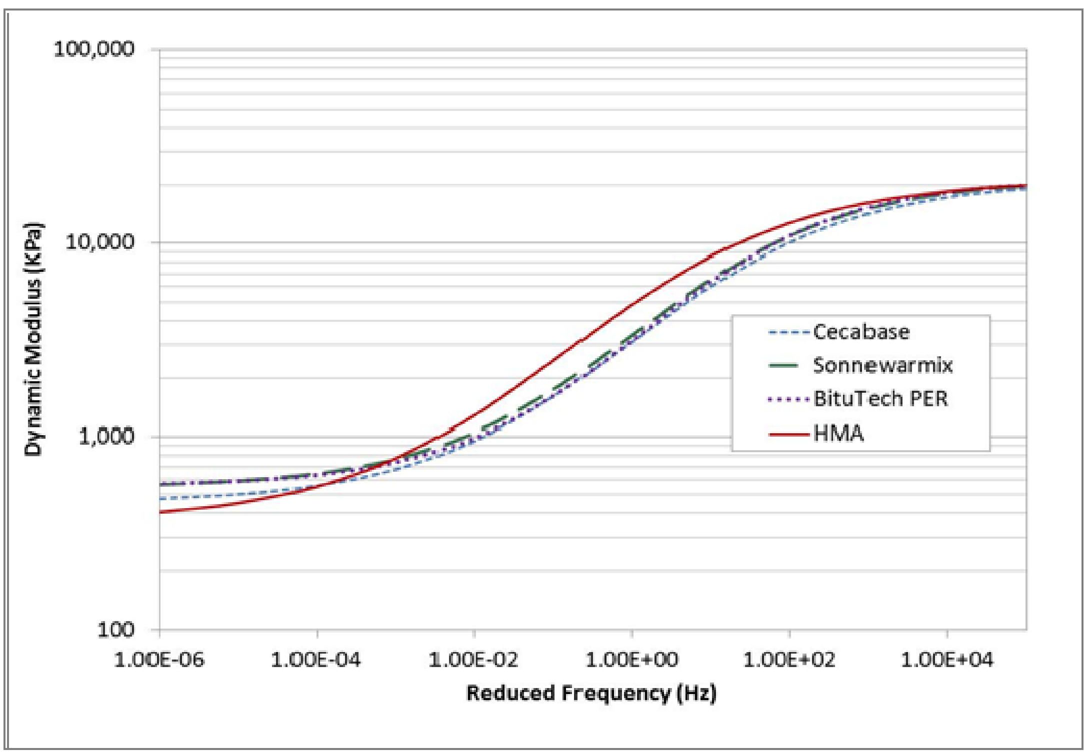


Figure 124 Dynamic Modulus Master Curves for New York, NY

Statistical Comparisons

In order to establish if there was actually a statistical difference in E* between HMA and WMA mixes on each project, two sample t-test analyses were conducted using a 90% confidence interval.

The first analysis was conducted by pooling together all data (all frequencies and temperatures) for each WMA technology compared to the control mix. Table 183 shows the results of this statistical analysis. There was a statistically significant difference in E* between the HMA and WMA mixes for the following projects:

- Centreville, VA (Astec DBG)
- Walla, Walla, WA (Aquablack)
- Baker, MT (Evotherm DAT)
- New York, NY (BituTech PER, Cecabase, SonneWarmix)

Table 183 Summary of Statistical Analyses of Dynamic Modulus Test Results

Project	Additive	Dunnett's test of mean versus control	Difference Statistically Significant? (Y or N)
		p-value	
Centreville, VA	Astec DBG	0.0784	Y
Walla Walla, WA	Aquablack	0.0048	Y
Jefferson Co., FL	Terex	0.9863	N
Baker, MT	Evotherm DAT	0.0604	Y
Casa Grande, AZ	Sasobit	0.6270	N
Rapid River, MI	Advera	0.8757	N
	Evotherm	0.1687	N
Munster, IN	Evotherm	0.4529	N
	Gencor Foam	0.5306	N
	Heritage Wax	0.5801	N
New York, NY	BituTech PER	0.0056	Y
	Cecabase	0.0005	Y
	SonneWarmix	0.0377	Y

A second t-test analysis was conducted specifically at frequencies of 0.1, 1 and 10 Hz. Table 184 through Table 186 show the results of the statistical analyses of E* at 0.1, 1 and 10 Hz, respectively.

Table 184 Summary of Statistical Analyses of Dynamic Modulus Test Results at 0.1 Hz

Project	WMA Tech.	Test Temp. (°C)	Avg. E* (MPa) WMA	Avg. E* (MPa) HMA	Std. Dev. E* (MPa) WMA	Std. Dev. E* (MPa) HMA	2-sample t-test *p*-value (α = 0.10)	Diff. Sig.? (Y/N)
Walla, Walla, WA	Aquablack	4	7,240	7,699	433	392	0.074	Y
		20	1,613	2,227	20	30	0.002	Y
		40	748	767	16	5	0.248	N
Centreville, VA	Astec DBG	4	7,887	8,694	746	297	0.188	N
		20	2,333	2,564	563	398	0.506	N
		45	767	765	29	20	0.948	N
Jefferson Co., FL	Terex Water Injection	4	8,124	8,274	163	372	0.626	N
		20	2,616	2,748	27	149	0.211	N
		45	900	823	49	18	0.14	N
Baker, MT	Evotherm DAT	4	3,247	4,460	16	243	0.015	Y
		20	857	1,074	73	106	0.017	Y
		40	561	537	17	9	0.235	N
Casa Grande, AZ	Sasobit	4	10,519	11,809	236	293	0.042	Y
		20	3,724	4,117	174	203	0.162	N
		40	1,066	1,136	69	37	0.340	N
Rapid River, MI	Advera	4	2,306	2,371	278	247	0.851	N
		20	855	956	33	56	0.189	N
		35	599	639	24	18	0.234	N
		45	543	566	24	20	0.448	N
	Evotherm	4	2,031	2,371	136	247	0.252	N
		20	837	956	61	56	0.218	N
		35	601	639	26	18	0.278	N
		45	557	566	23	20	0.709	N
Munster, IN	Evotherm	4	8,587	9,671	258	558	0.088	Y
		20	2,779	3,141	44	106	0.011	Y
		40	1,109	1,058	20	29	0.098	N
	Gencor Foam	4	8,903	9,671	297	558	0.115	N
		20	2,615	3,141	243	106	0.025	Y
		40	939	1,058	87	29	0.148	N
	Heritage Wax	4	8,947	9,671	141	558	0.116	N
		20	2,814	3,141	89	106	0.086	Y
		40	1,041	1,058	35	29	0.681	N
New York, NY	BituTech PER	4	6,356	8,241	302	424	0.029	Y
		20	1,435	2,385	124	319	0.055	Y
		40	725	754	29	36	0.208	N
	Cecabase	4	5,970	8,241	309	424	0.006	Y
		20	1,490	2,385	34	319	0.032	Y
		40	653	736	42	12	0.105	N
	Sonne-Warmix	4	7,071	8,241	203	424	0.081	N
		20	1,561	2,385	16	319	0.051	Y
		40	736	754	12	36	0.556	N

Table 185 Summary of Statistical Analyses of Dynamic Modulus Test Results at 1 Hz

Project	WMA Tech.	Test Temp. (°C)	Avg. E* (MPa) WMA	Avg. E* (MPa) HMA	Std. Dev. E* (MPa) WMA	Std. Dev. E* (MPa) HMA	2-sample t-test *p*-value (α = 0.10)	Diff. Sig.? (Y/N)
Walla, Walla, WA	Aquablack	4	10,908	11,306	645	564	0.169	N
		20	3,204	4,378	29	49	0.001	Y
		40	1,087	1,231	15	74	0.079	Y
Centreville, VA	Astec DBG	4	11,560	12,237	1,098	235	0.386	N
		20	4,345	4,763	353	448	0.359	N
		45	1,106	1,161	68	43	0.192	N
Jefferson Co., FL	Terex Water Injection	4	11,453	11,433	141	407	0.946	N
		20	4,580	4,716	61	206	0.247	N
		45	1,359	1,192	76	41	0.116	N
Casa Grande, AZ	Sasobit	4	13,410	15,014	475	275	0.061	Y
		20	6,299	6,859	191	267	0.140	N
		40	1,833	1,992	167	63	0.317	N
Baker, MT	Evotherm DAT	4	6,046	7,614	90	395	0.026	Y
		20	1,662	2,173	93	241	0.037	N
		40	700	710	25	21	0.531	N
Rapid River, MI	Advera	4	4,364	4,297	493	440	0.912	N
		20	1,444	1,659	49	101	0.131	N
		35	744	828	32	33	0.154	N
		45	622	672	32	36	0.33	N
	Evotherm	4	3,921	4,297	192	440	0.396	N
		20	1,343	1,659	127	101	0.135	N
		35	733	828	40	33	0.144	N
		45	630	672	34	36	0.359	N
Munster, IN	Evotherm	4	12,702	13,786	445	691	0.125	N
		20	5,384	5,787	130	175	0.011	Y
		40	1,581	1,739	84	61	0.196	N
	Gencor Foam	4	13,233	13,786	276	691	0.239	N
		20	5,066	5,787	405	175	0.033	Y
		40	1,365	1,739	123	61	0.053	Y
	Heritage Wax	4	13,027	13,786	263	691	0.946	N
		20	5,400	5,787	138	175	0.247	N
		40	1,466	1,739	84	61	0.116	N
New York, NY	BituTech PER	4	10,119	11,960	397	473	0.129	N
		20	3,029	4,604	155	486	0.04	Y
		40	977	1,316	53	111	0.033	Y
	Cecabase	4	9,400	11,960	321	473	0.009	Y
		20	3,050	4,604	42	486	0.026	Y
		40	914	1,316	49	111	0.009	Y
	Sonne-Warmix	4	10,786	11,960	340	473	0.046	Y
		20	3,148	4,604	37	486	0.037	Y
		40	1,014	1,316	31	111	0.033	Y

Table 186 Summary of Statistical Analyses of Dynamic Modulus Test Results at 10 Hz

Project	Additive	Test Temp. (°C)	Avg. E* (MPa) WMA	Avg. E* (MPa) HMA	Std. Dev. E* (MPa) WMA	Std. Dev. E* (MPa) HMA	2-sample t-test *p*-value (α = 0.10)	Diff. Sig.? (Y/N)
Walla, Walla, WA	Aquablack	4	14,799	15,156	835	695	0.65	N
		20	5,972	6,988	61	855	0.16	N
		40	2,065	2,430	31	319	0.17	N
Centreville, VA	Astec DBG	4	15,509	15,945	1,441	144	0.641	N
		20	7,355	7,863	593	517	0.396	N
		40	1,979	2,142	163	147	0.13	N
Jefferson Co., FL	Terex Water Injection	4	14,988	14,790	73	431	0.51	N
		20	7,471	7,504	111	265	0.77	N
		45	2,404	2,043	143	93	0.11	N
Baker, MT	Evotherm DAT	4	9,801	11,409	197	595	0.056	Y
		20	3,600	4,437	140	391	0.038	Y
		40	1,090	1,186	42	91	0.158	N
Casa Grande, AZ	Sasobit	4	16,239	18,157	709	397	0.095	N
		20	9,422	10,207	210	312	0.111	N
		40	3,583	3,842	270	95	0.279	N
Rapid River, MI	Advera	4	7,539	7,237	673	689	0.618	즰
		20	2,881	3,178	92	172	0.165	N
		35	1,158	1,320	62	66	0.117	N
		45	852	948	50	82	0.273	N
	Evotherm	4	6,959	7,237	188	689	0.738	N
		20	2,669	3,178	247	172	0.191	N
		35	1,122	1,320	76	66	0.159	N
		45	845	948	60	82	0.329	N
Munster, IN	Evotherm	4	17,011	17,983	699	787	0.201	N
		20	9,141	9,482	302	246	0.096	N
		40	2,921	3,394	280	188	0.222	N
	Gencor Foam	4	17,648	17,983	372	787	0.505	N
		20	8,887	9,482	655	246	0.137	N
		40	2,802	3,394	570	188	0.173	N
	Heritage Wax	4	17,278	17,983	346	787	0.111	N
		20	9,006	17,983	108	787	0.003	Y
		40	2,655	3,394	204	188	0.028	Y
New York, NY	BituTech PER	4	14,198	15,816	458	525	0.014	Y
		20	5,877	7,815	220	574	0.025	Y
		40	1,765	2,719	118	243	0.009	Y
	Cecabase	4	13,183	15,816	278	525	0.014	Y
		20	5,789	7,815	62	574	0.025	Y
		40	1,645	2,719	65	243	0.009	Y
	Sonne-Warmix	4	14,645	15,816	484	525	0.182	N
		20	5,943	7,815	83	574	0.036	Y
		40	1,802	2,719	66	243	0.012	Y

Table 184 shows that there were significant differences between the HMA and WMA mixes at 0.1 Hz for the following projects:

- Walla, Walla, WA: Aquablack (4, 20°C)
- Baker, MT: Evotherm DAT (4, 20°C)
- Munster, IN: Evotherm (4, 20°C), Gencor Foam (20°C), Heritage Wax (20°C)
- Casa Grande, AZ: Sasobit (4°C)
- New York, NY: BituTech PER (4, 20°C), Cecabase (4, 20°C), SonneWarmix (20°C)

Table 185 shows that there were significant differences between the HMA and WMA mixes at 1 Hz for the following projects:

- Walla, Walla: Aquablack (20, 40°C)
- Casa Grande, AZ: Sasobit (4°C)
- Baker, MT: Evotherm DAT (4°C)
- Munster, IN: Evotherm (20°C), Foam (20, 40°C)
- New York, NY: BituTech PER (20, 40°C), Cecabase (4, 20, 40°C), SonneWarmix (4, 20, 40°C)

Similarly, Table 186 shows that there were significant differences between the HMA and WMA mixes at 10 Hz for the following projects:

- Baker, MT: Evotherm (4, 20°C)
- Munster, IN: Heritage Wax (4, 20°C)
- New York, NY: BituTech PER (4, 20, 40°C), Cecabase (4, 20, 40°C), SonneWarmix (20, 40°C)

For all cases where significant differences were found, the WMA had lower E* than the corresponding HMA mix. The evaluation by frequencies agrees with the overall analysis for the Walla Walla, WA and New York, NY projects. For Munster, IN, Baker, MT and Casa Grande, AZ, the analyses by frequencies show that the differences are specific to certain temperatures and frequencies.

Flow Number

Specimens for flow number test were compacted either in the field ("Hot Samples") or in the laboratory ("Reheated Samples"), in accordance with AASHTO PP 30. Two sets, three specimens per set, of flow number tests were conducted. The first set was tested unconfined in accordance with the recommendations from NCHRP Project 9-43. A deviator stress of 87 psi was used for the unconfined specimens. The second set was tested confined with a confining pressure of 10 psi. A deviator stress of 100 psi was used for confined testing.

Table 187 shows the results of the statistical analysis for the unconfined flow number tests for hot and reheated samples. Variances of the unconfined flow number results were significantly different for all projects, except for the hot samples from Walla Walla, WA and Casa Grande, AZ, and reheated samples from Walla Walla, WA and Jefferson Co., FL. For mixes compacted hot, variances of the HMA mixes were higher than for the corresponding WMA. HMA mixes had higher unconfined flow number results than WMA for the following projects:

- Walla, Walla (Reheated)
- Centreville (Reheated),
- Jefferson (Hot)
- Rapid River (Reheated, both WMA technologies),
- Munster (Hot, all three WMA technologies), and
- New York (Hot, all three WMA technologies).

For the other projects, the differences between HMA and WMA flow number results were not significant at $\alpha = 0.1$. However, except for the Casa Grande projects, the *p*-values for the t-tests comparing the flow number results were fairly low (0.118-0.146), indicating that the WMA mixes have a greater susceptibility to deformation compared to HMA. Figure 125 summarizes the results presented in Table 187.

Table 188 shows the results of the confined flow number tests. All confined flow number tests ran 20,000 cycles before being terminated by the software. Since tertiary flow was not achieved for any of the mixes, the accumulated microstrain at 20,000 cycles was used as the parameter to evaluate the relative deformation resistance. For all the projects except for New York, the variances were not statistically different. However, the statistical analysis indicates that there was a difference in mean accumulated microstrain between the WMA and corresponding HMA mix for nine of fourteen mixes compared. For these nine comparisons, the average accumulated microstrain for the WMA mixes was higher than the corresponding HMA. The remaining comparisons between WMA and HMA mixes that were not statistically different were:

- Walla, Walla, WA (Reheated)
- Baker, MT (Reheated),
- Casa Grande, AZ (Hot)
- George, WA (Reheated), and
- Munster, IN (Hot, Evotherm 3G only).

Considering the combined unconfined and confined flow number test results, most WMA mixes were less resistant to rutting than their corresponding HMA mixes. Although there are a few cases where flow number results were similar for WMA and HMA, the finding that these laboratory tests generally indicate that WMA mixes have a greater rutting potential compared to HMA is consistent with other laboratory studies.

Table 187 Summary of Statistical Analyses of Unconfined Flow Number Results

Single WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (cycles)	F test; *p*-value	Avg. (cycles)	*t*-test; *p*-value
Walla Walla, WA (Reheated)	HMA	111	0.025	426	0.090
	Aquablack	13		227	
Walla Walla, WA (Hot)	HMA	94	0.183	332	0.146
	Aquablack	30		200	
Centreville, VA (Reheated)	HMA	300	0.048	1855	0.015
	Astec DBG	47		439	
Baker, MT (Reheated)	HMA	29	0.007	98	0.140
	Evotherm DAT	2		58	
Jefferson Co., FL (Reheated)	HMA	68	0.154	231	0.124
	Terex foam	20		127	
Jefferson Co., FL (Hot)	HMA	70	0.062	414	0.024
	Terex foam	12		157	
Casa Grande, AZ (Hot)	HMA	19	0.560	61	0.367
	Sasobit	12		46	
Graham, TX (Hot)	HMA	202	0.029	570	0.118
	Astec DBG	26		259	
Multiple WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (cycles)	Bartlett's test of equal variance *p-value*	Avg. (cycles)	Dunnett's test of mean vs. control *p-value*
Rapid River, MI (Reheated)	HMA	28	0.010	199	
	Advera	1		60	0.0001
	Evotherm 3G	11		65	0.0001
Munster, IN (Hot)	HMA	217	0.000	561	
	Evotherm 3G	6		177	0.0067
	Gencor Ultrafoam	4		217	0.0123
	Heritage wax	39		314	0.0594
New York, NY (Hot)	HMA	56	0.012	291	
	Bitutech PER	12		128	0.0004
	Cecabase	3		115	0.0002
	SonneWarmix	17		123	0.0003

Table 188 Summary of Statistical Analyses of Confined Flow Number Results, Accumulated Microstrain at 20,000 Cycles

Single WMA Technology Projects

Project Location	WMA Technologies	Std. Dev. (με)	F test *p-value*	Avg. (με)	*t*-test *p-value*
Walla Walla, WA (Reheated)	HMA	2223	0.437	45,020	0.468
	Aquablack	4202		47,219	
Centreville, VA (Reheated)	HMA	1532	0.815	26,338	0.000
	Astec DBG	1848		43,379	
Baker, MT (Reheated)	HMA	13,376	0.363	60,930	0.869
	Evotherm DAT	6301		62,531	
Jefferson Co., FL (Hot)	HMA	4667	0.829	49,802	0.087
	Terex foam	3927		57,739	
Casa Grande, AZ (Hot)	HMA	7407	0.664	42,780	0.518
	Sasobit	10502		50,774	
George, WA (Reheated)	HMA	5332	0.907	22,441	0.872
	Sasobit	4954		23,051	
Multiple WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (με)	Bartlett's test for equal variance *p-value*	Avg. (με)	Dunnett's test of mean versus control *p-value*
Rapid River, MI (Reheated)	HMA	5651	0.630	41,554	
	Advera	3131		85,113	0.000
	Evotherm 3G	6855		97,706	0.000
Munster, IN (Hot)	HMA	2570	0.783	33,188	
	Evotherm 3G	1480		28,976	0.103
	Gencor Ultrafoam	1489		42,955	0.001
	Heritage wax	2748		39,710	0.015
NYC, NY (Reheated)	HMA	931	0.010	26,568	
	Bitutech PER	1995		34,397	0.067
	Cecabase	6781		67,141	0.000
	SonneWarmix	410		42,722	0.001

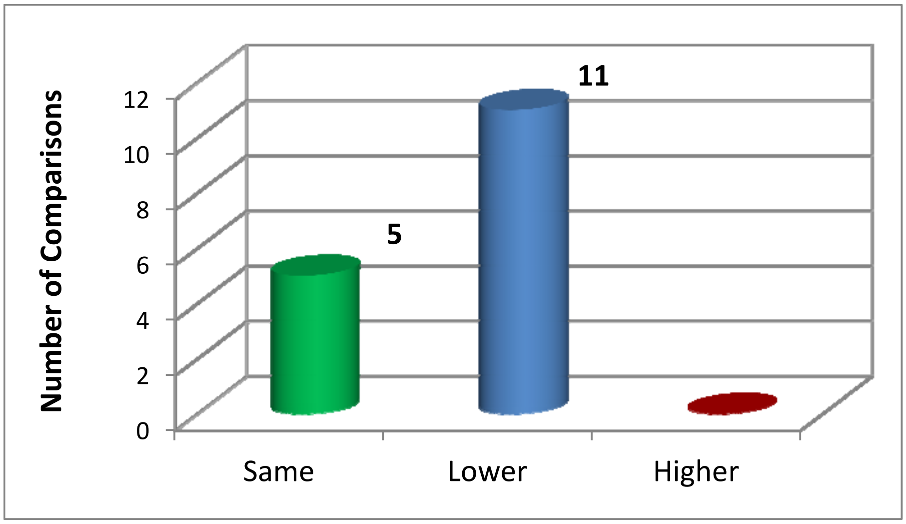


Figure 125 Comparison of WMA vs. HMA –Unconfined Flow Number

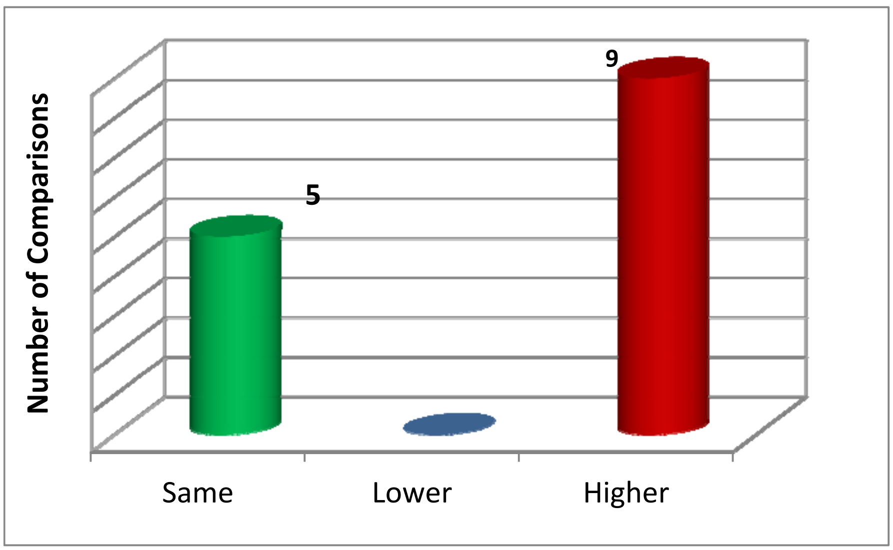


Figure 126 Comparison of WMA vs. HMA – Accumulated Microstrain at 20,000 Cycles-Confined Flow Number

Tensile Strength

Tensile Strength from Cores

Tensile strength tests were conducted on cores taken after compaction operations were completed on the projects and on cores taken during project inspections after approximately 1 and 2 years of construction for the new projects. Tensile strength tests were also conducted on laboratory-molded specimens tested as part of AASHTO T283.

Table 189 shows a summary of the statistical analysis of tensile strengths from cores taken after compaction. Except for the Casa Grande project, variances were not statistically different. Mean tensile strengths of WMA and HMA were not statistically different ($\alpha = 0.10$) except for Iron Mountain, MI and Rapid River, MI (Advera only). On the Iron Mountain project, the Sasobit section had a lower tensile strength than the HMA section. On the Rapid River project, the Advera section had a higher tensile strength than the HMA section. Overall, tensile strengths on these two projects are lower than the other projects because of the softer virgin binder used (PG 58-34) in the northern part of Michigan. The statistical analyses presented in Table 189 are summarized in Figure 127.

Table 190 shows a summary of analysis of unconditioned tensile strengths from laboratory-molded specimens tested as part of AASHTO T283. All of these specimens were molded in the NCAT mobile laboratory without reheating the mixes. The results of the statistical analysis shows that for seven of the nine projects, variances were not statistically different ($\alpha = 0.10$). The two cases which did have different variances for tensile strength results were Jefferson Co. FL, and Rapid River, MI. However, the mean tensile strengths of WMA and HMA were statistically different for all projects except for Walla Walla, WA. It can also be seen that the tensile strengths of the WMA mixes were lower than the corresponding HMA except for the New York, NY project which had had higher tensile strengths for each of the WMA mixes compared to the HMA mix. Statistically lower tensile strengths for laboratory-molded WMA compared to HMA have also been found on several other field projects by the research team. However, the contrast in findings for tensile strengths for laboratory molded samples and cores are surprising and difficult to explain. A possible reason is that the thinner field cores allow the WMA binder to cure or stiffen more between the time the specimens are obtained from the field and tested for tensile strength. Figure 128 summarizes the statistical analyses presented in Table 190.

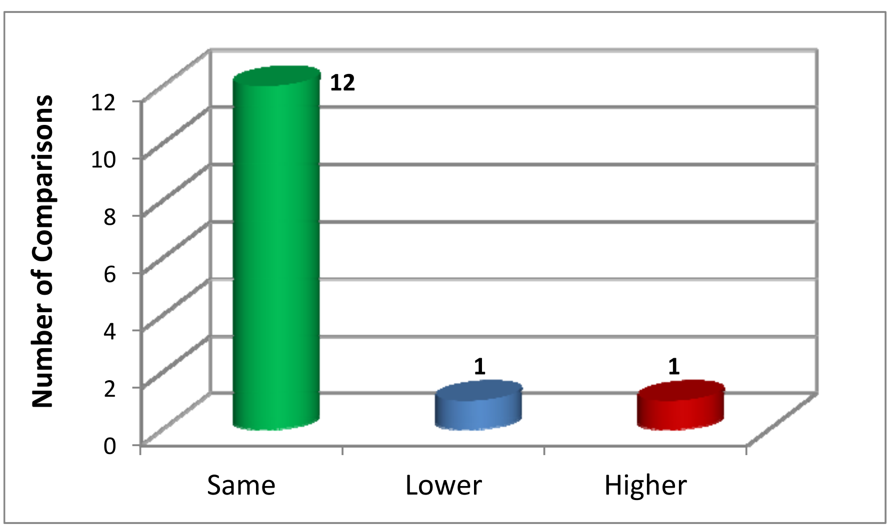


Figure 127 Comparison of WMA vs. HMA Tensile Strength –Post Construction Cores

Table 189 Summary of Statistical Analyses of Post-Construction Core Tensile Strengths

Single WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (psi)	F test *p-value*	Avg. (psi)	*t*-test *p-value*
Walla Walla, WA	HMA	10.7	0.643	161	0.474
	Aquablack	8.4		165	
Centreville, VA	HMA	10.9	0.725	132	0.578
	Astec DBG	12.8		136	
Baker, MT	HMA	7.2	0.843	68	0.646
	Evotherm DAT	7.9		65	
Casa Grande, AZ	HMA	19.0	0.050	117	0.120
	Sasobit	5.0		132	
Jefferson Co., FL	HMA	10.2	0.305	151	0.821
	Terex foam	16.7		153	
Iron Mtn., MI	HMA	3.6	0.957	52	0.014
	Sasobit	3.5		46	
Multiple WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (psi)	Bartlett's test for equal variance *p-value*	Avg. (psi)	Dunnett's test of mean versus control *p-value*
Rapid River, MI	HMA	3.8	0.931	54	
	Advera	4.4		59	0.091
	Evotherm 3G	3.7		50	0.312
Munster, IN	HMA	14.8	0.428	90	
	Evotherm	12.0		106	0.273
	Gencor foam	15.1		101	0.527
	Heritage wax	24.5		93	0.962
New York, NY	HMA	13.6	0.735	103	
	BituTech PER	10.5		99	0.914
	Cecabase	16.6		93	0.513
	SonneWarmix	17.2		92	0.402

Table 190 Summary of Statistical Analyses of Lab-Molded Specimen Tensile Strengths

Single WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (psi)	F test *p-value*	Avg. (psi)	*t*-test *p-value*
Walla Walla, WA	HMA	15.1	0.204	120	0.192
	Aquablack	5.1		102	
Centreville, VA	HMA	8.2	0.987	185	0.003
	Astec DBG	8.3		143	
Baker, MT	HMA	2.5	0.509	72	0.006
	Evotherm DAT	1.5		63	
Jefferson Co., FL	HMA	1.7	0.069	198	0.018
	Terex foam	8.8		160	
Iron Mtn., MI	HMA	3.5	0.671	55	0.003
	Sasobit	2.5		71	
Multiple WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (psi)	Bartlett's test for equal variance *p-value*	Avg. (psi)	Dunnett's test of mean versus control *p-value*
Rapid River, MI	HMA	1.7	0.093	50	
	Advera	3.0		31	0.000
	Evotherm 3G	0.8		37	0.000
Munster, IN	HMA	2.5	0.299	160	
	Evotherm	4.4		97	0.000
	Gencor foam	6.0		111	0.000
	Heritage wax	15.1		174	0.008
New York, NY	HMA	3.6	0.144	103	
	BituTech PER	2.9		107	0.000
	Cecabase	8.8		122	0.000
	SonneWarmix	9.8		115	0.000
St. Louis, MO	HMA	22.9	0.356	142	
	Evotherm ET	15.1		114	0.000
	Sasobit	15.3		106	0.000
	Aspha-Min	7.4		167	0.021

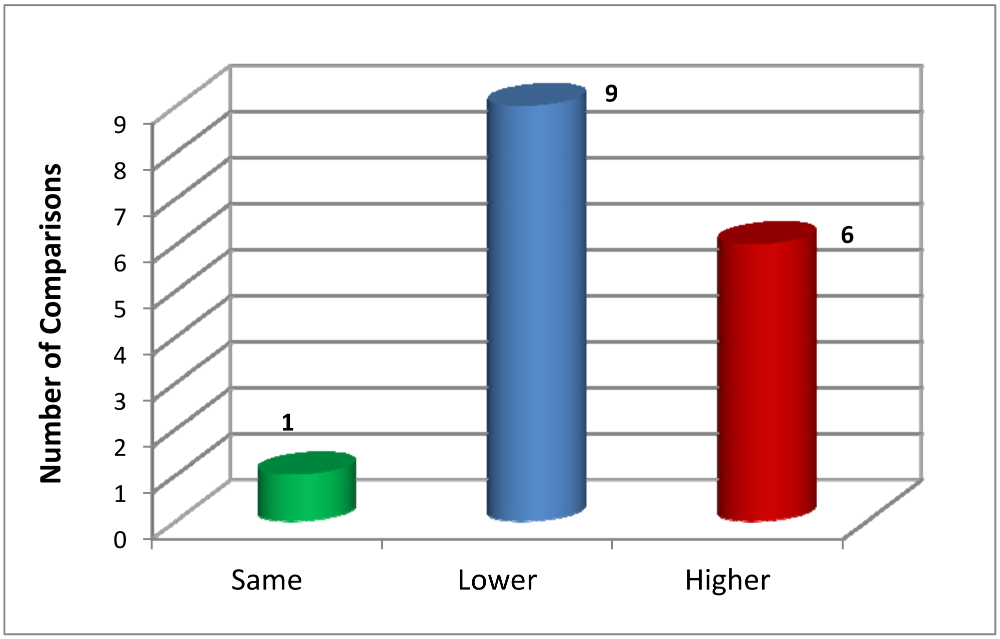


Figure 128 Comparison of WMA vs. HMA Tensile Strength –Lab Molded Samples

Table 191 shows a summary of the statistical analysis of tensile strengths from cores taken approximately one year after construction. Except for the New York, NY project, variances for WMA and HMA tensile strengths were not statistically different. Mean tensile strengths of WMA and HMA were not statistically different (α = 0.10) except for Baker, MT, Rapid River, MI (Advera only) and New York, NY (BituTech PER only). For the Baker, MT and New York, NY projects, tensile strength of the WMA sections were lower than the corresponding HMA. The Advera mix from the Rapid River, MI project had statistically higher tensile strength values than the HMA.

Table 192 provides a summary of the statistical analysis of tensile strengths from cores after 2 to 2.5 years. For four projects (Walla, Walla, WA, Baker, MT, Rapid River, MI, and New York), variances for WMA and HMA tensile strengths were statistically different. Mean tensile strengths of WMA and HMA were statistically different (α = 0.10) only for three projects: Baker, MT, Rapid River, MI (Advera only), and New York (BituTech and Cecabase). For the Baker, MT project and the New York project, the WMA cores had lower tensile strengths than the corresponding HMA cores, but the Advera mix from Rapid River had a higher tensile strength than the corresponding HMA mix.

Table 193 shows a summary of the statistical comparisons of tensile strengths from cores after at least three years. Only the George, WA project had statistically different variances for WMA and HMA. Mean tensile strengths of WMA and HMA were statistically different (α = 0.10) for only two mixes: St. Louis, MO (Sasobit only) and Franklin, TN, (Evotherm DAT only).

Both of these WMA mixes had a statistically higher tensile strength than the corresponding HMA mix.

Figure 129 through Figure 131 summarize the statistical analyses presented in Table 191 through Table 193. In these figures, “same” means that there was no statistical difference between the mean values, and lower or higher means there were differences.

Table 191 Summary of Statistical Analyses of 1 Year Cores Tensile Strengths

<table>
<tr><td colspan="6">Single WMA Technology Projects</td></tr>
<tr><td rowspan="2">Project Location</td><td rowspan="2">WMA Technologies</td><td rowspan="2">Std. Dev. (psi)</td><td>F test</td><td rowspan="2">Avg. (psi)</td><td>t-test</td></tr>
<tr><td>p-value</td><td>p-value</td></tr>
<tr><td rowspan="2">Walla Walla, WA</td><td>HMA</td><td>11.4</td><td rowspan="2">0.128</td><td>105</td><td rowspan="2">0.175</td></tr>
<tr><td>Aquablack</td><td>24.1</td><td>120</td></tr>
<tr><td rowspan="2">Centreville, VA</td><td>HMA</td><td>47.8</td><td rowspan="2">0.466</td><td>111</td><td rowspan="2">0.240</td></tr>
<tr><td>Astec DBG</td><td>33.8</td><td>142</td></tr>
<tr><td rowspan="2">Baker, MT</td><td>HMA</td><td>6.1</td><td rowspan="2">0.91</td><td>59</td><td rowspan="2">0.070</td></tr>
<tr><td>Evotherm DAT</td><td>5.8</td><td>51</td></tr>
<tr><td rowspan="2">Casa Grande, AZ</td><td>HMA</td><td>32.5</td><td rowspan="2">0.27</td><td>238</td><td rowspan="2">0.395</td></tr>
<tr><td>Sasobit</td><td>20.2</td><td>249</td></tr>
<tr><td rowspan="2">Jefferson Co., FL</td><td>HMA</td><td>17.4</td><td rowspan="2">0.439</td><td>199</td><td rowspan="2">0.345</td></tr>
<tr><td>Terex foam</td><td>25.1</td><td>188</td></tr>
<tr><td colspan="6">Multiple WMA Technology Projects</td></tr>
<tr><td rowspan="2">Project Location</td><td rowspan="2">WMA Technologies</td><td rowspan="2">Std. Dev. (psi)</td><td>Bartlett’s test for equal variance</td><td rowspan="2">Avg. (psi)</td><td>Dunnett’s test of mean versus control</td></tr>
<tr><td>p-value</td><td>p-value</td></tr>
<tr><td rowspan="3">Rapid River , MI</td><td>HMA</td><td>5.2</td><td rowspan="3">0.62</td><td>48</td><td></td></tr>
<tr><td>Advera</td><td>8.2</td><td>67</td><td>0.001</td></tr>
<tr><td>Evotherm 3G</td><td>7.3</td><td>54</td><td>0.250</td></tr>
<tr><td rowspan="4">Munster, IN</td><td>HMA</td><td>8.6</td><td rowspan="4">0.539</td><td>105</td><td></td></tr>
<tr><td>Evotherm</td><td>16.6</td><td>119</td><td>0.315</td></tr>
<tr><td>Gencor foam</td><td>14.0</td><td>109</td><td>0.945</td></tr>
<tr><td>Heritage wax</td><td>19.0</td><td>120</td><td>0.282</td></tr>
<tr><td rowspan="4">New York, NY</td><td>HMA</td><td>13.2</td><td rowspan="4">0.087</td><td>74</td><td></td></tr>
<tr><td>BituTech PER</td><td>5.0</td><td>55</td><td>0.048</td></tr>
<tr><td>Cecabase</td><td>18.3</td><td>64</td><td>0.368</td></tr>
<tr><td>SonneWarmix</td><td>10.8</td><td>71</td><td>0.954</td></tr>
</table>

Table 192 Summary of Statistical Analyses of 2-2.5 Year Cores Tensile Strengths

Single WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (psi)	F test *p-value*	Avg. (psi)	*t*-test *p-value*
Walla Walla, WA	HMA	4.5	0.001	177	0.396
	Aquablack	30.6		165	
Centreville, VA	HMA	31.1	0.225	166	0.704
	Astec DBG	55.8		176	
Baker, MT	HMA	6.0	0.052	79	0.005
	Evotherm DAT	2.5		70	
Jefferson Co., FL	HMA	45.9	0.685	184	0.816
	Terex foam	55.6		177	
Graham, TX	HMA	13.1	0.899	258	0.792
	Astec DBG	12.4		256	
Multiple WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (psi)	Bartlett's test for equal variance *p-value*	Avg. (psi)	Dunnett's test of mean versus control *p-value*
Rapid River, MI	HMA	2.1	0.088	71	
	Advera	6.0		79	0.010
	Evotherm 3G	3.4		66	0.110
Munster, IN	HMA	12.0	0.256	124	
	Evotherm	36.7		130	0.976
	Gencor foam	33.1		143	0.589
	Heritage wax	26.9		131	0.952
New York, NY	HMA	32.9	0.029	133	
	BituTech PER	18.8		100	0.028
	Cecabase	7.6		105	0.069
	SonneWarmix	14.5		108	0.119
Silverthorne, CO	HMA	12.6	0.6	94	
	Advera	6.0		97	0.940
	Evotherm DAT	7.0		97	0.915
	Sasobit	7.5		98	0.859

Table 193 Summary of Statistical Analyses of Cores > 3 Year Cores Tensile Strengths

Single WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (psi)	F test *p-value*	Avg. (psi)	*t*-test *p-value*
Iron Mtn., MI	HMA	9.4	0.923	71	0.123
	Sasobit	9.1		81	
George, WA	HMA	11.3	0.034	189	0.357
	Sasobit	33.2		175	
Multiple WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (psi)	Bartlett's test for equal variance *p-value*	Avg. (psi)	Dunnett's test of mean versus control *p-value*
St. Louis, MO	HMA	33.0	0.122	161	
	Aspha-min	13.0		175	0.491
	Evotherm ET	18.0		181	0.230
	Sasobit	16.7		188	0.081
Silverthorne, CO	HMA	3.1	0.110	63	
	Advera	5.3		60	0.864
	Evotherm DAT	7.1		61	0.925
	Sasobit	10.2		56	0.255
Franklin, TN1	HMA	27.3	0.147	123	
	Advera	14.2		162	0.015
	Sasobit	11.0		153	0.035
Franklin, TN2	HMA	10.6	0.537	139	
	Astec DBG	14.0		157	0.174
	Evotherm DAT	19.4		176	0.005

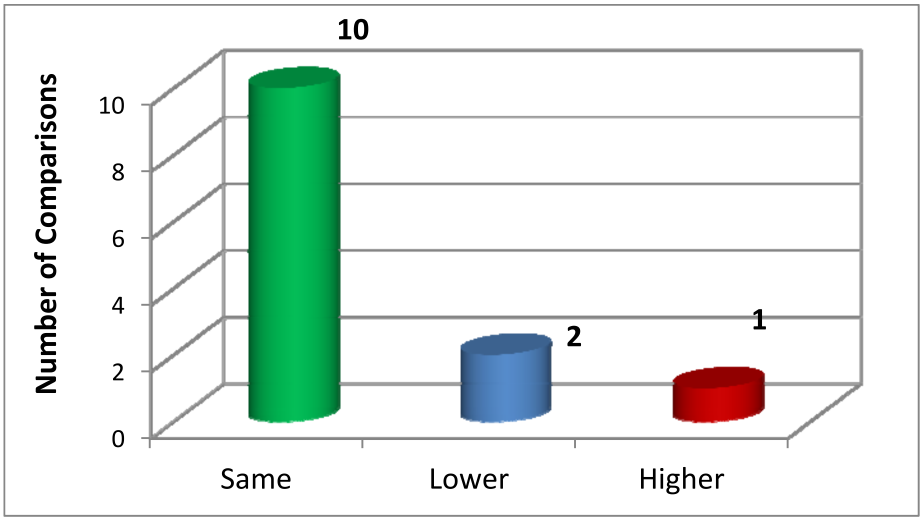


Figure 129 Comparison of WMA vs. HMA Tensile Strength –1 Year Cores

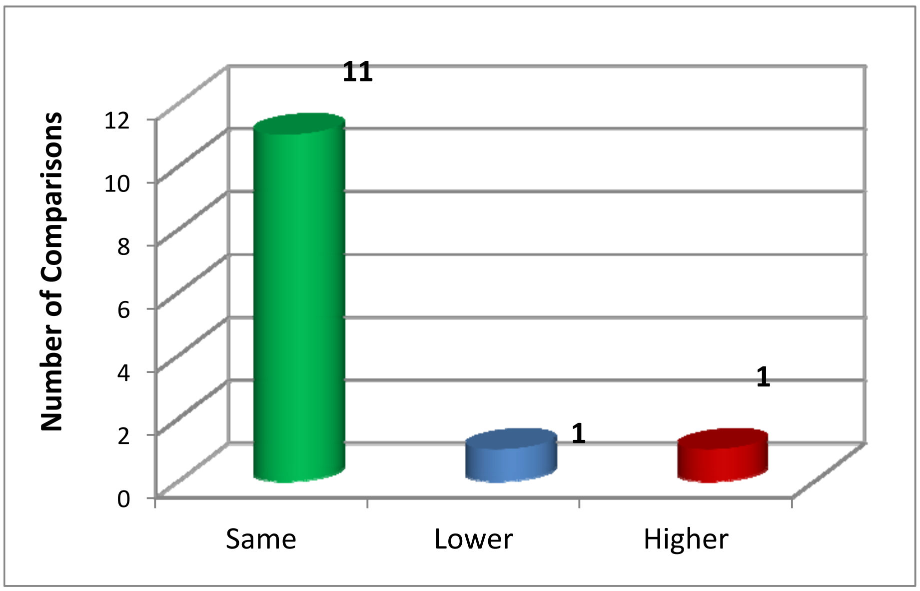


Figure 130 Comparison of WMA vs. HMA Tensile Strength –2-2.5 Year Cores

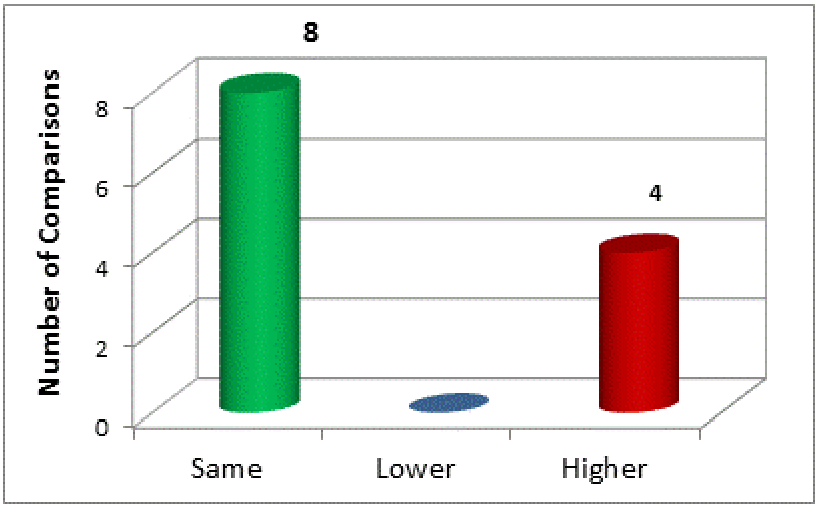


Figure 131 Comparison of WMA vs. HMA Tensile Strength Cores > 3 Years Old

Tensile Strength Ratio

Table 194 summarizes the tensile-strength ratios (TSR) for all the mixtures of each project. AASHTO M323-07 recommends a minimum TSR of 0.8 for moisture-resistant mixes. The following mixtures did not pass the minimum criteria:

- Jefferson, Co., FL (Terex Foam),
- Munster, IN (Evotherm),
- Franklin TN (HMA and Evotherm DAT),
- St. Louis, MO (HMA and Sasobit).

The mix with the poorest TSR was the Evotherm DAT mix from the Franklin, TN project.

According to NCHRP Research Results Digest 351 *(28),* the within-laboratory repeatability of AASHTO T 283 is 9 percent. Nine of the 22 WMA-HMA comparisons had TSRs that differed by more 9 percent; six of those had TSRs for the WMA more than 9 percent lower than the corresponding HMA (identified by light blue shading in Table 194), and three had TSRs for the WMA more than 9 percent higher than the corresponding HMA (identified by light pink shading in Table 194). Since there are no replicates for TSR values, comparison of the WMA and HMA results was made using a paired *t*-test for all projects. The *p*-value of the paired *t*-test was 0.312, which indicates that overall TSR values of the WMA and HMA mixes are not significantly different.

Table 194 TSR Results

Project Location	WMA Technologies	TSR	Criteria Pass/Fail
Walla Walla, WA	HMA	0.89	P
	Aquablack	0.86	P
Centreville, VA	HMA	0.89	P
	Astec DBG	0.83	P
Baker, MT	HMA	1.04	P
	Evotherm DAT	0.94	P
Casa Grande, AZ	HMA	0.98	P
	Sasobit	0.92	P
Jefferson Co., FL	HMA	0.91	P
	Terex foam	0.76	F
Graham, TX	HMA	0.90	P
	Astec DBG	0.87	P
Rapid River , MI	HMA	0.95	P
	Advera	0.88	P
	Evotherm 3G	1.00	P
Munster, IN	HMA	0.90	P
	Evotherm	0.78	F
	Gencor foam	0.83	P
	Heritage wax	0.83	P
New York, NY	HMA	0.83	P
	BituTech PER	0.85	P
	Cecabase	0.84	P
	SonneWarmix	0.80	P
Franklin, TN	HMA	0.73	F
	Astec DBG	0.83	P
	Evotherm DAT	0.53	F
Silverthorne, CO	HMA	1.00	P
	Advera	0.83	P
	Sasobit	1.11	P
	Evotherm DAT	0.80	P
St. Louis, MO	HMA	0.76	F
	Sasobit	0.78	F
	Evotherm ET	0.80	P
	Aspha-min	1.15	P

Hamburg Wheel Tracking Test

The moisture damage susceptibility of the WMA and HMA mixes was also assessed using the Hamburg wheel tracking test per AASHTO T 324. All Hamburg specimens were fabricated in the field. Two twin sets were tested per mix. Specimens were conditioned and tested in a 50°C water bath. Submerged specimens were subjected to 10,000 cycles (20,000 passes) of wheel loadings.

Table 195 shows a summary of the statistical analyses of the Hamburg rut depths. The variances were statistically different for two of five projects, Franklin, TN (groups A and B) and St. Louis, MO. For these projects (Franklin group A and St. Louis), there was only one replicate for one of the WMA technologies evaluated (Sasobit and Aspha-min, respectively). Because of this, the variances for these cases were excluded from the analysis. The mean rut depths of the WMA and respective HMA were statistically different for the following nine WMA mixes:

- Baker, MT, Evotherm
- Jefferson Co., FL, Terex foam
- Casa Grande, AZ, Sasobit
- Rapid River, MI, Advera and Evotherm 3G
- Munster, IN, Gencor Ultrafoam, and
- New York, NY, BituTech PER, Cecabase, and SonneWarmix.

Except for the Sasobit mix from Casa Grande, AZ, all of these WMA mixes had statistically higher Hamburg rut depths than their corresponding HMA mixes. However, the Terex foam WMA from Jefferson Co., FL performed very well in the Hamburg and would not be considered to be different from its companion HMA in a practical sense. The statistical results presented in Table 195 are summarized in Figure 132.

Table 195 Summary of Statistical Analyses of Hamburg Rut Depths

Single WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (mm)	F test *p-value*	Avg. (mm)	*t*-test *p-value*
Walla Walla, WA	HMA	4.853	0.631	7.43	0.730
	Aquablack	3.295		8.69	
Centreville, VA	HMA	0.256	0.499	2.483	0.966
	Astec DBG	0.444		2.497	
Baker, MT	HMA	1.473	0.230	15.00	0.077
	Evotherm DAT	4.089		20.94	
Jefferson Co., FL	HMA	0.218	0.420	1.243	0.009
	Terex foam	0.423		2.553	
Graham, TX	HMA	8.098	0.853	20.91	0.939
	Astec DBG	6.428		20.27	
George, WA	HMA	0.295	0.922	3.85	0.768
	Sasobit	0.273		3.777	
Casa Grande, AZ	HMA	0.567	0.010	5.05	0.093
	Sasobit	2.538		1.75	
Multiple WMA Technology Projects					
Project Location	WMA Technologies	Std. Dev. (mm)	Bartlett's test for equal variance *p-value*	Avg. (mm)	Dunnett's test of mean versus control *p-value*
Franklin, TN (A)	HMA1	0.382	0.064	15.220	
	Advera	7.552		18.540	0.825
	Sasobit	n =1 rep		8.890	0.661
Franklin, TN (B)	HMA2	13.831	0.065	24.510	
	Astec DBG	0.142		10.500	0.335
	Evotherm	7.184		17.780	0.706
Rapid River, MI	HMA	8.988	0.362	54.553	
	Advera	11.714		116.10	0.008
	Evotherm 3G	25.781		122.44	0.005
Munster, IN	HMA	1.031	0.323	4.860	
	Evotherm	2.571		8.863	0.2256
	Gencor Ultrafoam	4.455		11.613	0.0349
	Heritage wax	0.711		5.540	0.9779
New York, NY	HMA	1.309	0.492	2.930	
	Bitutech PER	3.741		14.966	0.0021
	Cecabase	3.666		20.829	0.0002
	SonneWarmix	1.742		13.449	0.0049
St. Louis, MO	HMA	5.231	0.008	7.392	
	Evotherm	1.319		3.743	0.107
	Sasobit	1.542		3.669	0.121
	Aspha-min	n=1		3.71	0.498

The results of the statistical analyses of Hamburg stripping inflection points (SIPs) are shown in Table 196. Except for the Walla Walla, WA project, variances of WMA and HMA SIPs were not statistically different. The Aquablack WMA from Walla Walla, WA had a statistically higher variance than its corresponding HMA. With regard to comparisons of the mean SIPs, the following WMA mixes were statistically lower (worse) than their corresponding HMA mixes:

- Franklin, TN, Advera,
- Rapid River, MI Advera
- New York, NY, BituTech, Cecabase and SonneWarmix

The SIP of the Aquablack WMA from Walla Walla, WA was statistically higher (better) than its corresponding HMA. It is important to mention that the mixes from Centreville, VA and Jefferson Co., FL did not have a stripping inflection point through 10,000 cycles, so the mean SIP was set at 10,000 cycles, but no statistical comparisons were conducted. Figure 132 summarizes the statistical analyses presented in Table 196; for twelve of eighteen comparisons, the stripping inflections points of WMA and HMA are the same (no statistical difference), five are lower (worse) and one is higher (better).

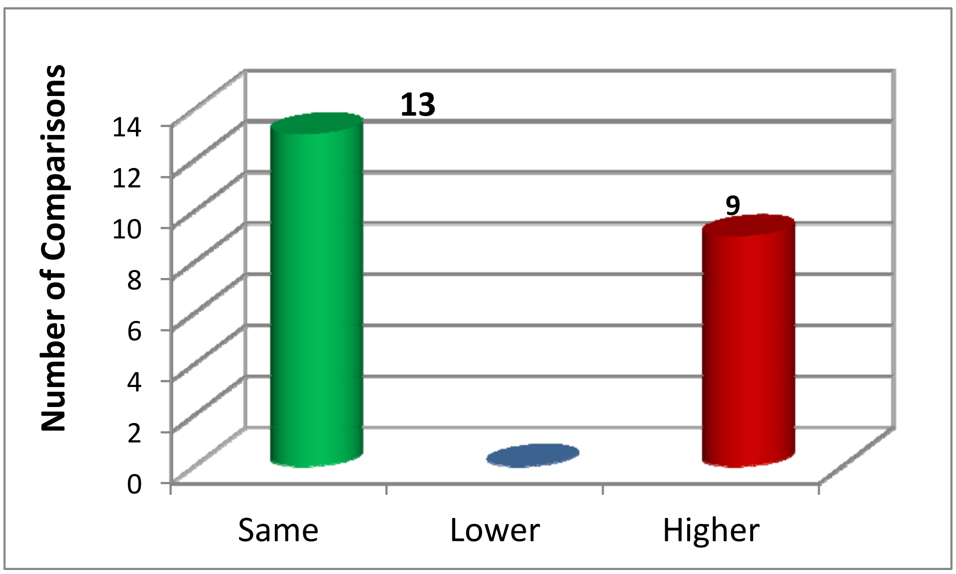


Figure 132 Comparison of WMA vs. HMA Hamburg Rut Depths

Table 196 Summary of Statistical Analyses of Hamburg Stripping Inflection Points

Single WMA Technology Projects

Project Location	WMA Technologies	Std. Dev. (cycles)	F test	Avg. (cycles)	*t*-test
Walla Walla, WA	HMA	58	0.010	5767	0.035
	Aquablack	802		8167	
Centreville, VA	HMA	N/A	N/A	>10000	N/A
	Astec DBG	N/A		>10000	
Baker, MT	HMA	420	0.266	5433	0.413
	Evotherm DAT	1071		4827	
Jefferson Co., FL	HMA	N/A	N/A	>10000	N/A
	Terex foam	N/A		>10000	
Graham, TX	HMA	354	0.835	7250	0.241
	Astec DBG	460		6575	
Casa Grande, AZ	HMA	N/A	N/A	>10,000	N/A
	Sasobit	184		9155	

Multiple WMA Technology Projects

Project Location	WMA Technologies	Std. Dev. (cycles)	Bartlett's test for equal variance *p-value*	Avg. (cycles)	Dunnett's test of mean versus control *p-value*
Franklin, TN (A)	HMA1	672	0.910	6925	
	Advera	583		3512	0.058
	Sasobit	n=1		8600	0.278
Franklin, TN (B)	HMA2	2563	0.406	6925	
	Astec DBG	1255		3512	0.862
	Evotherm	389		8600	0.162
Rapid River, MI	HMA	352	0.295	1157	
	Advera	114		703	0.089
	Evotherm 3G	142		807	0.184
Munster, IN	HMA	1605	0.240	5608	
	Evotherm	298		4438	0.444
	Gencor Ultrafoam	625		4437	0.443
	Heritage wax	1237		6450	0.667
New York, NY	HMA	1004	0.196	9202	
	Bitutech PER	190		3722	0.000
	Cecabase	297		3163	0.000
	SonneWarmix	553		3798	0.000
St. Louis, MO	HMA	2104	0.111	8850	
	Evotherm	1022		8913	0.999
	Sasobit	745		9042	0.990
	Aspha-min	n=1 rep.		10000	0.753

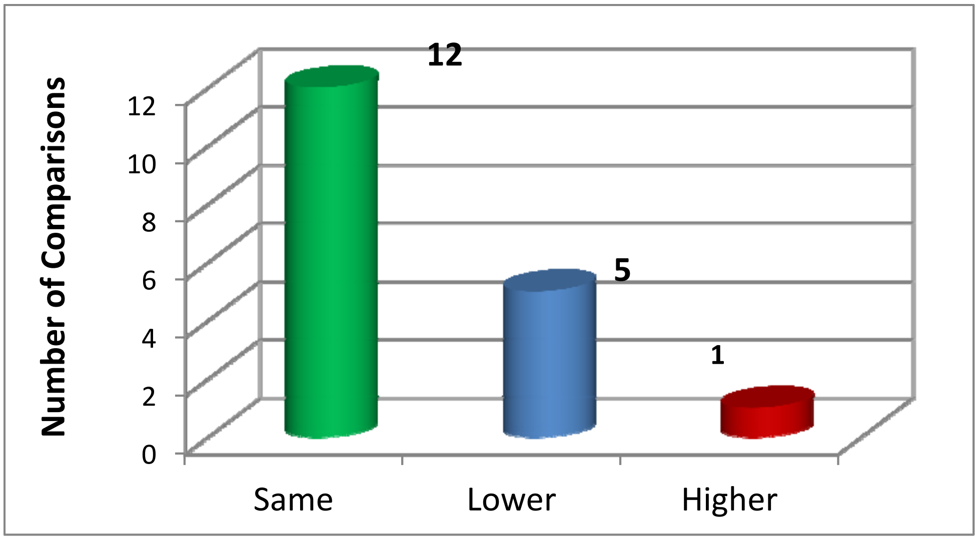


Figure 133 Comparison of WMA vs. HMA Hamburg Stripping Inflection Points

Fatigue

Uniaxial fatigue testing was performed to determine fatigue properties of the eleven mixes from Rapid River, MI, New York, NY, and Munster, IN. The fatigue testing followed the draft test procedure, *Determining the Damage Characteristic Curve of Asphalt Concrete from Direct Tension Fatigue Tests,* developed by the asphalt pavement research group led by Dr. Richard Kim at North Carolina State University (NCSU). To characterize the fatigue behavior of a mixture using the Simplified Visco-elastic Continuum Damage (S-VECD) model, two tests were performed in the AMPT. First, the dynamic modulus test was performed according to the AASHTO TP 79-10 to determine the linear viscoelastic (LVE) characteristics of the mix. Second, a controlled crosshead cyclic fatigue test was performed using the fatigue testing software in AMPT to acquire the necessary fatigue data.

Typically, three samples of mix were required for dynamic modulus testing and four to six samples were needed to get sufficient fatigue data. The controlled crosshead fatigue test is performed at 19°C at a frequency of 10 Hz.

The S-VECD fatigue data analysis was performed in an EXCEL® spreadsheet using the parameters developed by the NCSU fatigue analysis software. Five primary steps were needed for the data processing:

1. The number of testing cycles to failure was determined for each specimen based on the phase angle curve.
2. The AMPT dynamic modulus data were entered into the fatigue analysis software.
3. The fatigue data files were individually analyzed to determine the C (pseudo stiffness) versus S (damage parameter) curve.

4. The combined C versus S curve for the mix was then determined based on the individual C versus S curves. The composite C versus S curve is fit using a power law, shown as Equation 1 (where C_{11} and C_{12} are the regression coefficients).

$$C = 1 - C_{11} S^{C_{12}} \tag{1}$$

5. Finally, a fatigue prediction is made using the S-VECD model. Fatigue predictions for this study were made in terms of cycles to failure, N_f, using the controlled-strain assumption based on the formula in Equation 2.

$$N_f = \frac{(f_R)(2^{3\alpha})S_f^{\alpha - \alpha C_{12} + 1}}{(\alpha + \alpha C_{12} + 1)(C_{11}C_{12})^{\alpha}\left[(\beta + 1)(\varepsilon_{0PP})(|E^*|_{LVE})\right]^{2\alpha} K_1} \tag{2}$$

Where:

C = pseudo-stiffness
S = damage parameter
f_R = reduced frequency for dynamic modulus shift factor at fatigue simulation temperature and loading frequency
α = damage evolution rate for S-VECD model
$\varepsilon_{0,pp}$ = peak-to-peak strain for fatigue simulation
$|E^*|_{LVE}$ = dynamic modulus of mix from dynamic modulus mastercurve at the fatigue simulation temperature and loading frequency
C_{11}, C_{12} = power law coefficients from C vs. S regression
β = mean strain condition (assumed to be zero for this project)
K_1 = adjustment factor based on time history of loading – function of α and β

Figure 134 through Figure 136 show the pseudo-stiffness (C) versus damage parameter (S) curves for the mixes from the three projects Rapid River, MI, New York, NY and Munster, IN, respectively. These curves were modeled using the power model shown in Equation 1. The curves are plotted to the average C (pseudo-stiffness) at which the samples for that mix failed. Based on the results from these figures, the values of N_f from Equation 2 were plotted for each project at different strain levels. Figure 137 through Figure 139 show cycles to failures as a function of microstrain for all the mixes from the three projects mentioned above.

Of the Michigan mixes, the HMA and the Advera mix had similar laboratory fatigue results, and the Evotherm mix had a better fatigue result. Of the New York, NY mixes, the HMA, BituTech PER, and SonneWarmix WMAs had similar laboratory fatigue results. The Cecabase mix on the other hand, had a better fatigue result in terms of number of cycles to failure. Of the Indiana mixes, the HMA and the Gencor Foam mixes had similar fatigue results; the Evotherm 3G and Heritage Wax mixes had superior fatigue results compared to the HMA.

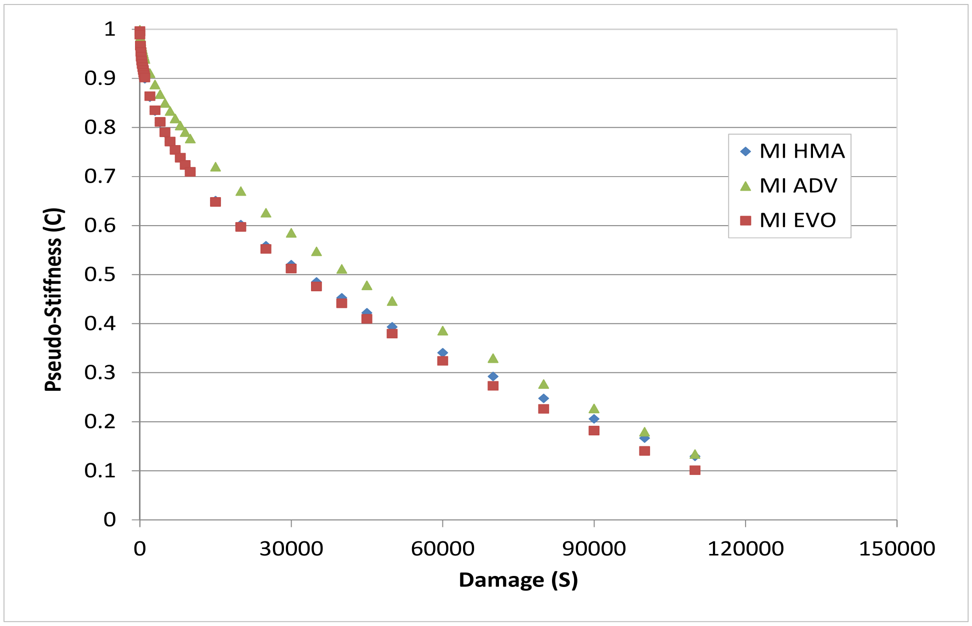


Figure 134 Pseudo-Stiffness (C) versus damage parameter (S) curves for HMA control mix and WMA technologies, Rapid River, MI project

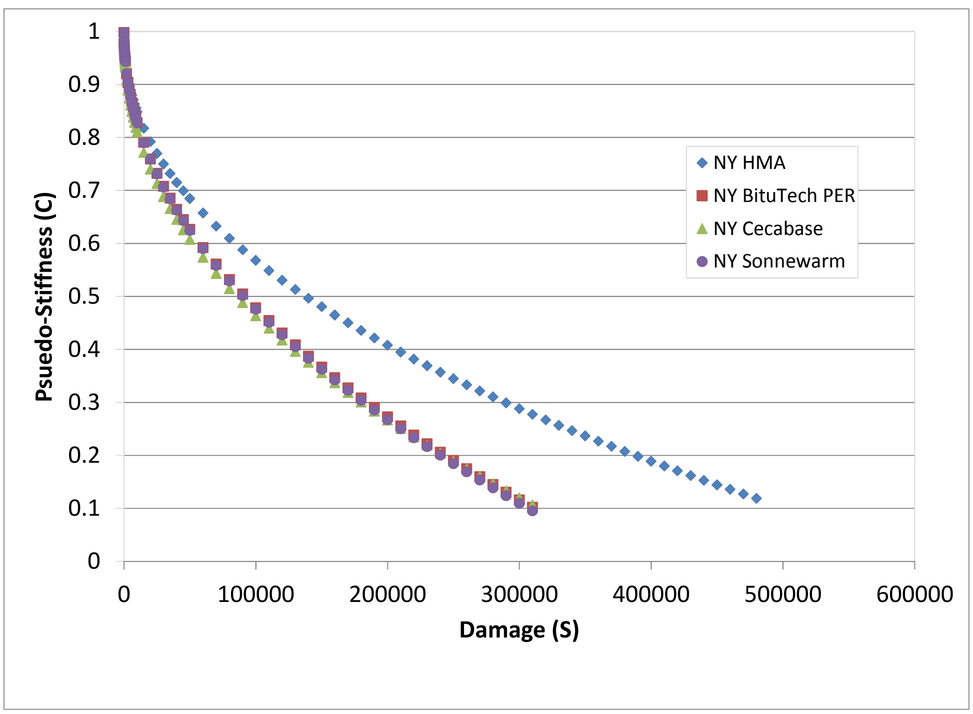


Figure 135 Pseudo-Stiffness (C) versus damage parameter (S) curves for HMA control mix and WMA technologies, New York, NY project

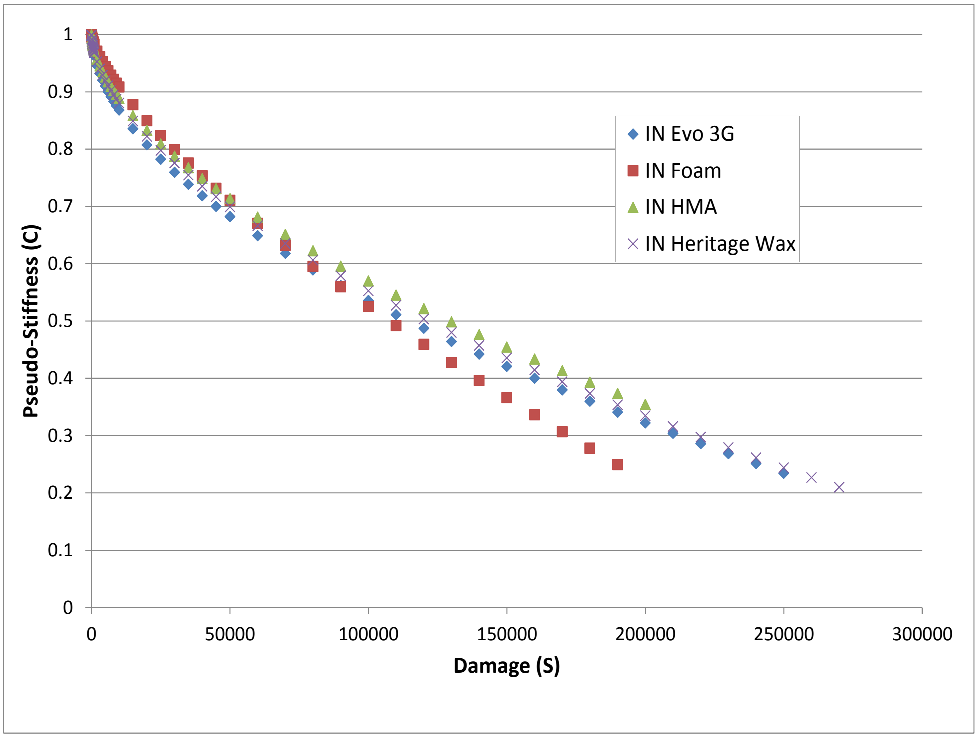


Figure 136 Pseudo-Stiffness (C) versus damage parameter (S) curves for HMA control mix and WMA technologies, Munster, IN project

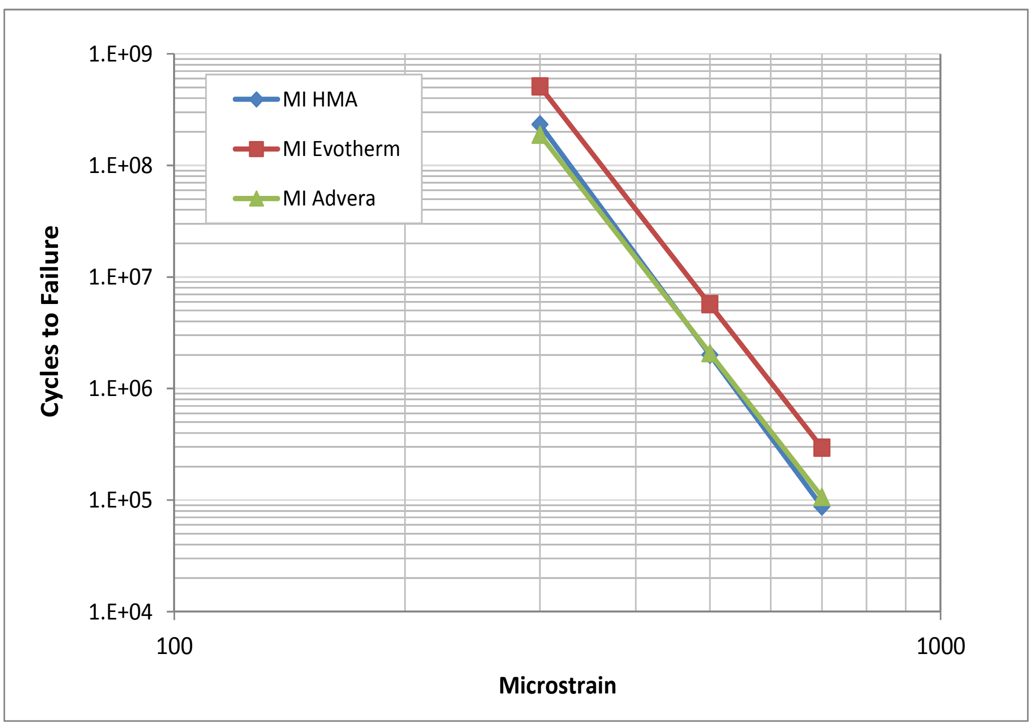


Figure 137 AMPT Fatigue Results for Rapid River, MI project

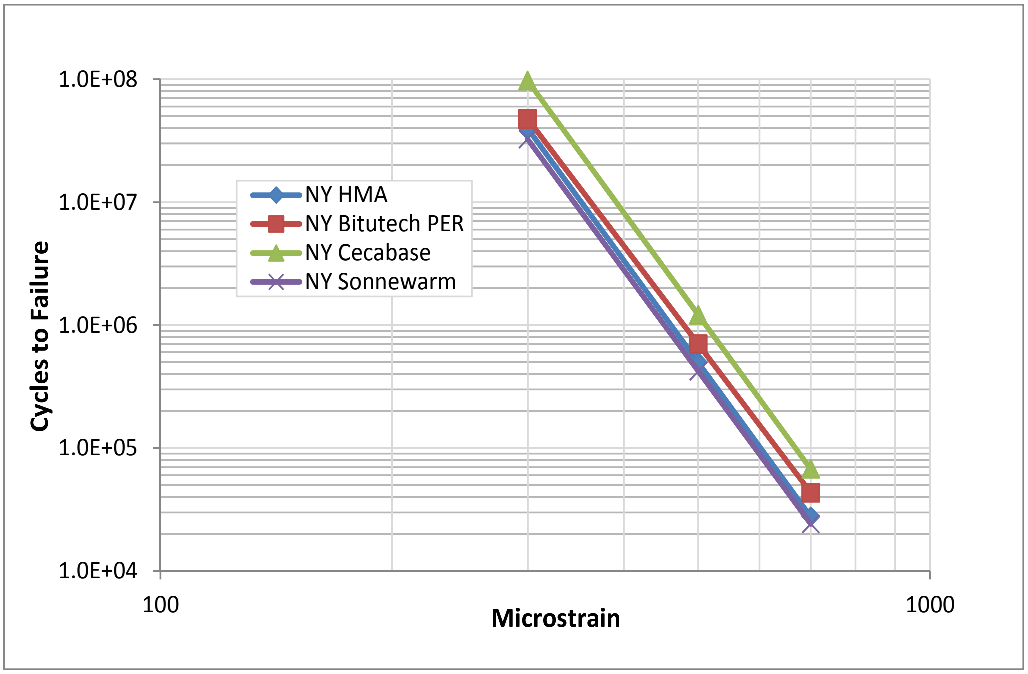


Figure 138 AMPT Fatigue Results for New York, NY project

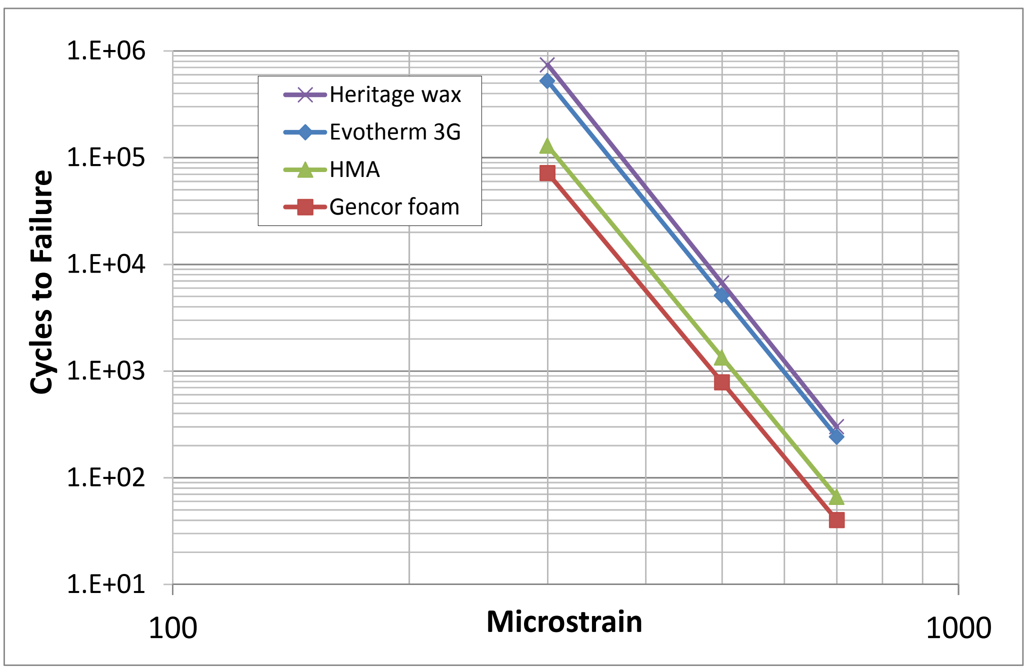


Figure 139 AMPT Fatigue Results for Munster, IN project

Indirect Tension Compliance and Strength

AASHTO T 322-07 was used to evaluate the resistance to thermal cracking for mixes from project locations with colder climates. The results are presented in Table 197. Although there are no consensus-required tensile strengths or failure times for asphalt mixtures to resist low-temperature cracking, for all projects, the WMA mixtures have longer failure time and lower critical low temperature that their corresponding HMA mixtures. This is an indication that WMA mixes should perform equal to or better than HMA with regard to low temperature cracking.

Table 197 AASHTO T 322 Indirect Tension Testing Results

Project Location	WMA Technologies	Average IDT Strength (MPa)	Failure Time (Hours)	Critical Low Temperature (°C)
Walla Walla, WA	HMA	3,772,509	4.50	-25.00
	Aquablack	4,034,005	4.56	-26.11
Centreville, VA	HMA	4,588,741	4.50	-25.00
	Astec DBG	4,085,364	4.61	-25.56
Rapid River, MI	HMA	3,922,690	5.17	-31.67
	Evotherm 3G	3,437,111	5.42	-34.17
	Advera	3,546,542	5.69	-36.94
Baker, MT	HMA	4,049,598	5.03	-30.68
	Evotherm DAT	3,596,706	5.17	-31.67
Munster, IN	HMA	4,411,905	4.39	-23.89
	Evotherm 3G	4,237,548	4.89	-28.89
	Gencor foam	4,451,076	4.39	-23.89
	Heritage Wax	4,555,655	4.67	-26.67

Comparison of Lab Test Results and Field Performance

This section discusses the results of the laboratory tests used to assess the resistance of the study mixtures to common asphalt pavement distresses and how those results compare to actual field performance. The section is organized to discuss results and performance related to rutting, moisture damage, fatigue cracking, and low temperature cracking.

Rutting

Since each of the field projects are subjected to different traffic (and environmental) conditions, comparisons of the laboratory results with field performance were sorted by the expected 20-year design ESALs determined for each project and compared to the suggested Flow Number criteria from NCHRP Report 673 for HMA and NCHRP Report 691 for WMA. Those criteria are shown in Table 198. The suggested Hamburg criteria shown in Table 198 are based on limited data from the NCAT Test Track (*4*) for tests conducted in accordance with AASHTO T 324 at 50°C.

All of the projects except Baker, MT had WMA and HMA sections placed in different lanes. For the Rapid River, MI, Jefferson County, FL, and Casa Grande, AZ projects the WMA and HMA mixes were placed in the travel lane but in opposite directions. The New York, NY mixes were all placed in the travel lane, but in two different directions. Data indicates half of the project receives lower traffic; the mixes which received lower traffic are noted (same ESAL range). Mix was placed in different lanes in the same direction for the Walla Walla, WA and Centreville, VA projects; mixes placed in the inner lanes are noted. For the Munster, IN project, mixes were placed in different directions and lanes; mixes placed in the inner lanes are noted, but visual observations indicate truck traffic was evenly divided between the lanes in this urban area.

Table 198 Recommended Criteria for Rutting Tests

Traffic Level Million ESALs	Min. Flow Number for HMA (cycles)	Min. Flow Number for WMA (cycles)	Max. Hamburg Rut Depth (mm)
< 3	--	--	--
3 to < 10	53	30	10
10 to <30	190	105	8
≥ 30	740	415	6

Two of the new projects were estimated to have less than one million ESALs for the 20-year design traffic. These projects were Rapid River, MI, and Baker, MT. Table 199 summarizes the field measured rutting and the results of the laboratory rutting tests for the mixes from these two projects.

Table 199 Laboratory Rutting Test Results and Field Performance for Projects < 3 Million ESALs

Project Location	WMA Technologies	Field Rutting (mm)	Unc. Flow No. (cycles) Hot/Reheated		Hamburg Rut Depth (mm)	
			Avg.	COV	Avg.	COV
Baker, MT Route 322	HMA	0.5	--/98	--/30	15.0	9
	Evotherm DAT	0.2	--/58	--/3	20.9	20
Rapid River, MI CR-315	HMA	0.0	--/199	--/14	54.6[1]	16
	Advera	0.0	--/60	--/2	116.1[1]	10
	Evotherm 3G	0.0	--/65	--/17	122.4[1]	21

[1] - extrapolated values

There are no recommended FN or Hamburg rut depth criteria for mixes used in pavements with design traffic less than 3 million ESALs. The FN results appear to be satisfactory for all mixes although the tests were conducted on reheated mix samples. The mixes did not perform well in the Hamburg test. However, no Hamburg criteria have been suggested for this traffic category. As previously noted, the Hamburg results for the Rapid River, MI mixes should be viewed with caution since the test temperature was not adjusted for the soft binder used in this cooler climate. Overall, the FN and Hamburg results for these mixes seem reasonable and the expected trend is evident—the results for HMA mixes are better than for the respective WMA mixes. Given that these mixes have performed very well in the field reinforces the idea that laboratory rutting tests are not appropriate for mixes intended for use in light traffic applications.

Two of the new projects were estimated to have about 3 million ESALs for the 20-year design period. Those projects were Jefferson Co., FL, and Casa Grande, AZ. Table 200 summarizes the field measured rutting and the result of the laboratory rutting tests for the mixes from these two projects.

Table 200 Laboratory Rutting Test Results and Field Performance for Projects ~ 3 Million ESALs

Project Location	WMA Technologies	Field Rutting (mm)	Unc. Flow No. (cycles) Hot/Reheated		Hamburg Rut Depth (mm)	
			Avg.	COV	Avg.	COV
Jefferson Co., FL US 98	HMA	2.9	414/231	8/29	1.2	18
	Terex foam	3.0	157/127	17/16	2.6	17
Casa Grande, AZ SR 84	HMA	3.2	61/--	31/--	1.8	32
	Sasobit	0.0	46/--	26/--	5.0	50

Three projects were estimated to have between 3 and 10 million ESALs. They were Walla Walla, WA, Munster, IN and New York, NY. Rutting test results for the mixes from these three projects are shown in Table 201. All of the mixes easily met the Flow Number criteria for the 3 to 10 million ESAL range and actually also met the criteria for the next higher traffic category. However, several of the WMA mixes did not satisfy the suggested Hamburg criteria (maximum, 10 mm). Although the excellent field performance of these mixes could justify revising the Hamburg criteria for WMA, it seems risky to raise the criteria so high that all of the WMA mixes would pass. More data would be helpful in establishing Hamburg criteria for WMA.

Table 201 Laboratory Rutting Test Results and Field Performance for Projects 3 - 10 Million ESALs

Project Location	WMA Technologies	Field Rutting (mm)	Unc. Flow No. (cycles) Hot/Reheated		Hamburg Rut Depth (mm)	
			Avg.	COV	Avg.	COV
Walla Walla, WA	HMA	4.6	332/426	28/26	7.4	65
	Aquablack	0.0*	200/227	15/6	8.7	38
Munster, IN Calumet Ave.	HMA	0.0	561/--	39/--	4.9	21
	Evotherm 3G	0.0*	177/--	3/--	8.9	29
	Gencor Ultrafoam	0.0	217/--	2/--	11.6	38
	Heritage wax	0.0*	314/--	12/--	5.5	13
New York, NY Little Neck Pkwy.	HMA	1.9	291/--	19/--	2.9	45
	Bitutech PER	2.7*	128/--	9/--	15.0	25
	Cecabase	0.3*	115/--	3/--	20.8	18
	SonneWarmix	0.0	123/--	13/--	13.4	13

*HMA and WMA were in different lanes, may have had slightly different traffic

The project with the highest estimated design traffic was Centerville, VA. The Centreville, VA has an estimated design traffic of about 32.5 million ESALs. As shown in Table 202, the Flow Number results for the Centreville mixes meet the Flow Number criteria for greater than 30 million ESALs, but the results are for reheated mix samples. It seems likely that the HMA mix would have met the minimum Flow Number criteria for hot compacted samples, but probably not for hot compacted WMA. On the other hand, the Hamburg results for the Centerville mixes met the suggested criteria.

Table 202 Laboratory Rutting Test Results and Field Performance for Project > 30 Million ESALs

Project Location	WMA Technologies	Field Rutting (mm)	Unc. Flow No. (cycles) Hot/Reheated		Hamburg Rut Depth (mm)	
			Avg.	COV	Avg.	COV
Centreville, VA	HMA	3.2	--/1855	--/16	2.5	10
	Astec DBG	2.7*	--/439	--/11	2.5	18

*HMA and WMA were in different lanes, may have had slightly different traffic

Based on the data from the thirteen mixes from eight project sites, the current FN criteria developed for assessing mixes during design seem to also be appropriate for monitoring field production. The suggested Hamburg criteria that were developed for HMA mixes based on performance on the NCAT Test Track seem appropriate for the HMA mixes in this study, but should probably be increased slightly for WMA mixes.

Moisture Damage

The TSR test and the Hamburg test were used to evaluate moisture damage susceptibility of the plant produced mixes. **Table 203** summarizes the results of these tests. Only six of the 34 mixes did not meet the standard minimum TSR criteria of 0.80 (identified by shaded cells), but four of those mixes had results just below the criteria with TSRs between 0.76 and 0.78. Some states also consider the conditioned tensile strengths as an indicator of moisture damage susceptibility. Except for the Baker, MT and Rapid River, MI projects that used softer asphalt grades, nearly all mixes had tensile strengths above 100 psi. The TSR and conditioned tensile strength results indicate that the WMA and HMA mixes were generally resistant to moisture damage, which is consistent with the observation of no stripping in any field cores. The only mixes with low TSRs and low tensile strengths were the Evotherm mixes from Munster, IN and Franklin, TN.

There are no nationally accepted criteria for the Hamburg Stripping Inflection Point. In other studies, NCAT has used 5000 cycles as a general minimum criterion for SIP (*29*). Eleven of the 34 mixes did not meet this suggested criterion. It is interesting to note that only one mix failed both TSR and Hamburg criteria, clearly indicating that the two methods do not provide consistent assessments of moisture damage susceptibility. Conflicting TSR and Hamburg results have been reported in other studies (*30*). Since no moisture damage was observed in any of the projects, both tests appear to give some false positive results. However, the TSR test appeared to have much fewer false positive results than the Hamburg test.

Table 203 TSR and Hamburg Results

Project Location	WMA Technologies	TSR	Conditioned Tensile Strength (psi)	Hamburg SIP (cycles)
Walla Walla, WA	HMA	0.89	119.7	5767
	Aquablack	0.86	101.9	8167
Centreville, VA	HMA	0.89	185.1	>10,000
	Astec DBG	0.83	143.3	>10,000
Baker, MT	HMA	1.04	72.1	5433
	Evotherm DAT	0.94	63.5	4827
Casa Grande, AZ	HMA	0.98	117.6	>10,000
	Sasobit	0.92	101.0	9155
Jefferson Co., FL	HMA	0.91	198.1	>10,000
	Terex foam	0.76	159.6	>10,000
Graham, TX	HMA	0.90	141.4	7250
	Astec DBG	0.87	96.6	6575
Rapid River , MI	HMA	0.95	50.0	1157
	Advera	0.88	30.8	703
	Evotherm 3G	1.00	36.6	807
Munster, IN	HMA	0.90	160.1	5608
	Evotherm	0.78	97.1	4438
	Gencor foam	0.83	110.6	4437
	Heritage wax	0.83	131.3	6450
New York, NY	HMA	0.83	173.3	9202
	BituTech PER	0.85	106.7	3722
	Cecabase	0.84	121.7	3163
	SonneWarmix	0.80	114.9	3798
Franklin, TN	HMA	0.73	115.5	6925
	Astec DBG	0.83	109.2	3512
	Evotherm DAT	0.53	73.4	8600
Silverthorne, CO	HMA	1.00	N.A.	7067
	Advera	0.83	N.A.	3300
	Sasobit	1.11	N.A.	5700
	Evotherm DAT	0.80	N.A.	6200
St. Louis, MO	HMA	0.76	126.9	8850
	Sasobit	0.78	101.9	8913
	Evotherm ET	0.80	102.7	9042
	Aspha-min	1.15	160.3	>10,000

N.A. - results not available

Fatigue Cracking

Laboratory fatigue cracking tests were conducted on a limited set of mixtures, namely the mixes from Rapid River, MI, Munster, IN, and New York, NY. The uniaxial fatigue test does not yield a unique test result, but rather a relationship between strain and the number of cycles to failure, as shown in Figure 137 to Figure 139. Therefore, the results provide a relative ranking of the fatigue behavior for a set of mixes that can be compared to field performance of sections subjected to the same loads, support conditions, and climate. Table 204 summarizes the cracking observed in the field and the relative ranking of laboratory fatigue characteristics. Note that the laboratory fatigue rankings are not statistically based since the log cycles to failure versus log microstrain relationships are not derived directly from replicate measurements as is commonly done for beam fatigue tests. Rather the rankings are based on engineering judgment considering typical variability of fatigue testing and the observed spacing of the fatigue relationships on the log-log plots.

The data in Table 204 indicate that each of the sections on the Rapid River, MI project were performing similarly. The minor amount of cracking was non-wheelpath, so the cracks are probably not load related. The uniaxial fatigue testing indicated that the Advera mix would be more fatigue resistant. Therefore, the comparison of laboratory and field results is inconclusive for this project. For the Munster, IN project, cracking was observed only in the outside lanes where the HMA and Gencor foamed WMA sections were placed. There was a substantial difference in the amount of cracking of these two sections, but the cracks were probably not load related. The uniaxial fatigue test results do correctly rank the HMA mix as being more resistant to fatigue cracking compared to the Gencor foamed WMA section. The other two sections on this project were placed in the inside lane and no cracking was observed in these lanes. The laboratory fatigue test indicated these mixes would have similar fatigue resistance and their fatigue characteristics were better than the mixes placed in the outside lanes. Therefore, the laboratory fatigue ranking appears to be consistent with field performance for this project. For the New York project, differences in cracking were observed in the four sections. However, the Cecabase section which had most cracking in the field had the best laboratory results. The BituTech section and the SonneWarmix section had similar amounts of cracking in the field, but the laboratory fatigue test ranked them differently. Therefore, the laboratory fatigue ranking does not appear to match field performance for this project. Overall, with regard to fatigue test results and field performance, one project appeared to match, one did not match, and one was inconclusive.

Table 204 Observed Field Cracking and Ranking of Lab Fatigue Results

Project Location (insp. age)	WMA Technologies (lane)	Cracking Total Length (m)	Orientation of Cracks	Severity of Cracks	Lab Fatigue Ranking
Rapid River, MI (22 mos.)	HMA (southbound lane)	0.3 non-WP	longitudinal	low	B
	Advera (northbound lane)	0.2 non-WP	longitudinal	low	A
	Evotherm 3G (northbound lane)	0.5 non-WP	longitudinal	low	B
Munster, IN (24 mos.)	HMA (outside lane)	0.9 3.3 non-WP	transverse longitudinal	low low	B
	Evotherm (inside lane)	0			A
	Gencor foam (outside lane)	6.1 29.6 non-WP	transverse longitudinal	low low	C
	Heritage wax (inside lane)	0			A
New York, NY (26 mos.)	HMA (southbound lanes)	5.5 0.3 WP 3.0 non-WP	transverse longitudinal longitudinal	low low low	C
	BituTech PER (northbound lane)	5.2 WP	longitudinal	low	B
	Cecabase (southbound lanes)	15.8 WP	longitudinal	low	A
	SonneWarmix (northbound lanes)	5.2 WP	longitudinal	low	C

Low Temperature Cracking

Thermal cracking characteristics were evaluated using the IDT Creep Compliance and Strength Test in accordance with AASHTO T 322 on mixes from five of the projects. The predicted critical low temperatures for thermal cracking for those mixes are summarized on Table 205. The table also includes a summary of observed transverse cracking for the five projects and the lowest air temperature during the periods between construction and the second project inspections from nearby weather stations from the Weather Underground website (www.wunderground.com). It can be seen that no transverse cracking had been observed for the first three projects. The recorded air temperatures for those projects were well above the critical low temperatures determined from the laboratory thermal cracking testing and analysis. For the Baker, MT project, the Evotherm WMA section had more cracking than the HMA section even though the calculated critical cracking temperature was one degree Celsius lower for the WMA mixture. The actual low temperature for Baker, MT was a few degrees colder than the critical cracking temperature for the two mixes on that project. For the Munster, IN project, the actual low temperature was higher than the calculated critical cracking temperature for all four test sections. The two sections with the lowest critical cracking temperature determined from laboratory tests (HMA and Gencor foamed WMA) did have cracking, but the amount of cracking was different. The other two WMA sections had higher critical cracking temperatures and had no transverse cracks were observed. Overall, the relationship between the IDT creep compliance and strength test results and the observed field performance was inconclusive.

Table 205 Predicted Critical Low Temperatures for Thermal Cracking

Project Location (Const. date, insp. age)	WMA Technologies	Observed Transverse Crack (m)	Critical Low Temperature (°C)	Lowest Recorded Temp. (°C)
Walla Walla, WA (Apr. 2010, 27 mos.)	HMA	none	-25.00	-19.2
	Aquablack	none	-26.11	
Centreville, VA (Jun. 2010, 24 mos.)	HMA	none	-25.00	-12.9
	Astec DBG	none	-25.56	
Rapid River, MI (Jul. 2010, 22 mos.)	HMA	none	-31.67	-29.4
	Evotherm 3G	none	-34.17	
	Advera	none	-36.94	
Baker, MT (Aug. 2010, 22 mos.)	HMA	3.7	-30.68	-32.8
	Evotherm DAT	7.3	-31.67	
Munster, IN (Sep. 2010, 24 mos.)	HMA	0.9	-23.89	-21.2
	Evotherm 3G	none	-28.89	
	Gencor foam	6.0	-23.89	
	Heritage Wax	none	-26.67	

CHAPTER 5

WMA PROJECT MIX VERIFICATION

The mixes from the multiple WMA technology projects (Michigan, Indiana, and New York) along with the mixes from Montana and Florida were verified according to the *Draft Appendix to AASHTO R35: Special Mixture Design Considerations and Methods for Warm Mix Asphalt (WMA)* presented in the final report FOR NCHRP Project 9-43 *(21).* This group of mixes provided a range of WMA technologies, aggregate types, and production and compaction temperatures.

Determination of Optimum Asphalt Content

One goal of the mix verifications was to determine if plant production of WMA could be simulated in the laboratory. Since changes in gradation during plant production affect the measured volumetric properties, the as-produced gradation and asphalt content were used as the target for the laboratory mix design verification for each combination of location and technology. Thus, within a given project, there were some differences in the target laboratory gradation, even though all of the mixes from a given location were based on the same design.

Rapid River, Michigan

Table 204 shows the JMF, measured field gradations and gradation checks of laboratory batched samples. For the Michigan project, the laboratory verification of the HMA mixture targeted the JMF rather than the field gradation to demonstrate that the research team could match the contractor's design. The asphalt contents for the "Field" mixes are those measured in the field samples; the "Lab" asphalt contents are the optimum asphalt contents determined from the mix verification. For this project, the optimum asphalt contents were selected at 4 percent air voids at 30 N_{design} gyrations. Both WMA technologies resulted in a reduction in optimum asphalt content compared to the HMA control.

Table 207 shows the volumetric properties at the asphalt contents used to bracket the field measured asphalt content. The field volumetric properties are also shown for comparison. The AASHTO T312 1s and d2s precision limits for multi-laboratory (NCAT personnel in NCAT mobile laboratory and AMS personnel in AMS laboratory) for relative density are 0.6 and 1.7 percent, respectively. All of the laboratory to field comparisons were within the d2s limit. It should be noted that the JMF gradation was targeted for the HMA laboratory verification and not the field produced HMA gradation. The HMA verification indicated a 0.02 percent difference in optimum asphalt content. The difference between relative density at N_{design} of the field produced and laboratory produced Evotherm 3G was 0.7 percent.

Table 206 Michigan Design, Field, and Verification Gradations and Asphalt Contents

Sieve Size	JMF	HMA		Advera		Evotherm 3G	
		Lab	Field	Lab	Field	Lab	Field
	Percent Passing						
19.0 mm	100	100	100	100	100	100	100
12.5 mm	93	94	95	95	95	90	95
9.5 mm	85	86	87	87	87	82	84
4.75 mm	66	67	72	69	68	62	64
2.36 mm	49	51	58	53	51	48	48
1.18 mm	36	38	44	40	38	37	36
0.60 mm	25	26	32	28	26	27	25
0.30 mm	17	17	21	19	18	18	18
0.15 mm	9	10	11	10	10	10	10
0.075 mm	5.8	5.7	6.3	6.1	6.0	6.0	6.4
AC (%)	5.30	5.32	5.00	4.95	5.34	4.83	5.00
Compaction Temp. (°F)		300		250		250	

Table 207 Summary of Michigan volumetric properties

AC (%)	G_{mm}	Air Voids (%)	VMA (%)	VFA (%)	P_{ba} (%)
HMA Field					
5.26	2.479	3.9	14.7	73	0.59
HMA Laboratory Verification					
4.76	2.504	6.1	15.4	61	
5.26	2.486	4.7	15.3	69	0.70[1]
5.76	2.467	3.2	15.1	79	
Advera WMX Field					
5.34	2.484	3.4	14.2	76	0.66
Advera WMX Laboratory Verification					
4.84	2.487	4.3	14.5	70	
5.34	2.468	2.9	14.4	80	0.47[1]
Evotherm 3G Field					
5.00	2.493	3.0	13.6	78	0.66
Evotherm 3G Laboratory Verification					
4.50	2.501	5.1	14.4	65	
5.00	2.482	3.7	14.2	74	0.45[1]
5.50	2.463	1.2	13.2	91	

[1]Maximum specific gravity tests were only performed at one asphalt content.

Baker, Montana

For the Montana project, N_{design} was specified as 75 gyrations. Table 208 shows the JMF, measured field gradations and gradation checks of laboratory batched samples. Table 209 shows the volumetric properties at the asphalt contents used to bracket the field measured asphalt content. The field volumetric properties are also shown for comparison.

Table 208 Montana Design, Field, and Verification Gradations and Asphalt Contents

Sieve Size	JMF	HMA		Evotherm DAT	
		Lab	Field	Lab	Field
	Percent Passing				
19.0 mm	100	100	100	100	100
12.5 mm	81	89	87	88	89
9.5 mm	69	75	76	76	75
4.75 mm	51	54	55	51	54
2.36 mm	31	33	30	30	33
1.18 mm	20	21	18	20	21
0.60 mm	14	13	12	13	13
0.30 mm	10	9	8	10	9
0.15 mm	7	6	6	7	6
0.075 mm	5.0	4.0	4.3	4.4	4.0
AC (%)	5.80	5.47	5.69	5.76	5.76
Compaction Temp. (°F)		270		235	

Table 209 Summary of Montana volumetric properties

AC (%)	G_{mm}	Air Voids (%)	VMA (%)	VFA (%)	P_{ba} (%)
HMA Field					
5.69	2.413	3	14.1	79	0.72
HMA Laboratory Verification					
5.19	2.446	4.4	14.0	69	
5.69	2.429	2.7	13.6	80	1.01
6.19	2.411	3.2	15.1	79	
Evotherm DAT Field					
5.76	2.407	4.0	15.5	74	0.65
Evotherm DAT Laboratory Verification					
5.00	2.445	7.3	16.5	56	
5.76	2.416	4.8	16.0	70	0.80
6.26	2.399	4.6	16.8	73	
6.76	2.382	4.6	17.8	74	

The laboratory verification of the Evotherm DAT mix could not achieve 4.0 percent air voids. At the field measured asphalt content, the air void content was 4.8 percent. Higher asphalt contents appeared to be on the wet-side of the VMA curve.

Munster, Indiana

For the Indiana project, N_{design} was specified as 75 gyrations. Table 210 shows the JMF, measured field gradations and gradation checks of laboratory batched samples.

Table 211 shows the volumetric properties at the asphalt contents used to bracket the field measured asphalt content. The field volumetric properties are also shown for comparison.

Table 210 Indiana design, field, and verification gradations and asphalt contents

Sieve Size	JMF	HMA		Wax		Foam		Evotherm J1	
		Lab	Field	Lab	Field	Lab	Field	Lab	Field
	Percent Passing								
12.5 mm	100	100	100	100	100	100	100	100	100
9.5 mm	92	95	94	95	94	95	94	96	94
4.75 mm	54	62	62	63	61	63	62	62	60
2.36 mm	41	39	40	40	40	40	41	36	39
1.18 mm	30	30	29	31	28	31	29	26	27
0.60 mm	22	22	20	22	20	22	20	18	18
0.30 mm	15	15	13	15	13	15	14	12	11
0.15 mm	10	10	9	10	9	10	10	8	8
0.075 mm	6.0	6.7	6.9	7.0	7.0	7.0	7.0	6.0	5.6
AC (%)	5.50	6.27	6.18	6.40	5.95	6.03	5.61	6.69	5.95
Comp. Temp. (°F)		285		240		230		240	

All of the laboratory-field comparisons were within the AASHTO T 312 d2s limit; only the Evotherm J1 and wax WMA exceeded the 1s limit. Higher optimum asphalt contents than both the JMF and field production were indicated in all cases. For this set of mixes, the laboratory percent binder absorbed (P_{ba}) was less than the field P_{ba} in all cases. Also, the P_{ba} of the WMA mixes were less than the HMA.

Table 211 Summary of Indiana volumetric properties

AC (%)	G_{mm}	Air Voids (%)	VMA (%)	VFA (%)	P_{ba} (%)
HMA Field					
6.18	2.526	5.6	16.4	66	1.58
HMA Laboratory Verification					
5.68	2.528	7.1	17.3	59	
6.18	2.509	5.0	16.5	70	1.29
6.68	2.490	3.7	16.5	78	
Foam Field					
5.61	2.525	5.6	16.0	65	1.18
Foam Laboratory Verification					
5.61	2.513	5.6	16.4	66	0.98
6.11	2.494	3.4	15.6	78	
6.61	2.470	2.1	15.7	86	
Evotherm J1 Field					
5.95	2.517	6.4	17.3	63	1.27
Evotherm J1 Laboratory Verification					
5.45	2.526	7.6	17.6	57	
5.95	2.507	7.1	18.3	61	1.10
6.45	2.488	5.7	18.0	69	
6.95	2.470	2.7	16.5	84	
Wax Field					
5.95	2.531	4.9	15.5	68	1.51
Wax Laboratory Verification					
5.95	2.505	6.1	17.4	65	1.10
6.45	2.486	3.8	16.5	77	

New York, New York

New York City DOT produces approximately 500,000 tons of the 1,000,000 tons of asphalt they place each year. Their typical surface mix is a 50-blow Marshall design with 40 percent RAP. They designed a Superpave mix with 25 percent RAP for this project. N_{design} was specified as 75 gyrations. Table 212shows the JMF, measured field gradations and gradation checks of laboratory batched samples. Table 213shows the volumetric properties at the asphalt contents used to bracket the field measured asphalt content. The field volumetric properties are also shown for comparison.

Table 212 New York Design, Field, and Verification Gradations and Asphalt Contents

Sieve Size	JMF	HMA		BituTech PER		Cecabase RT		SonneWarmix	
		Lab	Field	Lab	Field	Lab	Field	Lab	Field
	Percent Passing								
12.5 mm	100	100	100	100	100	100	100	99	100
9.5 mm	91	94	92	94	94	94	95	93	95
4.75 mm	56	57	55	59	59	59	61	58	62
2.36 mm	35	30	34	33	35	33	36	34	36
1.18 mm	25	23	24	24	24	24	26	25	25
0.60 mm	19	17	17	18	17	18	19	18	18
0.30 mm	13	11	12	12	12	12	13	13	13
0.15 mm	9	7	8	8	8	8	9	9	9
0.075 mm	6.4	5.3	5.0	6.3	5.4	6.3	6.3	6.6	6.1
AC (%)	5.30	6.88	5.38	6.06	5.48	5.96	5.66	6.20	5.30
Comp. Temp. (°F)		300		225		225		225	

New York State DOT Superpave requirements specify that the optimum asphalt content be selected at 3.5 percent voids. In all cases, the field air voids were higher than the target, therefore the optimum asphalt contents were higher than the values obtained from the field tests.

At the field produced asphalt content of 5.48%, the BituTech PER laboratory and field voids matched closely. However, the laboratory to field comparison for the Cecabase RT and SonneWarmix exceeded the d2s for relative density. The difference in voids for the HMA exceeded the 1s for relative density. Some of the differences between the laboratory and field results for the Cecabase RT and SonneWarmix may have been due to differences in gradations, particularly for the 2.36 and 4.75 mm sieves. Additional trials were prepared in an attempt to produce a closer gradation. These trials are shown in Table 214. The trials seem to confirm that the differences in gradation were not the primary cause for the differences in air voids. Instead, it appears that for some reason the laboratory mixes for the Cecabase RT and SonneWarmix did not properly replicate the field mixes.

The asphalt absorption results for all of the laboratory mixtures were all lower than for the corresponding field produced mixes. Practically, the differences were small.

Table 213 Summary of New York Volumetric Properties

AC (%)	Gmm	Air Voids (%)	VMA (%)	VFA (%)	P_{ba} (%)
HMA Field					
5.38	2.646	5.4	16.7	68	0.75
HMA Laboratory Verification					
4.88	2.656	7.6	18.0	58	
5.38	2.634	6.4	18.0	65	0.56
5.88	2.613	5.5	18.3	70	
6.38	2.591	4.6	18.6	76	
BituTech PER Field					
5.48	2.643	5.6	17.1	67	0.77
BituTech PER Laboratory Verification					
4.98	2.645	9.1	19.7	54	
5.48	2.624	5.5	17.6	69	0.46
5.98	2.602	3.8	17.3	78	
6.48	2.581	2.0	16.9	88	
Cecabase RT Field					
5.66	2.621	3.0	15.7	81	0.55
Cecabase RT Laboratory Verification					
5.16	2.637	6.7	18.0	63	
5.66	2.616	4.7	17.4	73	0.50
6.16	2.595	1.8	16.0	89	
6.66	2.574	1.5	16.9	91	
SonneWarmix Field					
5.30	2.641	4.9	16.4	70	0.61
SonneWarmix Laboratory Verification					
4.80	2.656	7.5	17.8	58	
5.30	2.634	6.7	18.3	63	0.50
5.80	2.612	4.4	17.4	75	
6.30	2.591	3.3	17.5	81	

Table 214 Validation tests for SonneWarmix and Cecabase RT

Sieve Size	SonneWarmix					Cecabase RT		
	Field Mix	Trial 1	Trial 2	Trial 3	Trial 4	Field Mix	Trial 1	Trial 2
12.5 mm	100	99	99	100	100	100	100	100
9.5 mm	95	90	94	95	94	95	94	96
4.75 mm	62	54	62	63	61	61	59	62
2.36 mm	36	33	38	39	38	36	33	35
1.18 mm	25	23	27	27	26	26	24	25
0.60 mm	18	18	20	20	19	19	18	18
0.30 mm	13	12	15	14	13	13	12	13
0.15 mm	9	8	11	10	7	9	8	10
0.075 mm	6.1	5.9	8.6	7.3	5.7	6.3	6.3	6.8
Air Voids (%)	4.9	6.2	3.7	5.6	7.0	3.0	4.7	5.3
VMA (%)	16.4	17.8	15.6	17.2	18.5	15.7	17.4	17.8

Jefferson County, Florida

The final mix verification was performed on the U.S. 98 Terex foamed WMA. N_{design} was specified as 75 gyrations. The mix design used a polymer modified PG 76-22 binder. This initially caused clogging in the laboratory foaming device. Straining the binder prior to putting it into the foaming device appeared to prevent clogging (Figure 140).

Figure 140 Hydrofoamer (left), polymer strained out of Florida PG 76-22 binder (right)

Table 215 shows the JMF, measured field gradations and gradation checks of laboratory batched samples. Table 216 shows the volumetric properties at the asphalt contents used to bracket the field measured asphalt content. The field volumetric properties are also shown for comparison. The predicted optimum asphalt was the same for both the WMA and HMA. The optimum asphalt content determined from the mix verifications was less than the JMF even though the P_{be} values for the laboratory produced mix were higher than that observed in the field.

Table 215 Florida Design, Field, and Verification Gradations and Asphalt Contents

Sieve Size	JMF	HMA		Terex Foam	
		Lab	Field	Lab	Field
	Percent Passing				
25.0 mm	100	100	100	100	100
12.5 mm	100	100	100	99	99
9.50 mm	89	91	91	91	91
4.75 mm	63	63	64	63	63
2.36 mm	46	44	45	42	44
1.18 mm	35	33	34	31	33
0.60 mm	27	25	26	24	25
0.30 mm	15	16	15	14	14
0.15 mm	8	10	9	8	8
0.075 mm	5.4	4.8	5.5	4.9	4.8
AC (%)	5.30	5.01	5.33	5.01	4.95
Compaction Temp. (°F)		295		250	

Table 216 Summary of Florida Volumetric Properties

AC (%)	G_{mm}	Air Voids (%)	VMA (%)	VFA (%)	P_{ba} (%)
HMA Field					
5.33	2.542	1.9	13.1	86	0.76
HMA Laboratory Verification					
4.83	2.577	4.3	13.6	68	
5.33	2.557	3.3	13.8	76	1.02
5.83	2.537	1.6	13.4	88	
Terex Foam Field					
4.95	2.556	3.4	13.6	75	0.74
Terex Foam Laboratory Verification					
4.95	2.568	4.2	13.9	70	
5.45	2.548	2.7	13.7	80	0.94

Summary Comparisons

The previous section presented the field and laboratory volumetric properties on a project by project basis. This section presents overall comparisons.

Maximum specific gravity tends to be a repeatable test. Maximum specific gravity is, however, sensitive to differences in mixture aging and binder absorption. Figure 141 shows field to laboratory comparisons for all of the mixtures evaluated. The comparisons were made at the field measured asphalt content. All of the laboratory samples were aged for two hours at the field compaction temperature. The whisker bars in the figure show the AASHTO T209 multi-laboratory d2s. All of the differences are well within the multi-laboratory d2s. With the exception of the Michigan project, all of the differences are in one direction, e.g., either all of the field results are higher or all of the laboratory results are higher.

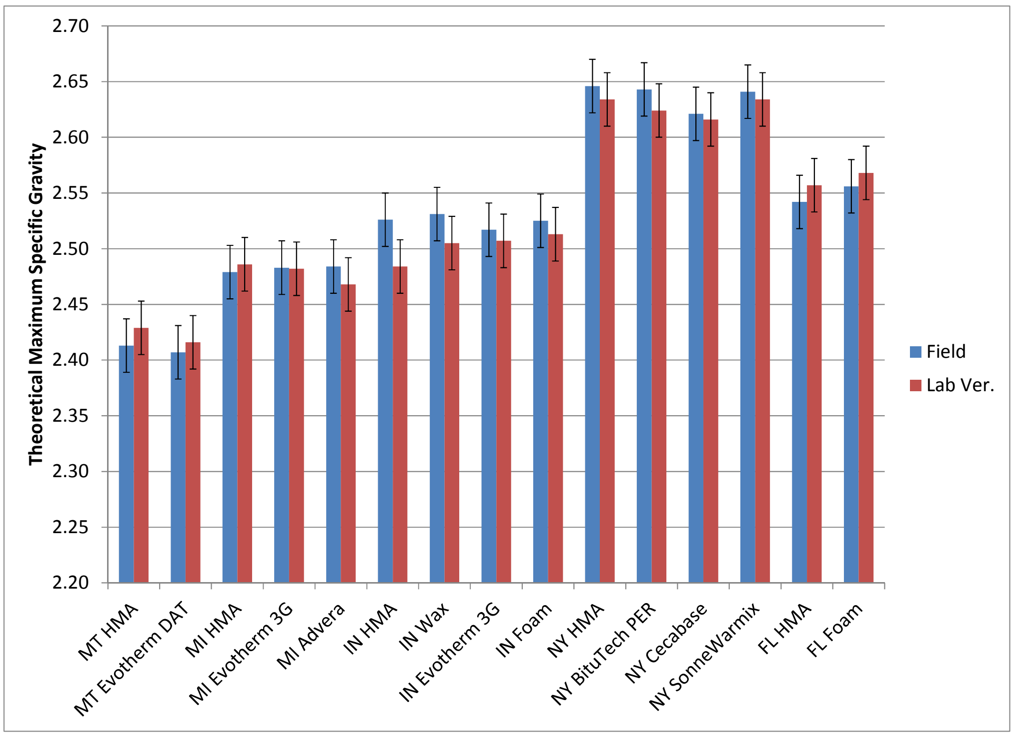


Figure 141 Comparison of Maximum Specific Gravity for Verification Mixtures

Percent binder absorption (P_{ba}) is calculated using the aggregate bulk (G_{sb}) and effective (G_{se}) gravities. The effective gravity is backcalculated using the mixtures maximum specific gravity and asphalt content. Therefore, differences in maximum specific gravity will affect the reported P_{ba}. Figure 142 shows the difference between the field and laboratory P_{ba}. With the exception of the Michigan data, the differences correspond to the differences in maximum specific gravity, e.g. higher maximum specific gravity equates to higher binder absorption. Figure 143 shows the difference between the WMA and HMA binder absorption for each project/mixture. As expected, WMA generally results in reduced binder absorption.

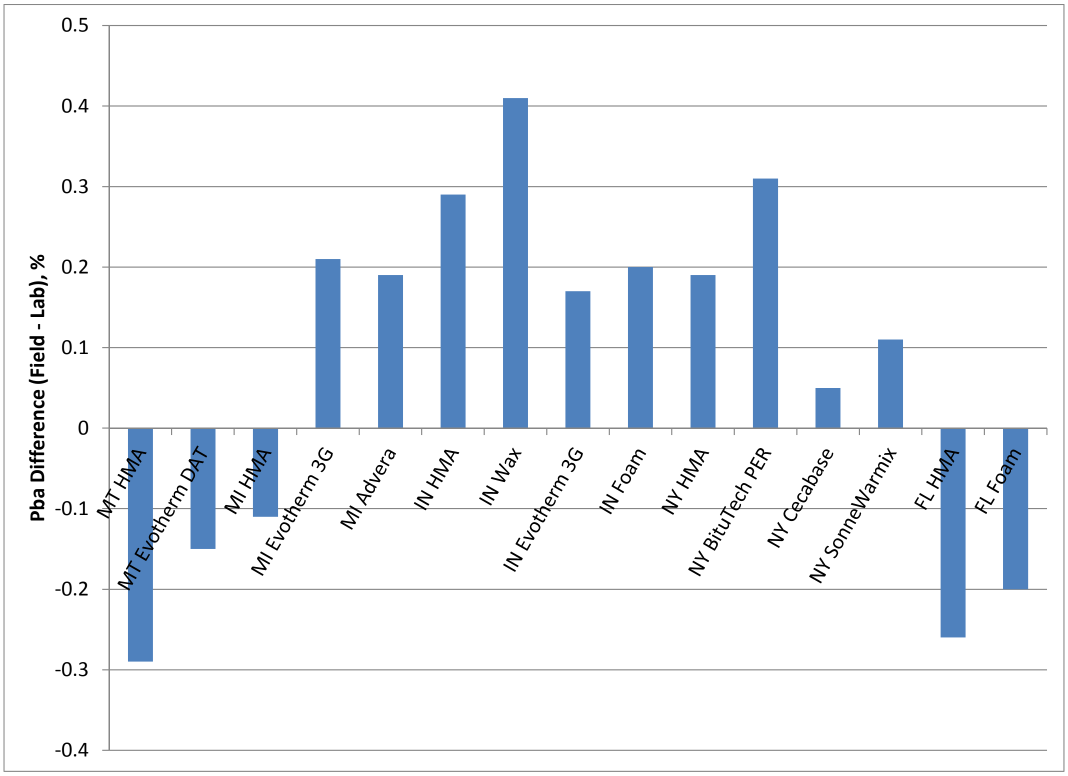


Figure 142 Difference Between Field and Laboratory Binder Absorption

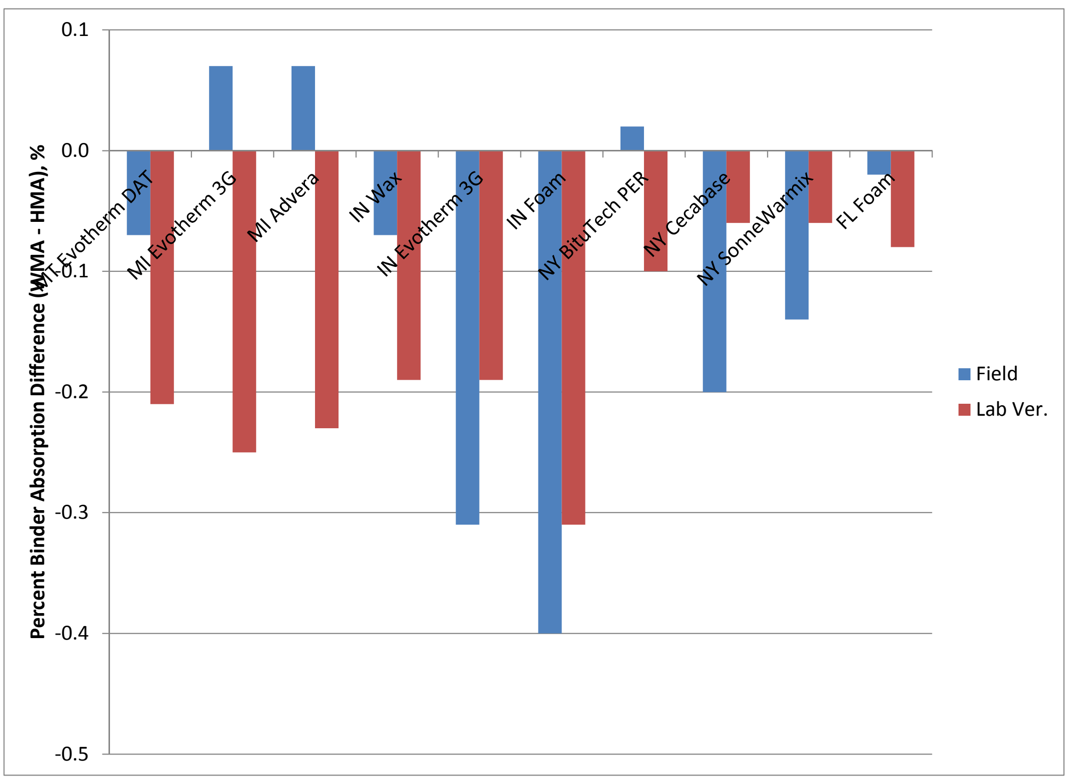


Figure 143 Difference Between HMA and WMA Binder Absorption

Ideally, the laboratory design should be able to replicate the field produced material in terms of volumetric properties. Differences in gradation can lead to differences in volumetric properties and the JMF is not always reproduced in the field. As noted previously, the laboratory verifications attempted to match, as closely as possible, the gradation of the field sample. Figure 144 shows the differences between the field and laboratory air voids. The AASHTO T312 multi-laboratory d2s for relative density (and therefore air voids) is 1.7 percent. Only one mix, the New York SonneWarmix, exceeded this limit. Additional testing with alternate gradation adjustments were presented in Table 214.

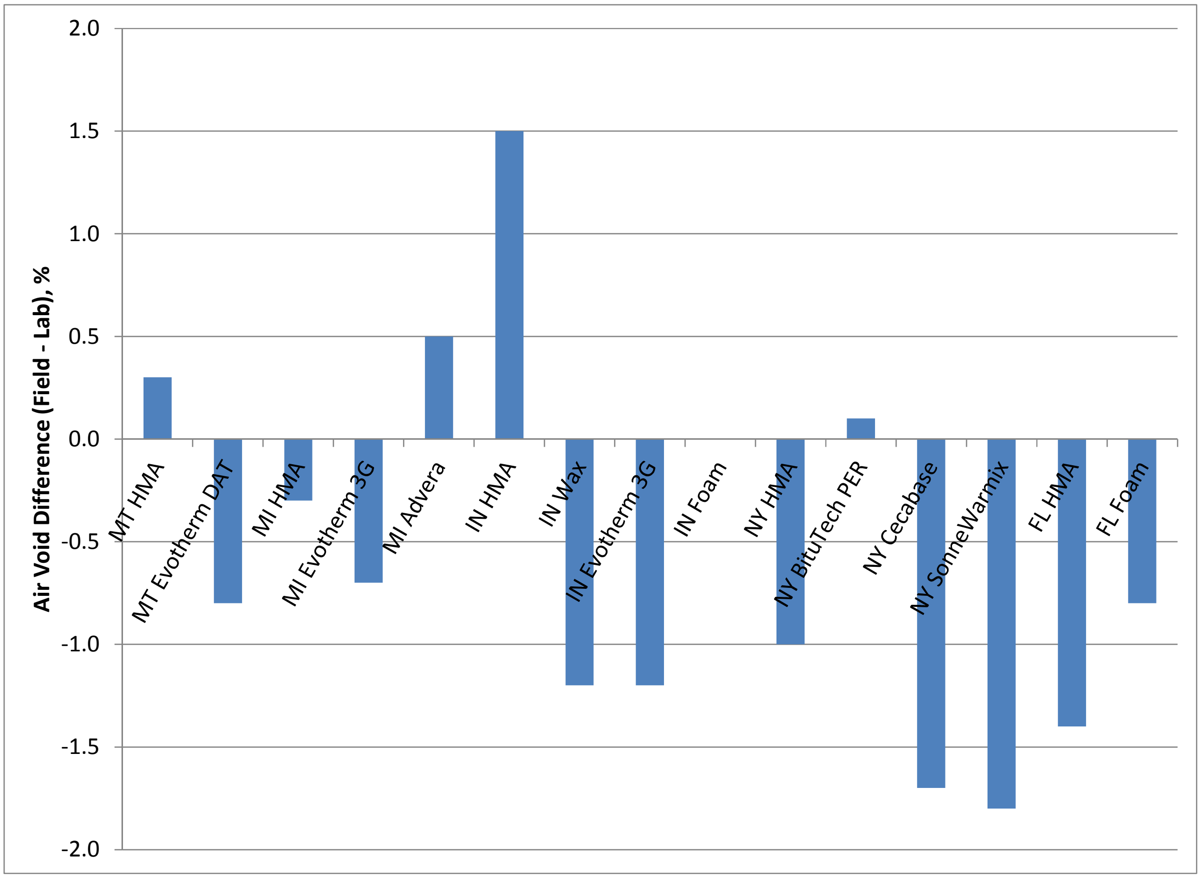


Figure 144 Difference between voids of field and laboratory produced mix.

One method of producing WMA is to foam the binder. Early drum plants reportedly used lower temperatures, resulting in incomplete drying of the aggregate and a degree of binder foaming. If the aggregate particles are coated before they are completely dry, heat transfer would tend to result in a degree of foaming with time. Essentially, this is the process used to produce low emission asphalt. Laboratory mix designs are produced using oven dry aggregates. Typical water addition rates for foaming are 2 percent by weight of binder. If there is 5 percent binder by total weight of mix, this would result in a mix moisture content of 0.1%. If mix moisture is producing a degree of foaming of the binder in the field, then this may explain part of the difference between laboratory and field air voids. Figure 145 shows field mix moisture contents versus the difference between field and laboratory void contents. There is an overall, albeit very poor, trend of higher laboratory versus field air voids with higher field mix moisture contents. Some of the larger differences occurred with the Munster, IN mixes using higher water absorption aggregates and with the New York, NY mixes that contained 25 percent RAP, both of which may contribute to higher mix moisture contents.

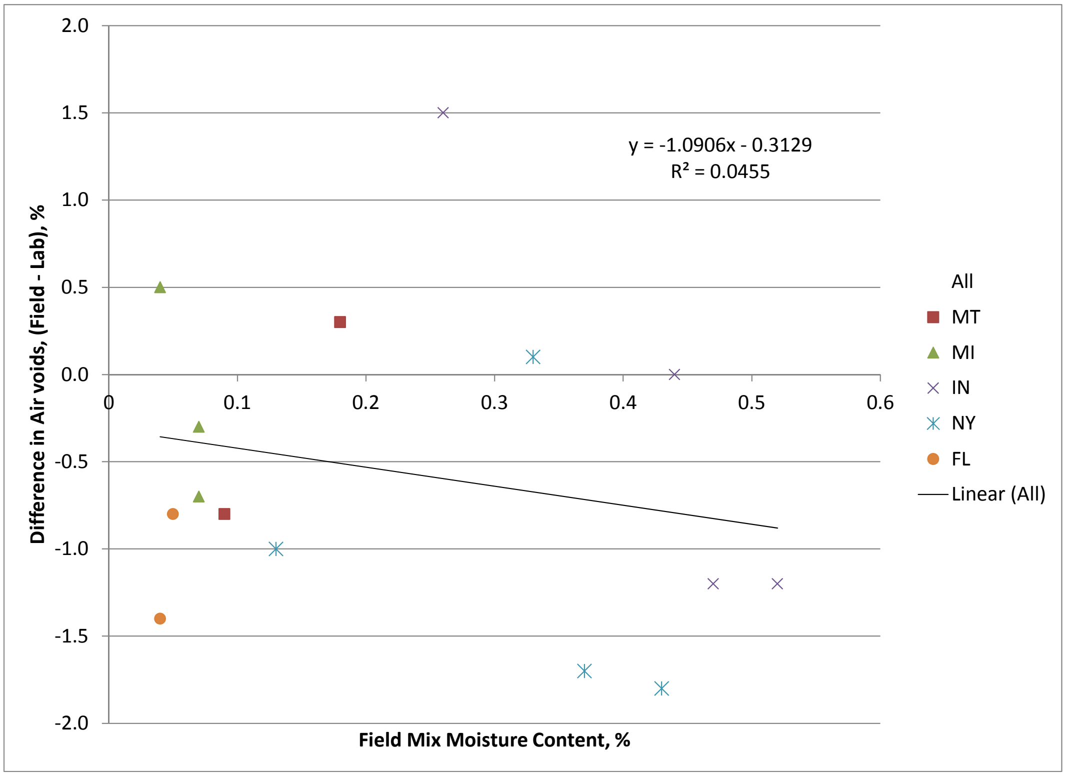


Figure 145 Field Mix Moisture Content Versus Air Void Content Difference

Figure 146shows the difference between the WMA and HMA optimum asphalt content for each project. It should be noted that differences may exist between the target gradation for the HMA and WMA. In six of ten cases the optimum asphalt content for the WMA was less than that for the HMA. The decrease ranged from -0.24 to -0.92 percent. The overall average difference (including the increases) was -0.27 percent. Table 217 shows both the contractor's optimum asphalt content based on the JMF and the laboratory verified optimum asphalt content. In this case, six of ten comparisons result in higher optimum asphalt contents for the WMA than what was reported on the JMF.

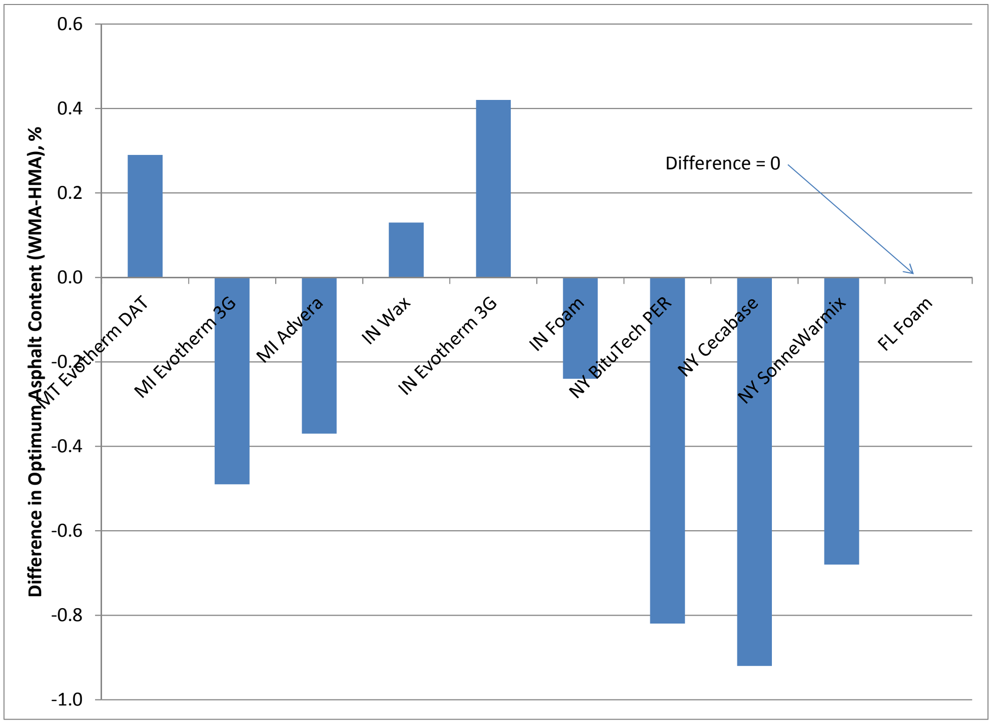


Figure 146 Comparison of WMA and HMA Optimum Asphalt Contents

Table 217 Reported and Verified Optimum Asphalt Contents

Project	Mix Type	Asphalt Content, %		Compaction Temperature, °F
		JMF	Lab Verified	
Michigan	Advera	5.30	4.95	250
	Evotherm 3G		4.83	250
Montana	Evotherm DAT	5.80	5.76	235
Indiana	Wax	5.50	6.40	240
	Evotherm 3G		6.69	230
	Foam		6.03	240
New York	Bitutech PER	5.30	6.06	225
	Cecabase		5.96	225
	SonneWarmix		6.20	225
Florida	Foam	5.30	5.01	250

Coating

Conventional HMA mix designs use equiviscous mixing and compaction temperatures based on rotational viscosity tests. Most WMA technologies cannot be adequately evaluated using this method. The NCHRP Project 9-43 research team proposed mixture tests as surrogates. These tests do not determine the appropriate mixing and compaction temperature, but rather evaluate whether or not the proposed temperature is adequate. The test used to evaluate the suitability of the mixing temperature is based on the coating of the aggregates with asphalt binder following the normal laboratory mixing process.

Once the laboratory optimum asphalt content was determined, mixture coating was evaluated using the AASHTO T195 Ross Count procedure. Samples were mixed for 90 seconds as specified in the Appendix to AASHTO R35. As noted previously, a more commonly available bucket mixer was used to prepare the samples rather than a planetary mixer. As can be seen in Table 218, this equipment generally produced coating results that were similar to the degree of coating achieved in field.

Table 218 Percent Coating for WMA

Project	Mix Type	Asphalt Content, %	Mixing Temp., °F	Coating, %	
				Field	Lab
Michigan	ADVERA	5.34	275	100.0	98.5
	Evotherm 3G	5.00	275	99.6	100.0
Montana	Evotherm DAT	5.80	250	98.8	98.5
Indiana	Wax	6.40	270	98.0	100.0
	Evotherm 3G	6.69	255	99.0	100.0
	Gencor Foam	6.03	275	99.0	96.0
New York	Bitutech PER	6.06	280	99.5	100.0
	Cecabase RT	5.96	250	100.0	100.0
	SonneWarmix	6.20	260	99.5	100.0
Florida	Terex Foam	5.01	300	99.0	97.0

Note: Appendix to R35 requires a minimum of 95 percent coating

Compactability

To evaluate the proposed WMA compaction temperature, the Appendix to AASHTO R 35 specifies that the ratio of the number of gyrations to 92 percent density at 30°C (54°F) below the proposed compaction temperature to the number at the proposed compaction temperature must be less than 1.25. Two sets of mix samples are mixed and aged at the same temperature, then one set is allowed to cool prior to compaction.

Table 219 shows the optimum asphalt content at which each mixture was tested, the difference between the optimum asphalt content of that mixture and the HMA control based on the laboratory mix design verification, the laboratory compaction temperature, the compactability ratio, and the average in-place density based on the field cores.

Six of ten WMA mixes failed the specified compactability ratio. Two of the six mixtures that failed the compactability ratio had optimum asphalt contents that were higher than the control. Four out of six mixes which failed the compactability ratio had in-place densities less than 92 percent. By comparison, two of four mixtures which passed the compactability ratio had in-place densities less than 92 percent. Higher optimum asphalt contents than that for the corresponding HMA were indicated for three of five mix with low in-place density. This may indicate that too low of a compaction temperature was selected for these mixes. The difference may also have resulted from differences in gradation.

Table 219 Gyratory Compactability Ratios

Project	Mix Type	Asphalt Content, %	Diff. HMA and WMA Optimum AC%	Lab Compaction Temp. °F	Compact-ability Ratio	Average In-place density, %
Michigan	Advera	4.95	-0.49	250	1.34	95.0
	Evotherm 3G	4.83	-0.37	250	0.92	94.3
Montana	Evotherm DAT	5.76	0.29	235	2.22	91.2
Indiana	Wax	6.40	0.13	240	1.31	88.7
	Evotherm 3G	6.69	0.42	230	1.21	90.4
	Gencor Foam	6.03	-0.24	240	2.44	90.3
New York	Bitutech PER	6.06	-0.82	225	1.35	92.4
	Cecabase	5.96	-0.92	225	1.11	92.1
	SonneWarmix	6.20	-0.68	225	1.17	89.9
Florida	Terex Foam	5.01	0.00	250	1.64	92.1

Moisture Susceptibility

As with all Superpave mix designs, the Appendix to AASHTO R 35 specifies the tensile strength ratio (TSR) test according to AASHTO T 283 for WMA mix designs. The tests were conducted at the optimum asphalt content as determined in the laboratory mix design verification. Figure 147 shows a comparison of the TSR results from the field and laboratory produced mixes. There was good agreement between the field and laboratory results for six of the ten mixes. Both Michigan WMAs had substantially lower TSR values for the laboratory produced mixes, but the laboratory verified optimum asphalt contents were also lower for these mixes. The Indiana wax WMA also showed a lower TSR during the laboratory verification. Both the unconditioned and conditioned tensile strengths were higher for the field produced Indiana wax mix.

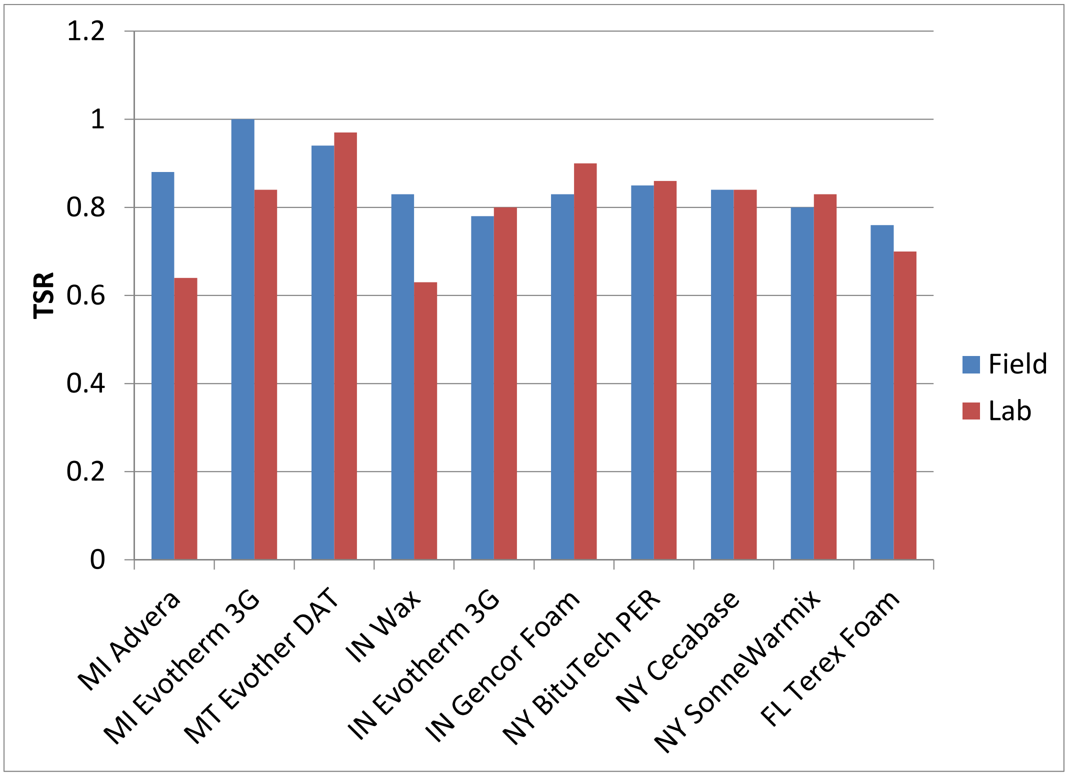


Figure 147 Comparison of Field and Laboratory TSR Values

Flow Number Test

WMA samples were prepared by AMS for flow number (FN) testing according to AASHTO PP 60 at the optimum asphalt content determined in the mix design verification. FN tests were performed by NCAT in the Asphalt Mixture Performance Tester according to AASHTO TP 79. The draft Appendix to AASHTO R 35 provides minimum FN requirements based on the 20-year design equivalent single axle loads (ESALs). The average FN for the WMA mixes tested, 20-year design ESALs, and FN criteria are shown in Table 220. At the optimum asphalt content determined from the mix verifications, all of the mixes except the Munster, Indiana Evotherm met the minimum FN requirements provided in the Appendix to AASHTO R35. After two-years of service, no rutting was observed in the field for the Indiana Evotherm section, although that section was placed in the passing lane and may have received lower traffic.

Table 220 Mix Verification Flow Number Results

Project	Mix	20-Year Design ESALS	R35 Appendix FN Criteria	Average FN
MI	Advera	225,355	NA	78
	Evotherm			66
MT	Evotherm DAT	242,990	NA	29
IN	Heritage Wax	10,499,416	105	144
	Evotherm 3G			64
	Gencor Foam			156
NY	SonneWarmix	8,251,905	30	67
	BituTech PER	6,040,268		49
	Cecabase			75
FL	Terex Foam	3,061,037	30	49

Table 221 shows a comparison of the laboratory- and field-mixed FN results. Since the laboratory produced samples were prepared at the optimum asphalt content determined from the mix verifications, differences in asphalt content as well as potential differences in aging effect the comparisons of the laboratory and field produced mix results. Some of the field produced mix was compacted in the field without reheating. Other samples were prepared from reheated field mix. These are noted in **Table 221**. There is little or no difference in the asphalt contents of the laboratory and field samples for Montana and Florida. In both cases, the field produced mix resulted in significantly larger FN. Asphalt contents were reduced for both Michigan laboratory produced mixes. The FN for the laboratory produced mixes are only higher than the FN for the field produced for the two Michigan WMA mixes. The field produced Indiana Evotherm mix, produced at a lower asphalt content, meets the Appendix to AASHTO R 35 FN criteria.

Table 221 Comparison of Laboratory and Field Produced FN Results

Proj.	Mix	Diff. Lab - Field AC%	Lab Avg.	Lab Std. Dev.	Field Avg.	Field Std. Dev.	F-Test Equal Variances Y or N	*t*-test 2 Tail *p-value*	*t*-test Signif.? Y or N
Field FN samples field compacted without reheating									
IN	Heritage Wax	0.45	144	38	314	39	Y	0.006	Y
	Evotherm	0.74	64	6	177	6	Y	0.000	Y
	Gencor Foam	0.42	156	2	217	4	Y	0.000	Y
NY	SonneWarmix	0.90	67	4	123	17	Y	0.005	Y
	BituTech PER	0.58	49	3	128	12	N	0.008	Y
	Cecabase	0.30	75	12	115	3	N	0.031	Y
FL	Terex Foam	0.06	49	3	157	12	Y	0.005	Y
Field mix reheated to prepare FN samples									
MI	Advera	-0.39	78	31	60	1	Y	0.423	N
	Evotherm	-0.17	66	7	65	11	N	1.000	N
MT	Evotherm	0.00	29	10	58	2	Y	0.022	Y
FL	Terex Foam	0.06	49	3	127	20	N	0.021	Y

PROPOSED REVISIONS TO THE DRAFT APPENDIX TO AASHTO R35

Based on the results of these mix verifications, the following revisions to sections 3, 7, and 8 of the Draft Appendix to AASHTO R 35 developed in NCHRP Project 9-43 *(21)* are proposed for consideration by the AASHTO Subcommittee on Materials.

3. ADDITIONAL LABORATORY EQUIPMENT

3.1.1 Mechanical mixer

Note 1 should be eliminated. Ten mix design verifications were performed as part of NCHRP Project 9-47A. A bucket mixer was used to prepare the mixes. In all cases, the laboratory produced mix exceeded the minimum 95 percent coating recommended in the Draft Appendix using the recommended 90 second mixing time. The two laboratory foam mixes had lower percent coatings than the field mix (average 2.5 percent less).

3.3.1 Laboratory foamed asphalt plant

Add the following paragraph to the end of the current language: “In lieu of a laboratory foamed asphalt plant, a trial batch or run may be produced at the asphalt plant. When producing a trial batch or run of WMA, it is recommended that the plant level out its production with HMA, then begin the water injection process and decrease the mixing temperature to the desired WMA production temperature. Once the desired WMA temperature is reached, obtain samples for testing."

Commentary

Full-scale asphalt plant foaming systems appear to provide better mixing and coating than laboratory-scale plants. Commercially available laboratory-scale foaming units use timers to control the amount of foam produced. The NCHRP Project 9-47A team has utilized two of the three commercially available units; the NCHRP Project 9-43 team used the third unit. This experience suggests that the laboratory systems do not control the amount of binder foam accurately enough for mix design purposes. Therefore, when using laboratory asphalt foaming systems, the binder needs to be foamed into a separate, pre-heated container and then weighed into the batch on an external scale. The container should be pre-heated to the mixing temperature to minimize foam collapse. Once the foam is weighed into the batch, the bucket or mixing bowl is immediately placed into the mixer and mixing started. The half-life of binder foam, or time it takes for the volume of foam to reduce by half, is typically measured in seconds. The delay caused by weighing on a separate scale instead of foaming directly into the moving mixer appears to reduce the effectiveness of the foaming.

Problems occurred when using D&H's Hydrofoamer (marketed by InstroTek as the AccuFoamer) with polymer modified PG 76-22 binder. Small particles of polymer or asphalt repeatedly clogged the binder nozzle going into the foaming chamber. These particles may have resulted from reheating the binder in gallon-cans. The problem could be reduced by straining the binder when pouring it into the Hydrofoamer. The straining is not expected to affect the binder grade.

7. PROCESS SPECIFIC SPECIMEN FABRICATION PROCEDURES

Volumetric Design

Section 7 describes procedures for replicating various types of WMA in the laboratory. Table 2 of Section 7 provides approximate specimen mass for volumetric design specimens. However, the appendix does not specifically state that the volumetric design should be conducted using laboratory produced WMA. The findings from NCHRP Project 9-47A suggest the volumetric design should first be completed as described in AASHTO R 35 **without the WMA additive/technology** and then the additional performance checks, coating, compactability, moisture sensitivity, and rutting resistance (if required) should be completed using laboratory produced (or in certain cases plant produced) WMA.

In production, contractors could make slight adjustments to the target asphalt content, consistent with current state practices, to ensure acceptable air voids. The field produced WMA would need to meet the minimum production VMA requirement, also consistent with current state practice.

Commentary

The following describes the findings relative to this proposed change.

NCHRP Project 9-47A evaluated 13 WMA mixtures sampled from 8 different projects. In all cases, the WMA technologies were "dropped" into existing HMA designs. Ten mix design verifications from five projects were performed using the procedures outlined in the Draft Appendix to AASHTO R 35. When performing the mix verifications, the research team tried, as closely as possible, to match the field measured gradation for a particular mix. The optimum asphalt content of the comparable HMA control was verified in the same manner. Using the Draft Appendix to AASHTO R 35 for the WMA mix design verifications, the optimum asphalt content decreased, on average, by 0.27 percent for WMA compared to the respective HMA, with a range of 0.42 percent increase to 0.92 percent decrease.

Several factors could justify lower asphalt contents for WMA:

1. The binder absorption of WMA is less than for HMA produced with the same aggregate blend.
2. WMA mixes densify to less than 4 percent air voids in the wheelpath,
3. WMA mixes are prone to rutting or bleeding in the field, suggesting they are over asphalted.

Binder Absorption: For the field produced mix, sampled and tested at the asphalt plant without reheating, the binder absorption of the WMA averaged 0.11 percent less than for the comparable HMA produced with the same aggregate blend. The difference in measured absorptions ranged from 0.07 percent greater to 0.40 percent less. For the laboratory mix produced according to AASHTO R 35, the binder absorption averaged 0.17 percent less for the WMA compared to the HMA. Table 222 presents the binder absorptions measured for each mix in the laboratory verifications, field mix sampled at the plant, and one- and two-year cores. Both the laboratory verifications and field mix indicate slightly lower binder absorptions for the WMA (approximately 0.2 and 0.1 percent, respectively). However, this difference is not apparent in the one- or two-year cores, indicating that after latent absorption, the mixes are equal. The two exceptions are the one-year results for New York, NY BituTech PER and Casa Grande, AZ Sasobit. The difference was not apparent in the two-year BituTech PER cores. Since the binder absorptions calculated for Casa Grande, AZ field mix were almost identical, this exception may be due to experimental error. Overall, this suggests that the binder content of WMA mixes should not be reduced to account for reduced absorption.

Pavement Densification: Pavements densify under traffic after construction. In theory, pavements are designed to reach an ultimate density of 96 percent of G_{mm} (4 percent air voids). For HMA pavements, the majority of the densification occurs in the first year after construction with the ultimate density being obtained after two years of traffic *(1)*. Table 223shows the average core density at the time of construction and after one and two years of traffic. The one-and two-year core data presented here were taken from the wheelpath. With two exceptions, New York SonneWarmix, and Florida Terex Foam, the same or higher in-place densities were obtained with the WMA at the time of construction. However, in only three cases, New York BituTech PER, New York SonneWarmix, and Florida Terex Foam, do the two-year WMA cores have higher densities than their HMA counterparts. All of these differences are less than 1 percent density. The one-year Arizona Sasobit cores also have higher density than the HMA. The fact that the WMA and HMA are densifying to the same levels suggests that the WMA mixes are not over- or under-asphalted compared to the HMA when using the drop-in approach to WMA mix design.

Rutting Potential: Although some laboratory tests indicate otherwise, WMA pavements constructed to date, including accelerated test sections at the NCAT Test Track and the University of California Pavement Research Center, have been rut resistant. The same holds true for the NCHRP Project 9-47A field test sections**.** Table 223shows the average rut depth measured after one- and two- years. The rut depths for the WMA and HMA sections are negligible and approximately equal. Based on the rutting performance observed to date, there is no need to reduce the asphalt content of WMA mixes.

Table 222 Comparison of WMA and HMA Binder Absorptions

Project Location	Technology	Avg. WMA Temperature, °F			HMA Field Comp. Temp. °F	Binder Absorption (%)							
		Field Mixing	Field[1] Comp.	Lab		Lab Verifications		Field Mix		1-Year Cores		2-Year Cores	
						WMA	HMA	WMA	HMA	WMA	HMA	WMA	HMA
Walla Walla, WA	Aquablack	285	270	NA	310	NA	NA	0.63	1.15	1.40	1.40	1.28	1.03
Centreville, VA	Astec DBG	288	268	NA	294	NA	NA	0.92	0.88	0.91	0.61	0.61	0.78
Rapid River, MI	Evotherm 3G	269	239	250	255	0.45	0.70	0.66	0.59	1.01	0.88	0.91	0.78
Rapid River, MI	Advera	269	227	250	255	0.47	0.70	0.66	0.59	1.04	0.88	0.97	0.78
Baker, MT	Evotherm DAT	262	NA	235	282	0.80	1.01	0.65	0.72	0.75	0.87	0.72	0.53
Munster, IN	Wax	268	235	240	249	1.10	1.29	1.51	1.58	1.26	1.29	1.49	1.55
Munster, IN	Gencor foam	277	222	240	249	0.98	1.29	1.18	1.58	1.48	1.29	1.48	1.55
Munster, IN	Evotherm 3G	256	210	230	249	1.10	1.29	1.27	1.58	1.39	1.29	1.53	1.55
New York, NY	BituTech PER	279	238	225	299	0.46	0.56	0.77	0.75	0.50	0.70	0.75	0.71
New York, NY	Cecabase	247	221	225	299	0.50	0.56	0.55	0.75	0.67	0.70	0.68	0.71
New York, NY	SonneWarmix	262	222	225	299	0.50	0.56	0.61	0.75	0.71	0.70	0.66	0.71
Jefferson Co., FL	Terex foam	297	247	250	269	0.94	1.02	0.74	0.76	0.84	0.77	0.77	0.77
Casa Grande, AZ	Sasobit	276	257	NA	297	NA	NA	0.62	0.64	0.27	0.51	-	-

NA = Not Tested; Casa Grande 2-Year Cores not collected.

[1] Where possible, based on average temperature recorded by PAVE-IR system.

Table 223 WMA and HMA Pavement Densification and One-Year Rut Depths

Project Location	Technology	In-Place Density, % G_{mm}						Avg. Rut Depth, mm			
		Construction Cores		1-Year Cores		2-Year Cores		1-Year		2-Year	
		WMA	HMA	WMA	HMA	WMA	HMA	WMA	HMA	WMA	HMA
Walla Walla, WA	Aquablack	94.4	94.7	95.4	96.2	95.9	96.6	0.00	0.99	0.31	4.59
Centreville, VA	Astec DBG	89.9	89.1	94.2	94.2	93.9	94.0	0.00	0.00	2.65	3.18
Rapid River, MI	Evotherm 3G	94.3	94.1	97.1	97.8	96.5	97.4	0.00	0.00	0.00	0.00
Rapid River, MI	Advera	95.0	94.1	95.8	97.8	96.6	97.4	0.00	0.00	0.00	0.00
Baker, MT	Evotherm DAT	91.2	91.3	95.0	93.8	94.5	94.7	0.18	0.35	0.18	0.52
Munster, IN	Wax	88.7	88.7	92.8	94.0	93.1	94.6	0.00	0.00	0.00	0.00
Munster, IN	Gencor foam	90.3	88.7	93.7	94.0	93.5	94.6	0.00	0.00	0.00	0.00
Munster, IN	Evotherm 3G	90.4	88.7	92.9	94.0	93.0	94.6	0.00	0.00	0.00	0.00
New York, NY	BituTech PER	92.4	90.8	95.1	94.7	96.5	95.7	0.67	1.00	2.65	1.85
New York, NY	Cecabase	92.1	90.8	93.8	94.7	95.0	95.7	0.33	1.00	0.33	1.85
New York, NY	SonneWarmix	89.9	90.8	95.7	94.7	96.5	95.7	0.00	1.00	0.00	1.85
Tallahassee, FL	Terex foam	92.1	93.0	91.6	93.0	90.9	90.4	2.44	1.87	3.02	2.93
Casa Grande, AZ	Sasobit	92.4	90.6	95.1	94.6	NA	NA	0.00	3.18	NA	NA

Interaction with Compactability: Based on the Draft Appendix to AASHTO R35, after the optimum asphalt content is determined, coating and compactability are evaluated at the proposed mixing and compaction temperatures. As noted previously, the optimum asphalt content of the WMA mixes decreased, on average, by 0.27 percent. While this did not affect the coating, it does appear to have an effect on compactability. Figure 148 shows the Superpave gyratory compactor (SGC) compactability ratio, described in the Draft Appendix to AASHTO R35 versus the average in-place density achieved at the time of construction. The diamonds represent the compactability ratio measured at the optimum asphalt content determined according to AASHTO R35. Based on the data determined at optimum asphalt content, there appears to be a poor relationship between compactability ratio and the density achieved in-place. The compactability ratio was measured again for four mixes at the asphalt content measured in the field. These data are indicated by the squares and shows lateral shifts in the compactability ratio. Where the asphalt content decreased by 0.74 and 0.90 percent, the compactability ratio increased; where the optimum asphalt content increased by 0.39 percent, the compactability ratio decreased, both as expected. A sample tested with a 0.17 percent increase in optimum asphalt content showed essentially no change in compactability ratio. This data suggests that field compactability is related to the asphalt content of the mixture. WMA is a compaction aid. If the optimum asphalt content of WMA mixes is decreased, the compaction benefits may be nullified.

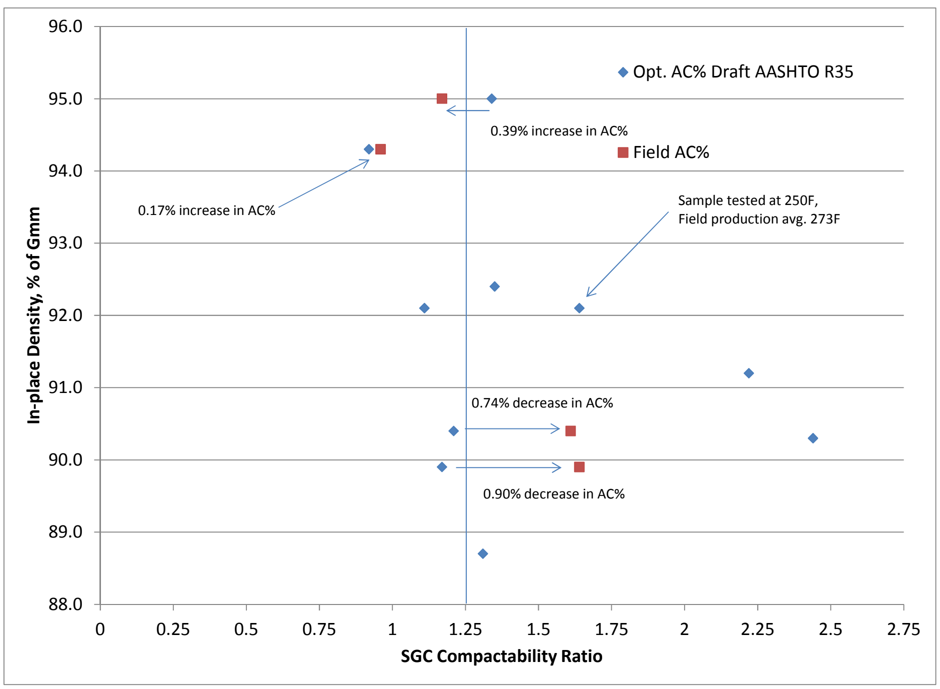


Figure 148 SGC compactability ratio versus achieved in-place density

WMA MIXTURE EVALUATIONS

Evaluating Moisture Sensitivity

Some WMA technologies contain anti-stripping additives. Others may affect the asphalt aggregate interaction. Therefore, moisture sensitivity should be evaluated at the optimum asphalt content determined in a mixture using the WMA technology. In the case of mechanical foaming technologies in particular, it may be advantageous to test WMA produced through the asphalt plant (trial batch).

Evaluating Rutting Resistance

The rutting performance of field WMA projects to date does not seem to justify additional testing not required for HMA. Therefore, flow number test requirements should be eliminated except for traffic levels in excess of 30 million ESALs. If the agency already requires performance tests for HMA, than these same tests should be applied to WMA with the understanding that different aging conditions or test criteria may be required.

CHAPTER 6

COST ANALYSIS OF WMA

Economics of a new technology, such as WMA, is often one of the principle factors that determine its acceptance into mainstream practice. In a permissive specification environment, such as for WMA in most cases, it is probably the dominant factor. For the asphalt contracting industry, the use of WMA has certain costs and potentially some economic benefits. The costs of WMA depend primarily on the type of WMA technology that is used. Economic benefits may be related to energy reductions at the plant, the potential for higher unit payments resulting from achieving higher in-place densities or smoother pavements, extended paving seasons, and the possibility of eliminating anti-stripping additives for some WMA additives.

Water-injection asphalt foaming systems typically have the lowest cost per ton of the WMA technology options. These systems require the installation of mechanical equipment and some modifications to the plant's control system. The early water-injection foaming systems cost around \$80,000. Other water-injection foaming systems that have entered the marketplace in the last few years cost as little as \$30,000 installed. Many contractors depreciate capital expenditures such as this over five to seven years. Assuming an average yearly production for a plant, the cost of the equipment can also be figured in a per ton basis. For example, if the water-injection foaming system cost \$50,000 and the plant produces an average of 120,000 tons per year, then depreciating the system over five years would add about 8¢/ton [\$50,000/(5×120,000) = \$0.08].

WMA additives are reported to increase mix costs by approximately \$2.00 to \$3.50/ton (*32*). Additive prices will also vary due to freight costs. WMA additive prices may have decrease some of the past few years as the addition of WMA additives at asphalt terminals has become more common.

Mix design costs are also likely to increase if the recommendations from NCHRP Report 691 are implemented. Adding the coating test, compactability test, and flow number test are estimated to increase mix design costs by \$1,500 to \$2,000.

As reported in Volume II, the energy audits for WMA projects in this study found energy savings for WMA production to be reasonably approximated by the following relationship:

$$\text{Energy savings} = 1100\ \text{BTU/°F/ton} \qquad (3)$$

Although theoretical energy calculations indicate that the reduction should be less than the result determined from Equation 3, the theoretical models do not appear to fully account for the energy transfer to heating the metal in the plant's drier and ductwork.

In practice, WMA production temperatures when using water-injection foaming technologies are typically about 25°F lower than HMA using the same mix design. WMA produced with additives tend to have substantially lower mixing temperatures. For the purpose of estimating energy savings, a temperature difference of 50°F is assumed for additive type WMA compared to HMA using the same mix design. Therefore, for water-injection type WMA, typical energy savings can be estimated to be 27,500 BTU/ton, and for additive type WMA, the energy savings can be estimated to be about 55,000 BTU/ton.

Most asphalt plants in the United States use either recycled fuel oil (RFO) or natural gas for burner fuel for drying and heating the aggregate. A typical energy density for RFO is 137,000 BTU per gallon (*33*). Recent cost for RFO is about \$2.00/gallon (*34*). Therefore, as shown below, for a 25°F drop from HMA to WMA for typical water-injection systems, the energy savings when using RFO is estimated to be \$0.39/ton of mix.

27,500 BTU/ton × 1 gallon of RFO/137,000 BTU × \$2.00/gallon = \$0.39/ton

Similarly, for a 50°F drop from HMA to WMA, the energy savings is estimated to be \$0.79/ton of mix.

In 2013, natural gas prices ranged from approximately \$4.30 to \$5.25 per million BTU (*35*). Adding approximately \$1/MMBTU for transportation and the supplier's overhead and profit, a contractor's cost for natural gas is estimated to be \$5.78 per million BTU. Therefore, for a 25°F drop from HMA to WMA, the energy savings when using natural gas is estimated to be \$0.16/ton of mix.

27,500 BTU/ton × \$5.78/1,000,000 BTU = \$0.16/ton

Similarly, for a 50°F drop from HMA to WMA using natural gas, the energy savings is estimated to be \$0.31/ton of mix.

Feedback from a few contractors who have monitored their plant's energy usage with and without WMA have indicated that their fuel savings is similar to the estimated values given above. A common response from contractors using water-injection foaming systems is that the energy savings is about 10% when using WMA. Based on this information, the estimated energy savings per ton for RFO-fueled plants would be about \$0.39, and for natural gas-fueled plants the savings are estimated to be about \$0.16/ton.

Other potential economic benefits to contractors using WMA could include higher pay per unit price based on incentive/disincentive specifications for in-place density and smoothness. Improving in-place density is a key to better pavement performance. Data this study showed that on a project by project basis, post-construction density for WMA pavements were not statistically different than HMA pavements with the same mix design. However, the difference may still be significant from a practical perspective. On average, the density improvement for WMA compared to HMA was 0.17% of G_{mm}. An analysis of the potential financial gain from a 0.17% higher density was conducted for a set of six randomly selected projects using a percent within limits (PWL) incentive/disincentive specification. The Florida DOT's PWL specification used in this example allows each lot of mix to receive up to a 5% bonus or a penalty as low as 80% of the bid price depending on the PWL results. In Florida, in-place density is one of four parameters used in the calculation of the composite pay factor for each lot. Density has weighting factor of 0.35, the highest of the four pay items used in the calculation of the composite pay factor. A typical bid price of $85/ton was used in this analysis. FDOT provided in-place density test results from the six randomly selected projects across the state. A summary of the project information and the results of the hypothetical analysis are shown in Table 224. To simplify the analysis, partial lots were excluded.

Table 224 Hypothetical Impacts of WMA on Density Pay Factors and Mix Savings

Project	Project Tons*	Actual Average Density Pay Factor	Adjusted Average Density Pay Factor	Hypothetical Savings $/ton
1	64,000	0.94	0.97	$1.13
2	108,000	0.94	0.96	$0.51
3	48,000	1.05	1.05	$0.00
4	92,000	1.03	1.03	$0.09
5	75,000	1.01	1.02	$0.25
6	92,000	0.87	0.91	$1.10

*partial lots were not evaluated

It can be seen that Project 3 achieved the highest possible pay factor for density on all lots, so there was no opportunity for a financial benefit for achieving higher density by using WMA on that project. Project 4 also had a high average pay factor for density, so a higher density for WMA was an advantage for only a few lots. The greatest advantage of the hypothetical 0.17% increase in density for WMA would occur on projects that often had pay deductions for density. A small improvement in density resulting from the use of WMA could have a substantial impact on the overall payment that contractors receive on some projects. Some contractors believe that this benefit alone is sufficient justification for their use of WMA.

Estimating the potential savings resulting from improved smoothness when using WMA is a little more challenging. Incentive/disincentive specifications for smoothness vary considerably among highway agencies. In most cases, penalties and bonuses for smoothness only apply to surface layers. Moreover, although there have been a few WMA projects that reported improved smoothness with a WMA overlay on a concrete pavement or overlays pavements with large, sealed cracks, the improvements were not quantified in the available literature. Nonetheless, as with potential benefit for density, many contractors routinely use WMA to help achieve smoother pavements.

Since some WMA chemical additives contain antistripping compounds, some agencies may waive the requirement for an antistripping agent if the mixture with the WMA additive can pass the agency's moisture damage susceptibility test. Eliminating the antistripping agent can also significantly reduce a mixture's cost. For example, consider a typical liquid antistripping dosage rate of 0.5% by weight of asphalt binder, a cost of antistrip agent of $1.50/pound, and a typical asphalt content of 5%. The savings that would be realized by eliminating the antistripping agent (ASA) is:

2000 lb/ton × 5% asphalt × .5% ASA × $1.50/lb of ASA = $0.75/ton of mix.

Hydrated lime is also required as an antistripping agent by some state DOTs. Although agencies that require hydrated lime seem less likely to allow it to be eliminated when a WMA additive with antistripping capabilities is used, the estimated savings for that case is:

1% Hydrated Lime/ton of mix × $150/ton for hydrated lime = $1.50/ton of mix.

A summary of the estimated costs and potential economic benefits associated with the use of WMA is provided in Table 225. For water-injection foaming systems for WMA, the cost of the technology can be offset by energy savings alone, even if the energy savings is about half of what has been estimated from controlled experiments in NCHRP 9-47A. It is important to note that the estimated unit cost for these systems is based on the system operating for <u>all</u> asphalt mix production over the depreciation period. For the WMA additive technologies, there must be additional savings beyond energy reduction for the technology to at least break even. It is easy to see that in a permissive specification environment that allows contractors to choose the WMA technology, an investment that has a more certain financial benefit will typically be selected.

Table 225 Summary of Estimated Costs and Potential Savings for WMA Technologies

WMA Type	Water Injection Foaming	Additive
Typical Technology Cost ($/ton)	($0.08)	($2.50)
Assumed Temperature Reduction	25°F	50°F
Typical Energy Savings ($/ton)		
Recycled Fuel Oil	$0.39	$0.79
Natural Gas	$0.16	$0.31
Typical Incentive/Disincentive Spec. Savings ($/ton)		
Density Improvement	0 to $1.13	0 to $1.13
Smoothness	?	?
Possible Savings from Eliminated Antistripping Agent		
Liquid ASA	0	0 to $0.75[1]
Hydrated Lime	0	0 to $1.50[1]

[1] Applicable only to WMA additives with antistripping capabilities

CHAPTER 7

FINDINGS

PRODUCTION AND CONSTRUCTION OF WMA

1. Lower mix production temperatures associated with WMA did not cause plant issues or construction problems for any of the project sites evaluated in this study. Even with WMA mix temperatures that averaged 48°F (27°C) lower than corresponding HMA mixes, there were no problems with the burner, baghouse, motor amperage, or mix storage. Excellent coating was achieved with all WMA technologies at the lower mixture production temperatures.
2. In most cases, moisture contents of the WMA mixes were slightly higher than the corresponding HMA, but the differences were small and are believed to be inconsequential. WMA using water foaming process had similar moisture contents to mixes using other WMA technologies. Measured moisture contents for nearly all mixes were at or below the common specification limit of 0.5% moisture in asphalt mixes.
3. The mix designs were not altered for any of the WMA trial projects. Laboratory SGC compaction temperatures were set to be equal to the mat temperature at the start of rolling for all HMA and WMA mixes. In most cases, the SGC air void contents of the WMA mixes differed from the corresponding HMA mixes by more than 0.5 percent, but there was a similar number of cases where the WMA laboratory air void contents were higher and lower than the corresponding HMA. In short, other differences between WMA and HMA pairs, such as differences in asphalt contents and gradations, confounded the effects of mix temperature and WMA technology on laboratory compacted air void contents.
4. There is evidence that WMA mixes had slightly less asphalt absorption (0.12%, on average) than corresponding HMA for mixes sampled after discharge from the plant. For the projects in this study, differences in asphalt absorption between WMA and HMA ranged from essentially no difference to as much as 0.5%. Such differences are likely attributed to interactions of mix production temperature, storage time, aggregate characteristics, and binder properties. After about one year, the differences in absorption between WMA and HMA were not statistically significant.
5. In almost all cases, using the same roller patterns resulted in statistically equivalent as-constructed densities for WMA mixes compared to the corresponding HMA, even at much lower temperatures for WMA. Only one of the 15 WMA to HMA comparisons had an as-constructed density of the WMA section statistically higher than its corresponding HMA.
6. No difference was observed between the opening times to traffic of WMA and HMA after rolling. This dispels the concern that WMA would need to cool for a longer period of time before opening to traffic.

ENERGY AND EMISSIONS

1. Producing asphalt mixtures at lower temperatures saves energy. The data collected as part of this study showed that decreasing the mix production temperature by an average of 48°F (27°C) resulted in an average burner fuel savings of 22 percent. The energy savings associated with WMA was found to be reasonably approximated by the relationship:
   Energy savings (BTU) = 1100 BTU/Δ°F/ton
2. Reductions in carbon dioxide emissions measured at asphalt plant stacks were directly proportional to reductions in fuel usage. These data were consistent with results reported in other studies. However, other emissions, such as carbon monoxide and volatile organic compounds depended more on fuel type and burner tuning than the use of WMA.
3. Worker exposures to respirable fumes during paving with WMA were significantly reduced. Measurements of Total Organic Matter (TOM) in breathing zones of paving crews were obtained on two projects with six different WMA technologies and two HMA control sections. With one exception, the WMA mixtures resulted in at least a 33 percent reduction in TOM. The amount of emissions depends on characteristics of the asphalt binder and paving temperatures. All of the polycyclic aromatic compounds from asphalt fumes reviewed by the International Agency for Research on Cancer were below detectible limits on both projects.

SHORT-TERM WMA FIELD PERFORMANCE

1. WMA sections have performed the same as corresponding HMA sections with regard to rutting. All of the field projects have less than 5 mm of rutting after two years of traffic. Evaluations of WMA at several accelerated pavement testing facilities have also demonstrated that WMA can hold up to heavy loading.
2. None of the field projects had any evidence of moisture damage. Cores taken from the projects after one to two years of traffic were inspected for visual evidence of stripping. Even the experiment using saturated pavement sections tested under a Heavy Vehicle Simulator by the University of California Davis did not exhibit moisture damage.
3. The use of WMA did not appear to effect density changes under traffic compared to HMA. This observation was confounded by the fact that many of the WMA test sections were constructed in different lanes than the HMA section.
4. Very little cracking of any type was observed in the field test sections monitored in this study. Transverse cracking was the most common type of cracks. Eight of the fourteen projects had minor amounts of transverse cracking, but many of these cracks were likely reflection cracks. Only two of the newer projects had any transverse cracking after about two years. Of the projects with transverse cracking, the WMA and HMA sections generally had similar amounts. Four of the fourteen projects had minor non-wheelpath cracking, and only three projects had low severity longitudinal wheelpath cracking. In most cases, WMA and HMA sections on these projects had similar amounts of cracking. In the few

cases where one section had more cracking than its project companion(s), the section with more cracking also had a lower asphalt content.

5. All of the test sections had similar amounts of surface texture and texture change after two or more years of traffic. Surface texture measurements were conducted with the sand patch test as an indicator of raveling. None of the test sections had significant amounts of raveling.

ENGINEERING PROPERTIES OF WMA

1. Testing of recovered binders from mixes obtained during construction generally showed that the WMA binders had aged slightly less than the corresponding HMA binders. The average difference in the high critical temperatures between HMA and WMA binders recovered from plant produced mixes was 2.3°C, and the average difference for the low critical temperatures was 1.3°C. Such small differences would not be expected to significantly impact pavement performance.
2. Testing of recovered binders from cores taken after approximately one to two years of service generally indicate that the true grades of HMA and WMA were not substantially different. These test results also indicate that very little or no stiffening had occurred for the binders from the time of construction. The PAV conditioning of the recovered binders as part of the performance grading process may mask the effects of the plant aging and short-term field aging.
3. Lower mixing temperatures for WMA can affect the amount of binder absorbed in the pores of the aggregate for mixes sampled immediately following production. Of the thirteen WMA to HMA comparisons, the calculated asphalt absorption values were within 0.1 percent for eight of the comparisons. The other five cases had slightly less absorption for the WMA compared to its companion HMA. The amount of absorption in any mix will be affected by temperature, storage time, and aggregate properties. Tests on mix samples from cores after one to two years of service generally indicate that asphalt absorption values are similar for WMA and HMA pavements.
4. Statistical analyses indicate that the dynamic moduli of WMA mixtures are lower than those of corresponding HMA mixtures in most cases. Eleven of the thirteen WMA to HMA mix comparisons were found to have a lower E* for the WMA for at least one temperature and frequency used in the standard dynamic modulus test. On average, the E* of WMA mixes were about 12 percent lower than its corresponding HMA, but the differences ranged from about 5 percent stiffer to 40 percent less stiff.
5. Flow Number test results for plant-produced WMA mixes were statistically lower than corresponding HMA mixes in more than 2/3 of the comparisons. The Flow Number criteria recommended in NCHRP Report 673 for HMA and NCHRP Report 691 for WMA seem appropriate for evaluating plant produced mixes.
6. Indirect tensile strengths determined on cores obtained immediately after construction were not statistically different in 12 of the 14 WMA to HMA comparisons from the "new"

projects. In the majority of cases, the tensile strengths of WMA and HMA cores from the same project remained statistically equivalent through at least two years. These tensile strength tests were conducted on the same cores used to determine and compare in-place densities.

7. Indirect tensile strengths determined on SGC-molded specimens using hot compacted samples from plant mix were statistically different for WMA and corresponding HMA mixes. In a little more than half of the comparisons, tensile strengths were statistically lower for WMA compared to HMA. On the other hand, 38 percent of the laboratory-molded WMA mixtures had higher tensile strengths compared to the companion HMA mixes. All of these laboratory-molded specimens had air void contents in the range of 7±0.5 percent. The contrast between the comparisons of tensile strengths for cores and laboratory molded specimens indicates that the method of compaction influences the properties of asphalt mixture specimens.
8. The Tensile Strength Ratio (TSR) test was conducted in accordance with AASHTO T 283 on all of the plant-produced mixtures from "existing" and "new" projects evaluated in this study. Eighty-two percent of the mixes passed the standard 0.8 minimum TSR criterion. The six mixes that failed the criterion included four WMA and two HMA mixes. Only two mixes would have failed a minimum TSR limit of 0.75. Since all of the field projects have performed well with no evidence of moisture damage, consideration should be given to adjusting the TSR criterion on plant mix samples to 0.75 to reduce the number of false negatives with the test.
9. Hamburg wheel tracking tests were used to assess the rutting potential of the plant-produced mixtures as well as their resistance to moisture damage. As for the rutting comparisons, 59 percent of the WMA mixes had statistically equivalent Hamburg rut depths to their corresponding HMA mixes, and the other 41 percent of the WMA mixes had greater Hamburg rut depths than their companion HMA mixes. Since no nationally accepted criteria for Hamburg rutting have been established, results were evaluated using suggested criteria from the NCAT Test Track based on limited data with HMA mixtures. Four of the WMA mixtures did not meet the suggested criteria for moderate trafficked pavements. However, as noted in the Conclusions on Short-Term Field Performance, all of the WMA and HMA pavements have performed very well, indicating that either the Hamburg rut depth criteria should be adjusted for WMA or conditioning of WMA mixtures should be changed to yield results consistent with field performance.
10. The Hamburg wheel tracking tests is also used by a growing number of state highway agencies to assess stripping potential. The Hamburg test currently lacks a precision statement and there is no consensus regarding criteria for evaluating moisture damage. NCAT has used a minimum of 5,000 cycles for the Stripping Inflection Point (SIP) in a number of studies. Ten of the 34 mixes evaluated in this study failed that criterion including nine of the 22 WMA mixes. These results indicate that the current Hamburg test method or the 5000 cycle limit for SIP is too severe for evaluating WMA.

11. The uniaxial fatigue test, also known as the Simplified Viscoelastic Continuum Damage (SVECD) test, was conducted using the Asphalt Mixture Performance Tester (AMPT) on eleven plant-produced mixes in the study. Although the laboratory results indicate some differences in fatigue behavior among the mixes, without validation of the procedure in a well-controlled field experiment, drawing conclusions about the laboratory results is not appropriate.
12. The indirect tensile creep compliance and strength test was conducted on thirteen plant-produced mixes from the study to evaluate their thermal cracking potential. Overall, the laboratory test results indicate that WMA mixtures would show a small improvement in low temperature cracking compared to their control HMA. However, there was not enough observed thermal cracking in the actual pavements with these mixtures at the time of the last project inspections to validate the laboratory results.

PREDICTED PERFORMANCE

1. The MEPDG predicted slightly more rutting for the WMA sections compared to the HMA sections, on the order of 0.2 mm. This predicted difference was consistent through 20-years of service. Statistically, the predicted differences were not significant. Further, comparisons with observed field performance over one to two years suggest the MEPDG over-prediction of rutting was greater for WMA as compared to HMA.
2. Short-term observed field and long-term predicted rutting performance indicate there is a discrepancy between laboratory and field rutting performance for WMA. Conversely, HMA mixes, as measured by laboratory rutting tests, may be more rut resistant than they need to be to provide adequate field performance.
3. The MEPDG performance predictions of top-down, longitudinal cracking after both 12 and 20 years of service were similar for both WMA and HMA. Numerically slightly more cracking was predicted for the HMA compared to the WMA sections; statistically they were not different.
4. Using Level 1, low temperature IDT inputs, the MEPDG predicted less low temperature cracking with time for the WMA sections compared to the HMA sections. The differences are not statistically significant.
5. Overall, the MEPDG predicted similar long-term performance for WMA and HMA mixes using the engineering properties measured from the field-produced mixes.

MIX DESIGN VERIFICATION

1. For laboratory produced mixes, aged for two hours at the observed field compaction temperature, maximum theoretical gravity and calculated binder absorption were generally lower than for field produced mix. In all cases, the binder absorptions of laboratory produced WMA were less than the binder absorptions of laboratory produced HMA.
2. The methods described in the Appendix to AASHTO R 35 were followed to produce the laboratory WMA. The optimum asphalt contents were verified for 15 mixes, 10 WMA and

5 HMA. In 6 of 10 cases, the optimum asphalt content for the WMA was less than for the HMA. Overall, the optimum asphalt contents for the WMA mixes averaged 0.27 percent less than the HMA.

3. A bucket mixer was used to produce the WMA mixes. After 90 seconds of mixing at optimum asphalt content all ten of the WMA mixes exceeded the 95 percent coating specified in the Appendix to AASHTO R 35. Six of ten mixes equaled or exceeded the observed field coating.
4. Six of ten WMA mixes failed the compactability ratio of 1.25 recommended in the Appendix to AASHTO R 35. Four of six mixes that failed compactability had low in-place density in the field; however, the asphalt contents were the laboratory verified optimum and not that measured in the field.
5. Three of ten TSR tests of laboratory produced WMA were less than 0.8. The field mixed, plant compacted TSR on one of these mixes also failed. As noted previously, no moisture damage was observed in the field after one to five years of service.
6. Flow number tests were conducted on laboratory produced mix at the optimum asphalt content determined from the mix verifications. Nine of ten mixes met the Appendix to AASHTO R 35 flow number criteria. The mix that failed had 0.0 mm rutting after two-years and therefore appears to be a false negative.

SUGGESTIONS FOR MODIFYING PRACTICE

Mix Design

1. The “drop-in approach” for WMA mix designs has worked well and avoids the potential of designing mixes with lower asphalt contents when using WMA. Therefore, mix designs should be conducted without the WMA technology to determine the optimum asphalt content for the mix. Coating, compactability, and TSR should be confirmed using the proposed WMA technology and temperatures.
2. Based on the field and predicted performance of WMA, Flow Number testing should only be required for pavements with predicted traffic over 30 million ESALs.
3. The Appendix to AASHTO R 35 should be modified as described in this report.
4. TSR criteria for plant-produced HMA and WMA should be decreased to 0.75 to reduce the number of false negatives (failing results but good performance).
5. If the Hamburg is used in the future to evaluate WMA mixes, two options may be considered to reduce the number of rejected mixes that would likely provide good field performance. One option, used by the Texas DOT, is to extend the conditioning of WMA mixtures from 2-hours to 4-hours at 275°F *(31).* Another option is to consider adjusting the rut depth criteria similar to what has been done for the Flow Number criteria.

Production

1. Best practices should be utilized to minimize stockpile moisture contents in order to maximize fuel savings.
2. Best practices should be utilized to maintain adequate baghouse temperatures in order to prevent condensation.
3. Dryer burners should be tuned to maximize performance and minimize fuel usage and emissions. Plant manufacturers should consider designs that will allow efficiency over a range of firing rates.
4. Handwork may require higher WMA production temperatures.

OTHER RESEARCH

Another significant NCHRP study titled evaluation of the moisture susceptibility of WMA technologies was recently completed. The final report for that project is published as nchrp report 763. Readers are advised to review the findings of that report. Another major wma related project, NCHRP Project 9-49A, *Performance of WMA Technologies: Stage II—Long-Term Field Performance*, has issued an interim report that may be obtained from NCHRP. The long-term field performance monitoring aspect of that project continues through 2015; the final report is anticipated to be completed in 2016. Also, the Long-Term Pavement Performance (LTPP) program has initiated a new WMA experiment that will involve building and monitoring new test sections.

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APPENDIX A

FALLING WEIGHT DEFLECTOMETER TESTING

APPENDIX A – FALLING WEIGHT DEFLECTOMETER TESTING

FLORIDA

The FWD data was provided by the Florida DOT. It was collected on January 17, 2005 by Applied Research Associates, Inc. The testing was conducted on SR 30 from mile post 0 to 7.412. The testing was done on the eastbound lane. The highway was overlaid on October 6, 2010 with both Hot Mix Asphalt (HMA) and Warm Mix Asphalt (WMA). The WMA was a foaming technology by Terex Corporation. The WMA was paved in the eastbound lane, and the HMA was paved in the parallel westbound lane. The FWD data provided was only for the eastbound lane, so the analysis was performed only on the eastbound (WMA) section. The analysis was completed using ModTag software developed by the Virginia Department of Transportation. According to global positioning satellite (GPS) readings taken at construction, the WMA section started at mile post 5.3 and ended at the Aucilla River. Cores were taken at both the one and two year revisits. The surface lift was not considered in the analysis because it had yet to be placed when the FWD data was obtained. The cores heights, minus the surface lift, were averaged and that value was used as an input in ModTag. Inputs for ModTag are summarized in Table 226. The Structural Number effective (SNeff) of the pavement and the Resilient Modulus of the Subgrade (Mr) are displayed in Figure 149. The Mr is labeled as Design Mr because it has been corrected by a factor of 0.33, according to the AASHTO standards.

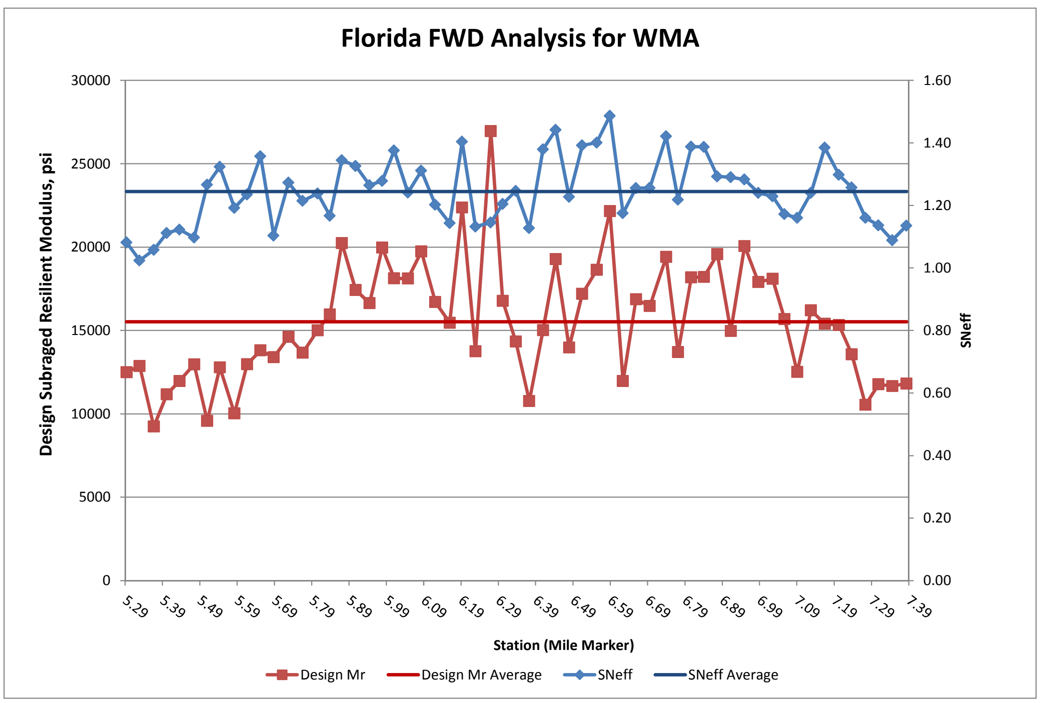

Figure 149 Florida FWD Analyses for Resilient Modulus and Structural Number

ARIZONA

The FWD data was provided by Arizona Department of Transportation. The data was collected on May 26, 2010. The data was collected on SR 84 E between mile post 166.4 and 172.0. The overlay for the eastbound lane was a section from milepost 169.3 to 172.0. This section was paved with a WMA containing Sasobit. This project also had an HMA and Advera section; however they were both paved parallel to the Sasobit, in the westbound lane. The Advera section was not tested as part of this project. The core data collected at the 1 year revisit was averaged to determine a pavement height of the Sasobit section. The surface layer height was removed from the core height because the FWD data was collected before the overlay. The inputs for this data can be found in Table 226. The Sasobit section is clearly marked in Figure 150. M_r changes significantly in the Sasobit section.

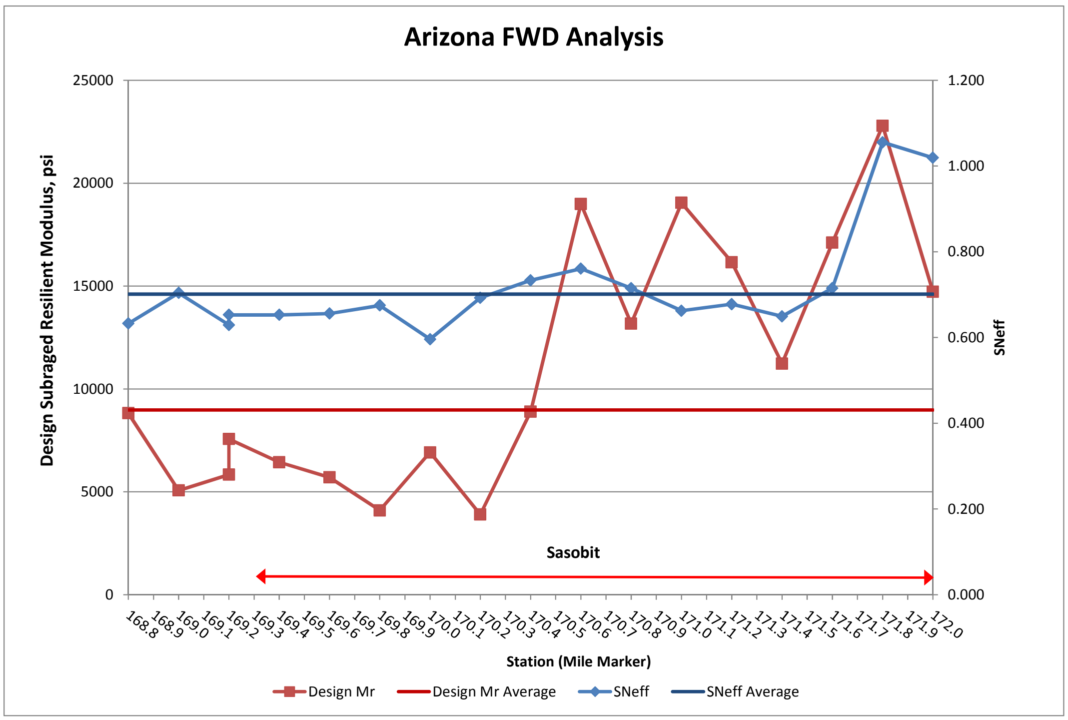

Figure 150 Arizona FWD Analysis for Resilient Modulus and Structural Number

INDIANA

The FWD data was collected by NCAT on September 13, 2010. The testing was completed on the outside lanes prior to overlaying the pavement. According to the field notes, the inside lanes for both the north and southbound lanes were not tested due to dangerous traffic conditions. It was assumed that the inside lane would be equivalent to outside lane. The HMA was placed in the outside southbound lanes, while one of the WMA technologies, Gencor Foam, was placed over the northbound outside lane. Since the FWD data was collected prior to the overlay, the surface lift height was removed from the overall core thickness. There were no available mile posts so the test locations were recorded every 500 feet from a known location. The southbound section began just north of the intersection of Main Street and Calumet Avenue, while the northbound section began at the intersection of 45th Avenue and Calumet Avenue. The FWD analysis can be found in Figure 151 and Figure 152. The average SN_{eff} are similar for the north- and southbound lanes, but the M_r is higher for the northbound lane.

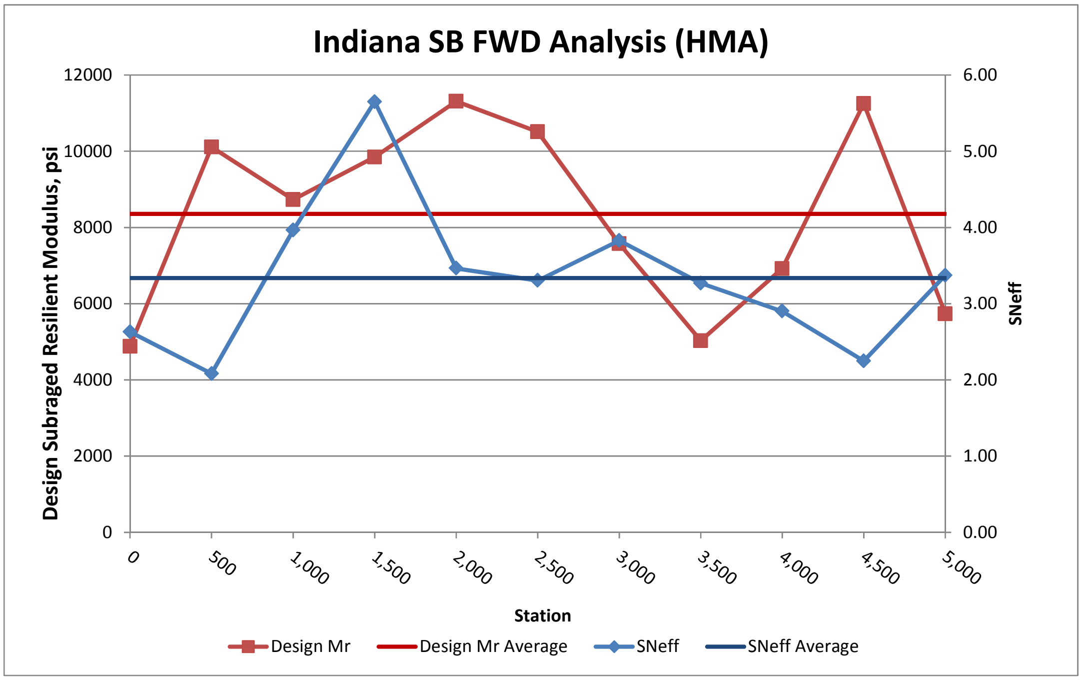

Figure 151 Indiana HMA Resilient Modulus and Structural Number

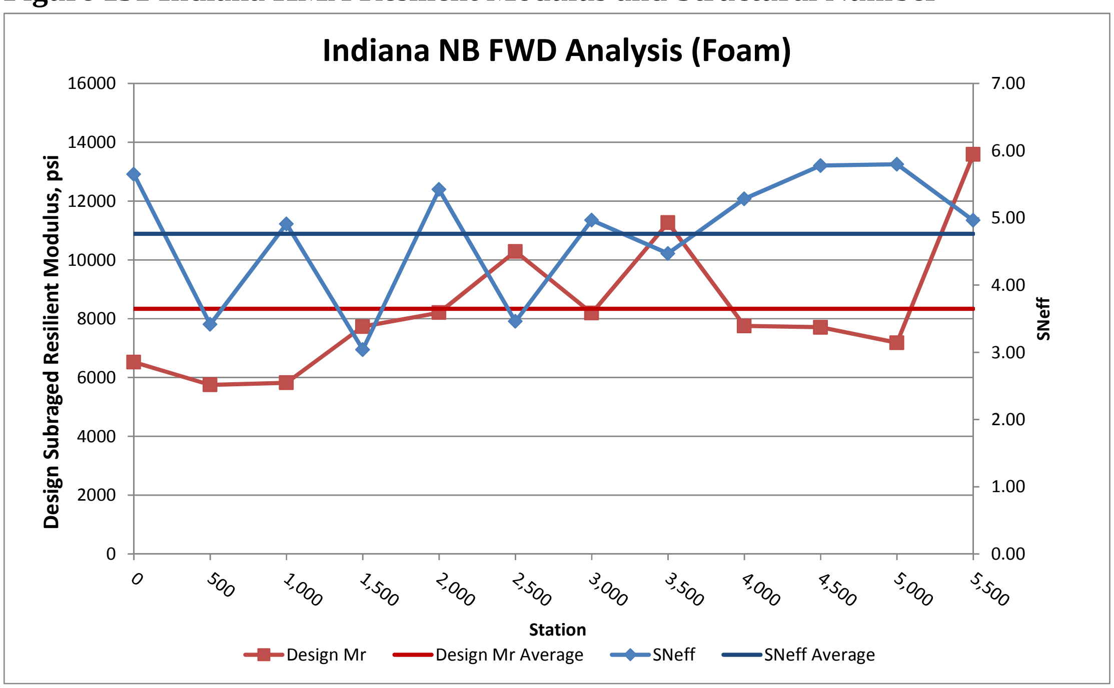

Figure 152 Indiana Gencor Foam Resilient Modulus and Structural Number

MICHIGAN

The FWD data was collected by NCAT on July 21, 2010. The HMA and the warm mix technology Advera, were placed on the surface prior to testing. The Evotherm section was tested on the intermediate layer. The surface lift height was removed from the core height for the Evotherm section; however the HMA and Advera sections used full depth core data. The construction start point was at the intersection of CR-513 and US-2. The test sections were recorded in feet but were converted into miles. This allowed the northbound and southbound sections to be compared. The construction start point begins at 0.1 miles. The analysis of the three sections can be found in Figure 153, Figure 154, and Figure 155. The SN_{eff} are similar for the HMA and Advera sections, which included the surface layer.

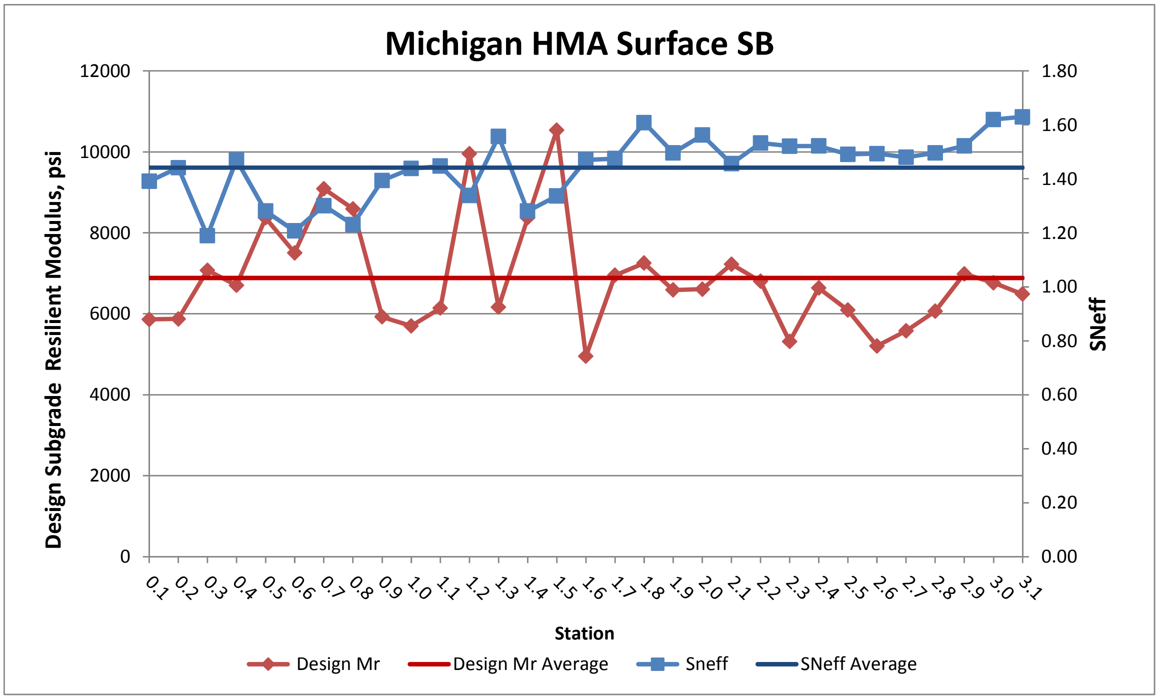

Figure 153 Michigan HMA Resilient Modulus and Structural Number

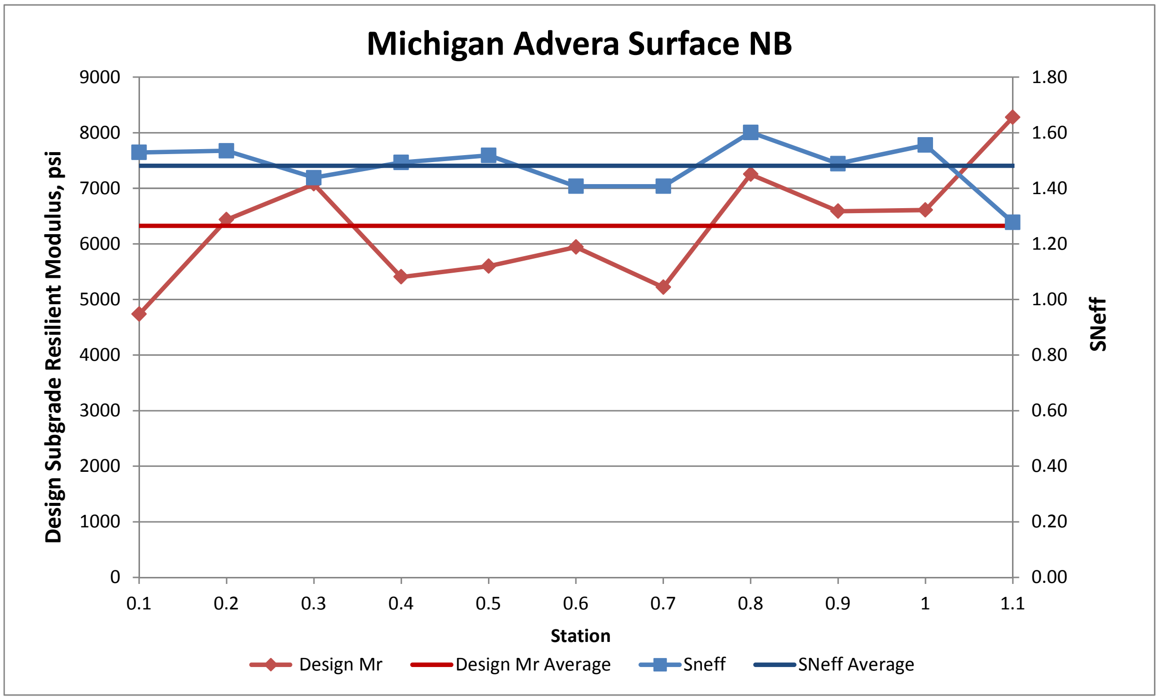

Figure 154 Michigan Advera Resilient Modulus and Structural Number

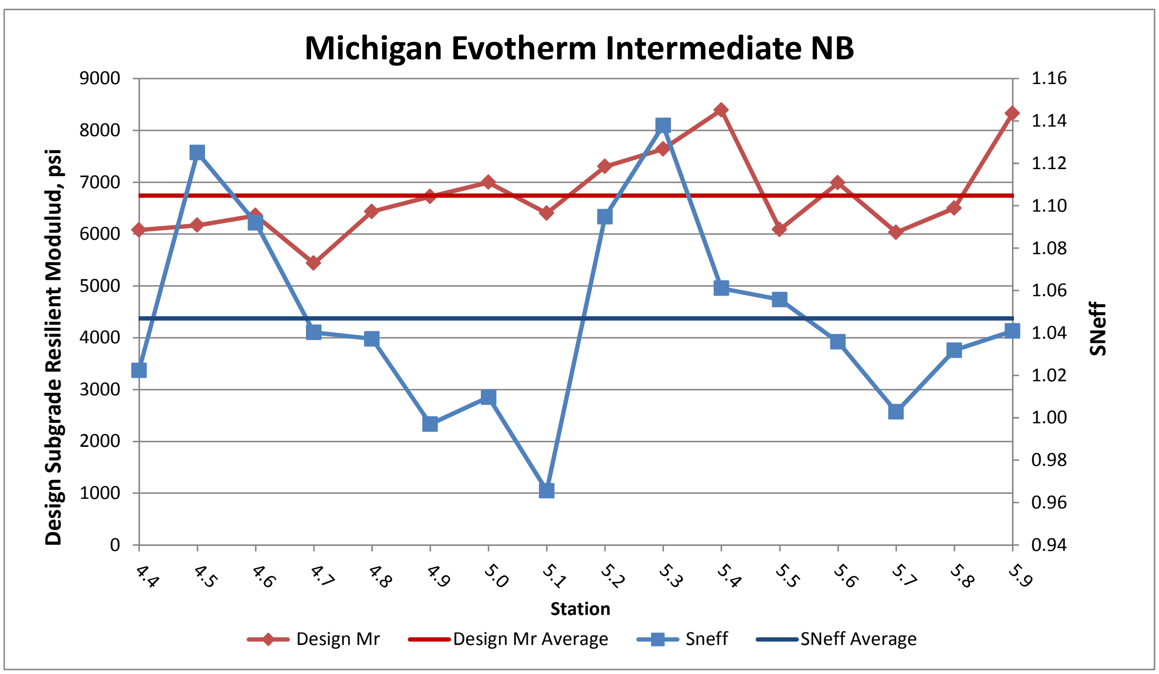

Figure 155 Michigan Evotherm Leveling Resilient Modulus and Structural Number

NEW YORK

The FWD data was collected by NCAT on October 19th and 20th of 2010. The testing was conducted on both the north and southbound lane of Little Neck Parkway. One full-depth core was taken at the end of construction and it was determined that a 6-inch concrete layer existed under the asphalt overlay. The SonneWarmix and Bitutech PER were constructed in the northbound lane, and the Cecabase and the HMA were constructed in the southbound lane. The test locations were measured in feet from a recorded location. There were no mile posts on this section of roadway. The SonneWarmix section started from the intersection of 87th Drive and Little Neck Parkway, while the Bitutech PER section started at the intersection of Hillside Avenue and Little Neck Parkway. In the southbound lane the Cecabase section started at the intersection Union Turnpike and Little Neck Parkway, while the HMA section started at the intersection of Hillside Avenue and Little Neck Parkway. The core heights from the one and two year visits were averaged and used as inputs in ModTag. The results for the north and southbound lanes can be found in the following figures. The SN_{eff} is higher for the Bitutech PER; lower for the SonneWarmix. The average M_r was also higher for the Bitutech PER.

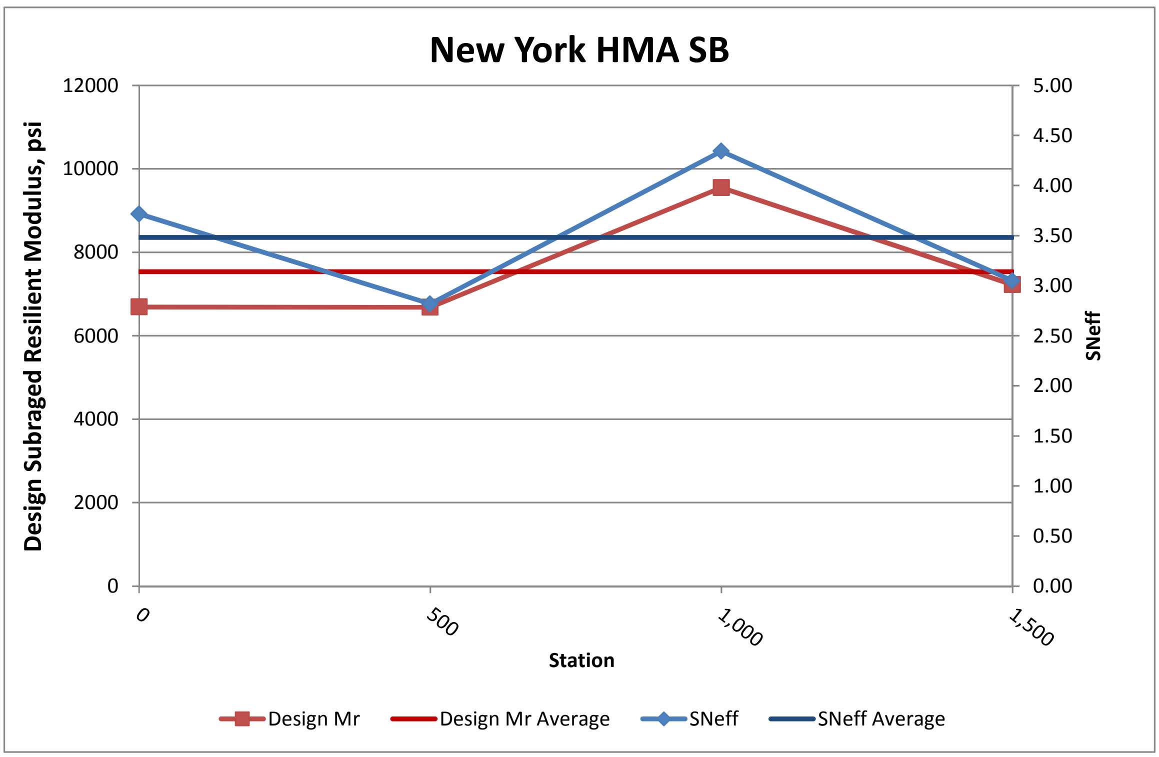

Figure 156 New York HMA Resilient Modulus and Structural Number

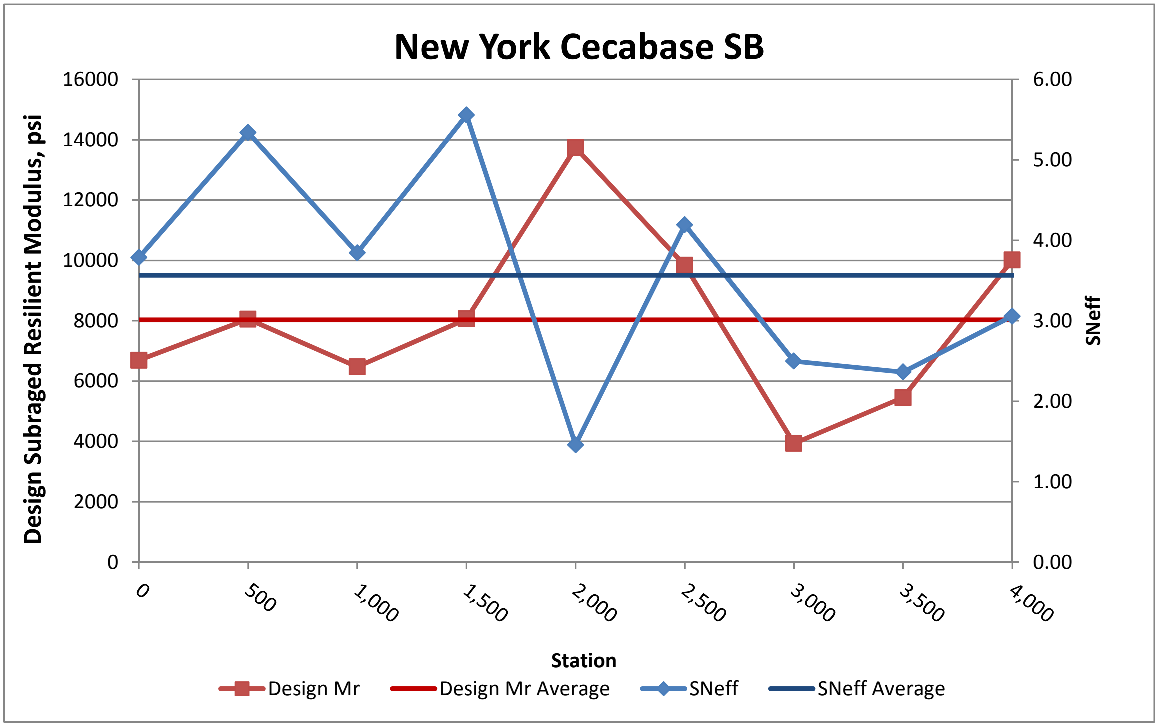

Figure 157 New York Cecabase Resilient Modulus and Structural Number

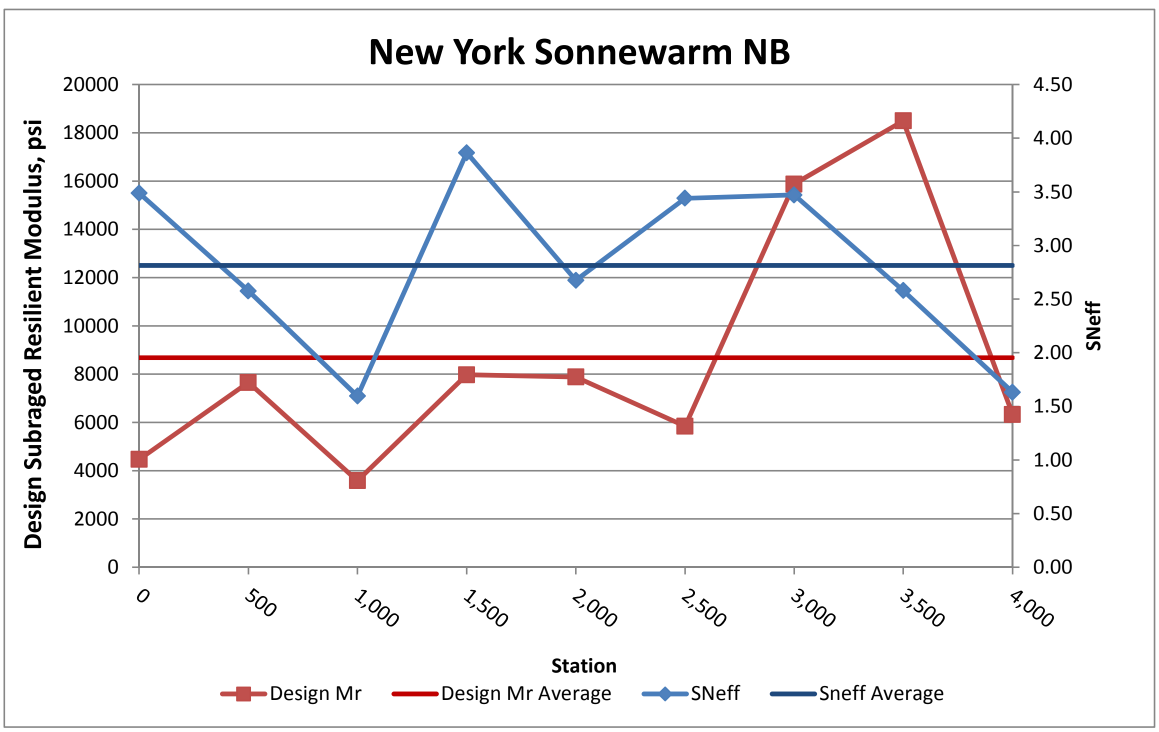

Figure 158 New York SonneWarmix Resilient Modulus and Structural Number

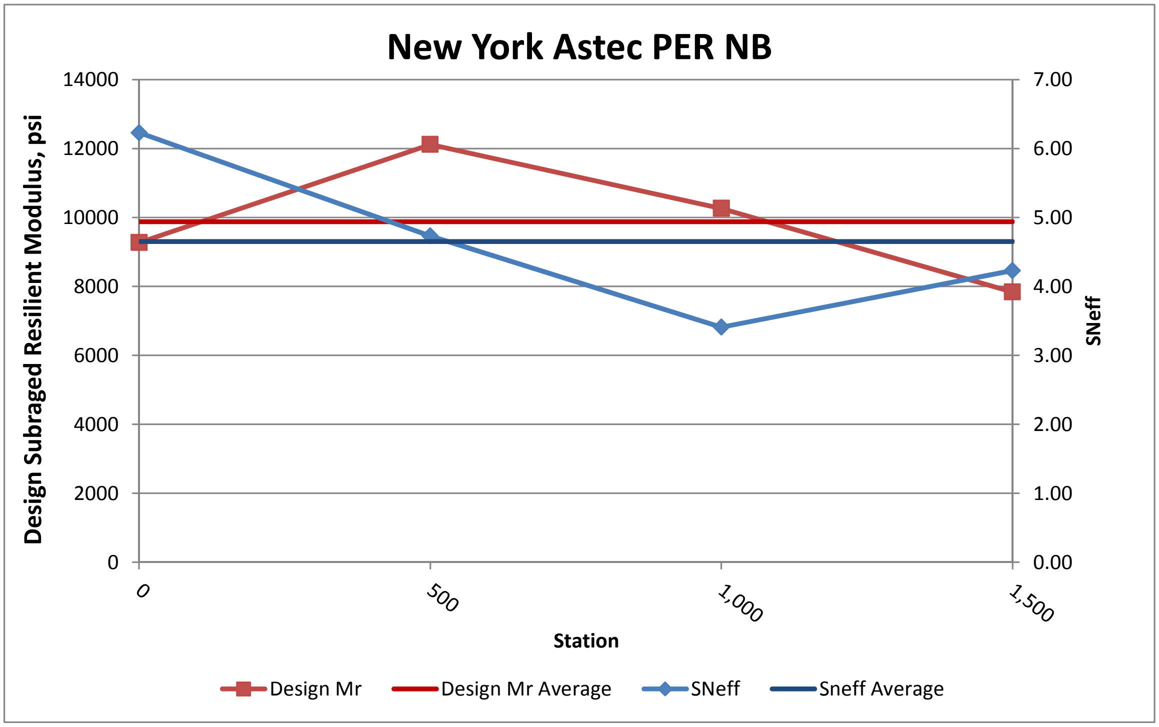

Figure 159 New York BituTech PER Resilient Modulus and Structural Number

MONTANA

The FWD data was provided by Montana Department of Transportation. The data was collected on June 5, 2013. The data was collected on CR-322 from the intersection of Route 7 to a point 2.6 miles east of Route 7. Both the HMA and Evotherm WMA were placed in the eastbound lane. The WMA mix started at a point 2.6 miles from the intersection with Route 7. This wa apparently not tested, so a comparison between the HMA and WMA could not be made. The HMA SN_{eff} and M_r are shown in HMA was also placed in the westbound lane. Core data was supplemented with ground penetrating radar (GPR) testing to determine the pavement thickness.

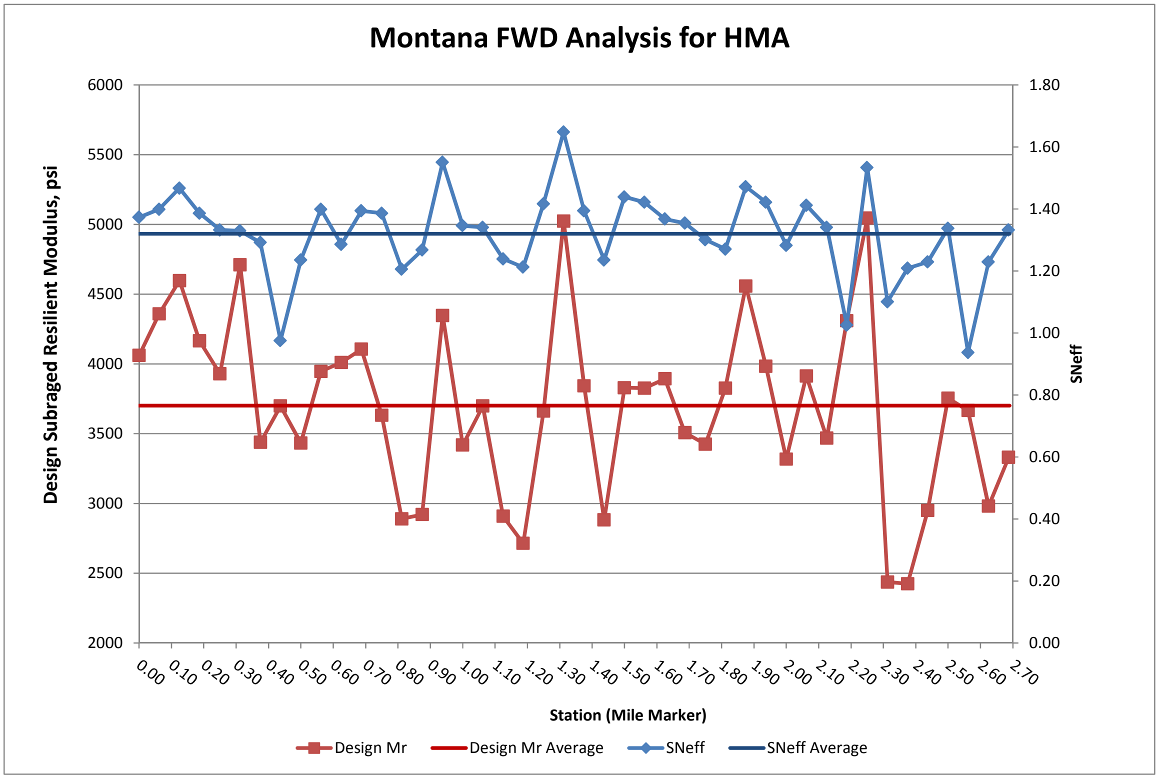

Figure 160 Montana HMA Resilient Modulus and Structural Number

Table 226 ModTag Inputs for NCHRP 9-47A FWD Analyses

State	Technology	Core Height (in)	Unbound Layer (in)
Florida	Terex Foam	4.6*	192.4
Michigan	HMA	4.2	295.8
	Advera	3.9	296.1
	Evotherm	2.3*	297.7
New York**	HMA	2.5	282.5
	Astec PER	2.8	291.2
	Cecabase	3.0	282.0
	Sonnewarm	2.8	246.2
Indiana	HMA	2.6	256.4
	Gencore Foam	4.8	295.2
Montana	HMA	6.9***	293.1
Arizona	Sasobit	4.4*	295.5

* Surface Lift Height Removed

** 6" of Existing Concrete Pavement under asphalt

***Pavement Thickness from GPR data