ALABAMA WARM MIX ASPHALT FIELD STUDY: FINAL REPORT

By

Andrea Kvasnak
Brian Prowell
Graham Hurley
Andre de Fortier Smit
Jaeseung Kim

May 2010

National Center for Asphalt Technology
NCAT
at AUBURN UNIVERSITY

277 Technology Parkway ■ Auburn, AL 36830

ALABAMA WARM MIX ASPHALT FIELD STUDY: FINAL REPORT

By

Andrea Kvasnak
Lead Engineer
National Center for Asphalt Technology
Auburn University, Auburn, Alabama
ank0004@auburn.edu
334-844-7303

Brian Prowell
Lead Researcher
Advanced Materials Services, LLC
Auburn, Alabama

Graham Hurley
Research Engineer
Advanced Materials Services, LLC
Auburn, Alabama

Andre de Fortier Smit
Research Engineer
Center for Transportation Research
Austin, Texas

Jaeseung Kim
Research Professor
National Center for Asphalt Technology
Auburn, Alabama

Sponsored by

Alabama Department of Transportation

NCAT Report 10-XX

May 2010

DISCLAIMER

ACKNOWLEDGEMENTS

The authors graciously thank the Alabama Department of Transportation for sponsoring this project. In particular, the researchers would like to thank Larry Lockett and Lyndi Blackburn for their invaluable input.

ABSTRACT

The Alabama Department of Transportation hosted a warm mix asphalt field demonstration in August 2007. The warm mix asphalt technology demonstrated was Evotherm Dispersed Asphalt Technology. The WMA and hot mix asphalt produced for the demonstration were sampled and evaluated in the laboratory. The construction of the WMA and HMA pavements were also documented along with the condition of the pavement sections up to one year. The results of the laboratory evaluation and field documentation are detailed in this report.

Table of Contents

INTRODUCTION

WMA is an emerging technology that allows for the production of asphalt mixes at lower temperatures than traditionally employed for HMA. Typical WMA production temperatures are 16 to 56°C (30 to 100°F) less than HMA. The production of an asphalt mix at temperatures less than 135°C (275°F) can result in lower emissions, decreased fuel usage, and reduced oxidation of the asphalt binder compared to mixes produced at 300°F and above (*1*). The reduced emissions and fuel usage can be environmentally beneficial and reduced fuel usage can be economically beneficial. The question that arises: Is the performance of the asphalt mix adversely affected by using a WMA technology? If it is, then the environmental and economic benefits are negated. If the performance of WMA pavements is as good as or better than HMA, then the change in production practices is worthwhile.

Recent interest in warm mix asphalt technologies (WMA) has lead to several states hosting WMA field demonstrations. One such demonstration was hosted by the Alabama Department of Transportation (ALDOT) and was documented and evaluated by the National Center for Asphalt Technology (NCAT). The mixes used in the demonstration included reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS). The WMA demonstration was conducted in August 2007. The WMA technology evaluated was MeadWestvaco's Evotherm™ Dispersed Asphalt Technology (DAT). Evotherm™ DAT is a WMA additive in the form of a liquid chemical package. The chemical solution contains coating, workability, and adhesion agents. The additive reduces the internal friction in a mix, which allows for proper coating and compaction at temperatures lower than conventional HMA.

This report summarizes the evaluation of the WMA demonstration. The construction of the WMA and HMA sections was documented and mix was sampled and evaluated. The mix testing that was conducted included moisture susceptibility, dynamic modulus, creep compliance and strength, and loaded wheel testing. The performance of the pavement was monitored after construction.

Scope

The objectives of this study were to (1) conduct mix design verifications for the WMA technology and HMA; (2) document the production and construction of WMA and HMA pavement sections; (3) evaluate and compare the WMA and HMA plant-produced mix; and (4) monitor the in-place performance of the WMA and HMA pavements.

EXPERIMENTAL PLAN

There were four phases to the research project. The first phase consisted of mix design verifications prior to construction. The documentation of construction and collection of materials was part of Phase II. Phase III of the project was the laboratory evaluations of the plant-produced mix. Phase IV of the project encompassed the monitoring of the pavement performance.

Phase I Experimental Plan

The goal of Phase I was to determine if both RAP and RAS could be incorporated into the WMA and HMA mixes that would be produced as part of the study. Two WMAs and two HMAs were evaluated during the mix selection process. Two HMA mix designs were the basis for the evaluations. The WMA mix designs were the HMA designs but produced at lower temperatures using a WMA additive. The four mix designs verified were:

1. An HMA with 10% reclaimed asphalt pavement (RAP) and 5% reclaimed asphalt shingles (RAS)
2. An HMA with 15% RAP
3. A WMA with 10% RAP and 5% RAS evaluated at two mixing temperatures
4. A WMA with 15% RAP evaluated at two mixing temperatures

The mix design verifications for both included volumetric analysis, coating evaluations (AASHTO T 195), and moisture susceptibility testing (ALDOT 361) (see TABLE 1). The coating evaluation was included to evaluate if sufficient asphalt from the RAP and

RAS was activated at the WMA temperatures. If aggregates are not properly coated with asphalt, the mix could be prone to moisture damage and durability issues.

TABLE 1 Phase I Experimental Plan

Evaluation	Number of Samples per Mix
Volumetrics	3 per mixing temperature
Coating Evaluations (AASHTO T 195)	3 at one mixing temperature
Moisture Susceptibility (ALDOT 361)	3 unconditioned and 3 conditioned at one mixing temperature

Phase II Experimental Plan

The documentation of the construction of the HMA and WMA consisted of reporting on production at the plant and placement on the road. TABLE 2 lists the information collected during the production and placement of the HMA and WMA pavement sections.

TABLE 2 Documentation of Field Project

Production Documentation	Placement Documentation
Mixing Temperature	Haul Distance
Introduction of WMA Technology	Lift Thickness
Temperature of Trucks after Loading	Mix Delivery Temperature
Differences between HMA and WMA Production	Temperature Behind Screed
	Density

In addition to documenting the production and placement of the HMA and WMA, mix components and loose mix were collected. A portion of the loose mix was used to compact specimens on site in the NCAT mobile laboratory. The specimens compacted on site in the mobile laboratory were for:

- Moisture Susceptibility Testing (ALDOT 361)
- Hamburg Testing (AASHTO T 324)
- Asphalt Pavement Analyzer Testing (AASHTO TP 63)
- Indirect Tensile Creep Compliance and Strength Testing (AASHTO T 322)

Additional mix was collected for further testing at the main NCAT laboratory. Dynamic modulus and flow number specimens were compacted at the main NCAT laboratory. Loose plant-produced mix was set aside for each sample collected to evaluate the aggregate gradation, asphalt content, and recovered asphalt binder performance grade. TABLE 3 summarizes the specimens made in the field, specimens made in the lab, and samples collected for evaluations.

TABLE 3 Samples for Tests and Evaluations of Plant-Produced Mix

Evaluation	Number of Specimens per Mix	Type of Specimens
Moisture Susceptibility (ALDOT 361)	6	Compacted Hot
Hamburg Testing (AASHTO T 324)	4	Compacted Hot
Asphalt Pavement Analyzer (AASHTO T 322)	6	Compacted Hot
Indirect Tensile Creep Compliance and Strength (AASHTO T 322)	3	Compacted Hot
Dynamic Modulus (AASHTO TP 62)	3	Compacted from Reheated Mix
Flow Number (NCHRP 09-29)	3	Compacted from Reheated Mix
Asphalt Content (AASHTO T 319)	2 per Sample	Recovered from Loose Mix
Sieve Analysis of Recovered Aggregate (AASHTO T 30)	2 per Sample	Recovered from Loose Mix
Performance Grade of Recovered Asphalt (AASHTO R 29)	2 per Sample	Recovered from Loose Mix

Phase III Experimental Plan

Phase III of the study consisted of evaluating the materials collected during construction. The dynamic modulus and flow number specimens were also compacted during this phase. Specimens made in the field and the main NCAT laboratory were both tested at the main NCAT laboratory. The mix testing for the HMA and WMA included the following tests:

- Moisture Susceptibility Test (ALDOT 361)
- Hamburg Wheel Tracking Test (AASHTO T 324)

- Asphalt Pavement Analyzer Rut Test (AASHTO TP 63)
- Indirect Tensile Creep Compliance and Strength Test (AASHTO T 322)
- Dynamic Modulus Test (AASHTO TP 62)
- Flow Number Test (NCHRP 09-29)

Additional testing was conducted on the recovered asphalt (AASHTO R 29) and aggregate. (AASHTO T 30) to determine the recovered asphalt grade and extracted aggregate gradation, respectively.

Moisture Susceptibility Testing (ALDOT 361)

The moisture susceptibility testing was conducted in accordance with ALDOT 361, *Resistance of Compacted Hot-Mix Asphalt to Moisture Induced Damage*. Three specimens per mix were tested unconditioned. These unconditioned specimens were placed in a water bath at 25±1°C (77±1.8°F) one hour prior to testing. Another set of three specimens per mix were tested conditioned. The conditioned specimens were saturated and then placed in a 60±1°C (140±1.8°F) water bath for 24 hours followed by one hour in a water bath at 25±1°C (77±1.8°F). Both unconditioned and conditioned specimens were loaded diametrically at a rate of 50 mm per minute. The tensile strength for each specimen was then calculated using specimen dimensions and failure load. The tensile strength ratios were then calculated by dividing the average conditioned tensile strength by the average unconditioned tensile strength. The acceptable tensile strength ratio value employed was 80%.

Hamburg Wheel Tracking Testing (AASHTO T 324)

AASHTO T 324, *Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot-Mix Asphalt (HMA),* was used to evaluate the stripping and rutting potential. Specimens compacted in the field were cut in half to yield two cylindrical specimens. Two specimens were tested at once in a twin mold. All sets of specimens were conditioned and tested in a 50°C (122°F) water bath. The test was run for 10,000 cycles (20,000 passes) or until the specimens failed. The stripping inflection point, total rut depth at 10,000 cycles (20,000 passes), and rutting rate was determined for each set of specimens in accordance with AASHTO T 324. The acceptable stripping inflection point criterion was a value that is equal to or greater than 5,000 cycles (10,000 passes). The acceptable total rut depth at 10,000 cycles (20,000 passes) employed was less than 10 mm. A standard criterion for rutting rate does not exist and the value is typically used when comparing two mixes. The rutting rate of the WMA was compared to the HMA rutting rate.

Asphalt Pavement Analyzer Testing (AASHTO TP 63)

The Asphalt Pavement Analyzer (APA) is another loaded wheel rutting test. APA testing was conducted in accordance with AASHTO TP 63, *Standard Method of Test for Determining the Rutting Susceptibility of Asphalt Paving Mixtures Using the Asphalt Pavement Analyzer (APA)*. Six cylindrical specimens per mix were conditioned and tested in a heated air chamber at 64°C (147.2°F). A hose pressure of 100 psi and a load of 100 lbs. were used. The number of loading cycles applied before the termination of each test was 8,000 cycles. The acceptable criterion employed was a rut depth of less than 8 mm since it was a low volume road.

Indirect Tensile Creep Compliance and Strength (AASHTO T 322)

AASHTO T 322, *Standard Method of Test for Determining the Creep Compliance and Strength of Hot-Mix Asphalt (HMA) Using the Indirect Tensile Test Device*, was used to evaluate the potential of a mix to experience cracking. Three specimens per mix were compacted from loose mix in the NCAT mobile laboratory. Specimens were then long term aged in accordance with AASHTO R 30, *Standard Practice for Mixture Conditioning of Hot Mix Asphalt*. The specimens were cut and the target air voids after cutting were 7 ± 0.5%. Creep compliance testing was conducted at 0, -10, and -20°C (32, 14, and -4°F) and followed by tensile strength testing at -10°C (14°F). The creep rate and the indirect tensile strength were determined from the test. There is no standard acceptance criterion for this test. The results were used to compare the WMA to the HMA.

Dynamic Modulus Testing (AASHTO TP 62)

Dynamic modulus testing was conducted in accordance with AASHTO TP 62 to evaluate the stiffness of the WMA compared to HMA. Specimens were compacted in a

Superpave gyratory compactor to 170 mm and then cut and cored to yield specimens that were 150 mm tall by 100 mm in diameter. Three specimens per mix were tested. The test was conducted at multiple temperatures and frequencies shown in TABLE 4 within the elastic response range of each mix. Specimens were tested confined. The confining pressure was 138 kPa (20 psi).

TABLE 4 Frequencies and Temperatures for Dynamic Modulus Testing

Frequency, Hz	Temperature, °C (°F)
25	4.4 (40)
10	21.1 (70)
5	37.8 (100)
1	54.4 (130)
0.5	
0.1	

The data from the dynamic modulus test was used to construct a master curve for each mix, which relates a material's stiffness over a range of frequencies. Master curves were developed by shifting dynamic modulus test results from different testing temperatures and frequencies to form one continuous curve. A reference temperature of 21.1°C (70°F) was employed to build the master curves. There is no acceptance criterion for dynamic modulus; therefore, the results were used to compare the stiffness of the WMA to that of the HMA.

Extraction and Recovery of Asphalt and Aggregate (AASHTO T 319 and ASTM D 5404)

Asphalt and aggregate were extracted and recovered from mix and RAP samples. The extractions were conducted in accordance with AASHTO T 319 using trichloroethylene. The asphalt was then recovered from the extractions in accordance with ASTM D 5404.

Recovered Asphalt Binder Classification (AASHTO R 29)

Comparisons of binder properties were conducted to determine if the WMA additive altered the base binder properties. Testing of recovered binder from the plant-produced mix was conducted in accordance with AASHTO R 29, *Grading or Verifying the Performance Grade (PG) of an Asphalt Binder*.

Recovered Aggregate Sieve Analysis (AASHTO T 30)

Sieve analyses of the aggregates recovered during the binder extraction were conducted in accordance with AASHTO T 30. The recovered aggregates were dried at 110±5°C (230±9°F) prior to conducting the sieve analysis. The weight of the dried aggregate was obtained and then the recovered aggregate was placed in a container filled with water. The recovered aggregates in the water filled container were then agitated and poured over the No. 16 sieve and the No. 200 sieve to separate the fine and coarse aggregate. The coarse aggregate was then dried and sieved over a set of nested sieves.

Phase IV Experimental Plan

The fourth phase of the study consisted of monitoring and documenting the performance of pavement sections placed. Three site revisits were conducted at 3 months, 6 months and 1 year after construction. Distresses observed were photographed and recorded. At the 1 year revisit, six cores were obtained to evaluate the change in density, indirect tensile strength, and bond strength. The three indirect tensile strength cores were tested as unconditioned specimens in accordance with AASHTO T 283. The bond strength test was used to demonstrate that using a WMA additive would not interfere with the strength of the bond between two pavement layers. Bond strength testing was conducted in accordance with ALDOT 430. Three 150 mm field cores from each section were obtained and tested. A load was applied diametrically to the joint between the surface and binder course. A bond strength greater than 100 psi was considered acceptable.

MATERIALS

All mixes contained a PG 67-22 asphalt binder. The virgin aggregates were limestone and steel slag. The RAP was from a multi-source stockpile. The RAS was manufactured waste. The WMA contained the WMA additive Evotherm™ DAT.

Two ALDOT mix designs, originally intended and used for HMA, were employed in the study; one contained 15% RAP and the other contained 10% RAP and 5% RAS. The job mix formula (JMF) gradation for the 15% RAP mixes is displayed in TABLE 5. The target virgin binder content was 4.45%. An additional binder content of 0.75% came from the RAP. The JMF gradation for the two mixes containing 10% RAP and 5% RAS is displayed in TABLE 6. The virgin binder made up 3.87% of the mix and 0.53% and 0.90% of the mix came from RAP and RAS binders, respectively.

TABLE 5 JMF Gradation for 15% RAP Mixes

Sieve	Percent Passing
1”	100
3/4”	100
1/2”	90
3/8”	72
# 4	56
# 8	40
# 16	31
# 30	23
# 50	12
# 100	8
# 200	4.7

TABLE 6 JMF Gradation for 10% RAP 5% RAS Mixes

Sieve	Percent Passing
1”	100
3/4”	100
1/2”	90
3/8”	70
# 4	57
# 8	44
# 16	33
# 30	24
# 50	13
# 100	9
# 200	5.6

PROCEDURES AND RESULTS

Phase I: Mix Design Verifications

Two HMA designs that had been used by the contractor for other paving projects were verified. One mix contained 15% EAP and the other contained 10% RAP and 5% RAS. The two mixes were reproduced in the laboratory as both a HMA and a WMA. These four mixes were considered for the WMA demonstration to determine which ones would be most appropriate for the demonstration. The initial intent had been to use a mix design that included RAP and RAS for both the WMA and HMA. However, there was some concern about whether or not the RAS binder would become activated at the lower mixing and compaction temperatures; therefore, mixes with RAP but no RAS were also considered during the mix design verification stage.

Mix design verifications were conducted for each mix using materials supplied by Dunn Construction. During the mix verification process, two WMA target compaction temperatures were initially evaluated, 120°C (248°F) and 95°C (203°F). Design specimens along with indirect tensile strength specimens were made to validate the mix designs. An additional test outside of the typical mix design evaluation was conducted to assess the extent of coating.

Laboratory Mix: Specimen Production

The JMF gradations were used as the target gradation for the laboratory-produced specimens. One target mixing temperature was evaluated for HMA and two were evaluated for WMA for both mix designs. TABLE 7 lists the target mixing temperatures for the HMA and WMA produced in the laboratory for the mix design verifications. The column Mix Set identifies the first set of mixing temperatures used and the second set, which is only applicable to the WMA, identifies the second mixing temperature evaluated for both mix designs. Aggregate was oven dried and preheated overnight to the target mixing temperature prior to mixing. RAP was preheated for 30 minutes at the target mixing temperature.

TABLE 7 Target Mixing Temperatures for Mix Verifications

Mix Set	HMA	WMA
Mixing Temperature 1	163°C (325°F)	136°C (277°F)
Mixing Temperature 2		110°C (230°F)

The preheated virgin aggregate and RAP were combined in a bucket mixer and then dry blended with ambient temperature RAS (if applicable). After dry blending, the PG 67-22 binder was poured into a divot within the blended aggregate, RAP, and RAS. For the WMA, the Evotherm™ DAT was then poured on top of the binder. The components of the mix were then mixed for approximately two minutes. Mix was then short term aged for two hours at the target compaction temperature. TABLE 8 lists the respective compaction temperatures for the mix design verifications. After short term aging, specimens were compacted to 60 gyrations and theoretical maximum specific gravities were determined (AASHTO T 209). The theoretical maximum specific gravities determined for the HMA and WMA were 2.658 and 2.652, respectively.

TABLE 8 Target Compaction Temperatures for Mix Verifications

Mix Set	HMA	WMA
1	149°C (300°F)	120°C (248°F)
2		95°C (203°F)

The air void contents of the compacted mix verification specimens were determined in accordance with AASHTO T 166. TABLE 9 lists the average air void content for each mix. The rows identified as 15% RAP JMF and 10% RAP 5% RAS JMF relate information from the submitted designs. The air voids obtained during the mix verification differed from the JMF air voids in all cases. Another laboratory that was conducting an independent mix verification observed the same phenomenon of the verification specimens exhibiting a higher percentage of air voids. It was hypothesized that in the year that had passed between the time of the original designs being created and the mix design verifications, the characteristics of the RAP and slag changed thus altering the volumetrics. Based on the volumetrics and recommendations from MeadWestvaco, it was decided that the target compaction temperature of 120°C was the most appropriate for the WMA demonstration in Alabama.

TABLE 9 Air Void Content of Compacted Mix Verification Specimens

Mix	Average Air Voids (%)
15% RAP JMF	3.9
15% RAP HMA	5.7
15% RAP WMA (120°C)	5.0
15% RAP WMA (95°C)	5.7
10% RAP 5% RAS JMF	4.2
10% RAP 5% RAS HMA	4.8
10% RAP 5% RAS WMA (120°C)	5.0
10% RAP 5% RAS WMA (95°C)	6.8

Mix Verification Moisture Susceptibility Results

After the evaluation of the design specimens, moisture susceptibility testing was initiated. It was decided that the remainder of the mix design verification testing would be conducted only for one WMA compaction temperature, 120°C. Specimens were made in accordance with ALDOT 361 and following the mixing procedures previously mentioned for the fabrication of laboratory-produced compacted mix verification specimens.

After proper grouping of specimens by air void content, specimens were tested in accordance with ALDOT 361. TABLE 10 summarizes the average tensile strength results obtained for each of the laboratory-produced mixes. FIGURE 1 illustrates the results from TABLE 10 with the error bars representing the minimum and maximum indirect tensile strengths. In both cases, the HMA specimens exhibited higher tensile strengths and tensile strength ratios than the two WMAs. The mix with the RAS also had the higher of the two tensile strength ratios for the WMAs. The RAS appears to have improved the resistance to moisture damage. Most likely, the stiffer binder from the RAS contributed to the differences in tensile strength and tensile strength ratio.

TABLE 10 Tensile Strength Results for Laboratory Mix

Material	Average Unconditioned Strength (psi)	Average Conditioned Strength (psi)	TSR
DAT RAP (120°C)	159.2	110.3	69
HMA RAP	186.7	165.5	89
DAT RAP+RAS (120°C)	158.7	126.3	80
HMA RAP+RAS	176.1	161.3	92

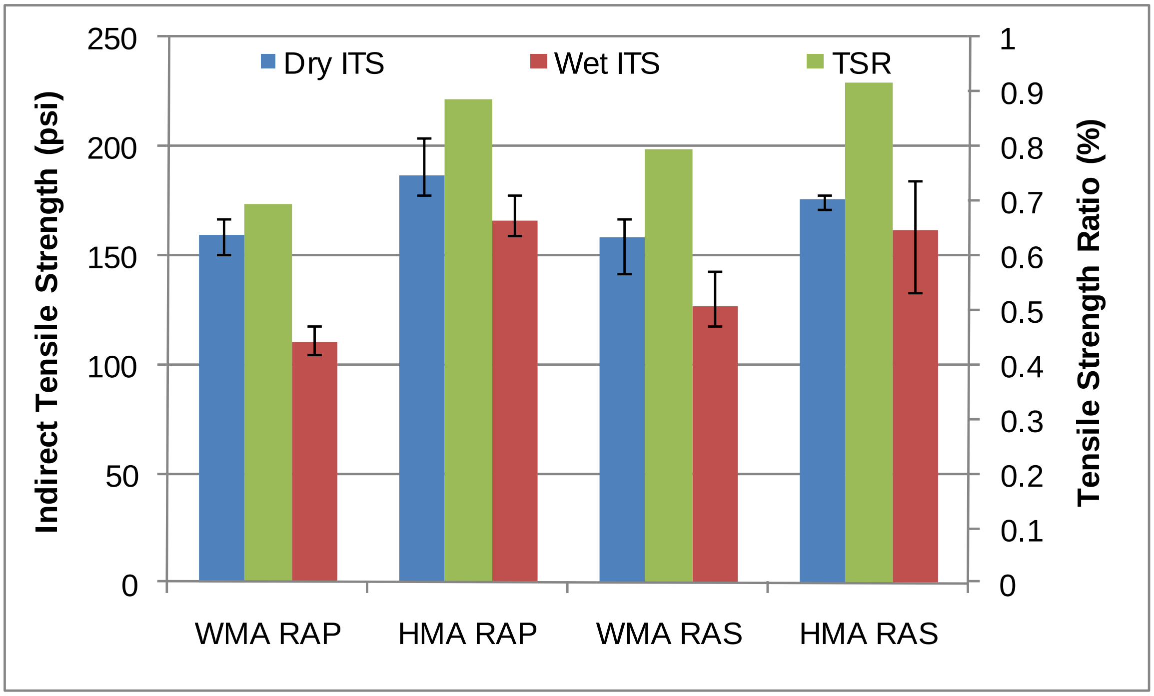

FIGURE 1 Mix Verification Moisture Susceptibility Results

Mix Verification Coating Results

The coating was determined to ensure proper coating was obtained at the WMA temperature. The coating evaluation was conducted in accordance with AASHTO T 195. HMA and WMA mixes were blended at the appropriate mixing temperature in the laboratory. After mixing, material was sieved and the particles evaluated. Any particle that was not completely coated was labeled as partially coated. The percent coated was calculated using the following equation:

$$Percent\ of\ Coated\ Particles = \frac{(Number\ of\ Coated\ Particles) \cdot 100}{Total\ Number\ of\ Particles} \qquad \textbf{Equation 1}$$

The percent coated values for each laboratory-produced mix are listed in TABLE 11. The HMA mixes exhibited higher percent coated values than the WMAs. The trend for all four mixes was the two mixes containing RAS exhibited lower percent coated values than the two without RAS. The lower percent coated values may have been affected by a lack of adequate mechanical agitation of the RAS at a high temperature to completely activate all of the asphalt in the RAS. If the RAS binder was not adequately activated, the lower virgin binder content in the mixes would not have been sufficient to properly coat the virgin aggregate.

TABLE 11 Percent Coated Values

Material	Percent Coated
DAT RAP (120°C)	82.8%
HMA RAP	88.5%
DAT RAP+RAS (120°C)	73.6%
HMA RAP+RAS	76.8%

Summary of Mix Design Verification

Mix design verifications were conducted to determine a) if a WMA can include RAS and b) which WMA mixing temperature was appropriate for the given mix designs. The air void analysis indicated that the lower mixing temperature and the inclusion of RAS both negatively affected the air voids and resulted in higher air void contents. The ALDOT 361 results suggested that the WMA mixes exhibited reduced moisture resistance compared to the HMA. The inclusion of RAS improved the WMA moisture susceptibility results. However, the use of RAS in both the HMA and WMA reduced the percent coated values indicating that the selected temperatures or mixing times may not have been sufficient to result in proper coating. Based on the results of the mix design verifications, ALDOT selected the HMA with RAS and RAP and the WMA with RAP as the two mixes to use as part of the field demonstration. RAS was not used in the WMA due to concerns about the asphalt in the RAS not being activated at the lower temperatures.

Phase II: Documentation of Production and Construction

The production and construction of the four pavement sections, two test strips and two full nights of paving, occurred the evenings of August 26th, 27th, 28th, and 30th 2007. The documentation of the production and construction on these nights is described herein.

Plant Production Documentation

Two mixes, a WMA and HMA, were produced for a WMA demonstration in Tarrant City, Alabama. A test strip was constructed for each of the mixes to establish a rolling pattern. Following the nights of paving the two test strips, two full nights of paving occurred for each mix.

The first night of paving, August 26th, was the production of the HMA for the HMA test strip. Originally, the WMA test strip was scheduled for the first night. However, due to issues with the portable distribution pump for the Evotherm™ DAT, the WMA test strip was delayed to August 27th. The target mixing temperature was 163°C (325°F). On August 26th, mix temperatures were taken and recorded; these temperatures ranged from 146 to 179°C (295 to 355°F).

On the second night of paving, August 27th, the WMA test strip mix was produced. The target mixing temperature was 121°C (325°F), which was lower than the temperature employed during the mix design verifications. Production was delayed due to an amperage issue concerning the slat chain. It was believed that the lower production temperatures were causing a strain on the slat conveyor, increasing the amperages needed to keep the slat moving. This increase eventually caused the emergency shut-off switch to activate, causing a slight delay until this could be resolved. When production started, approximately 500 tons of WMA were produced, with an average load-out temperature of 124°C (255°F).

On the third night of paving, August 28th, the WMA for the full night of paving was produced. Initially, the target mixing temperature was 121°C (325°F); however, the mixing temperature was reduced throughout the night. During production, the mix temperature at load-out began at 116°C (240°F), eventually decreasing to a temperature of 107°C (225°F) by the end of the night. One of the major observations made during the production of WMA was the lack of fumes as the mix was being loaded into the truck. This can be seen in FIGURE 2. Production of WMA averaged between 200-225 tons per hour.

The final HMA pavement section could not be produced until August 30th due to a plant issue that occurred during the daytime paving operations. The production temperature on the fourth night ranged between 168 to 179°C (335 to 355°F).

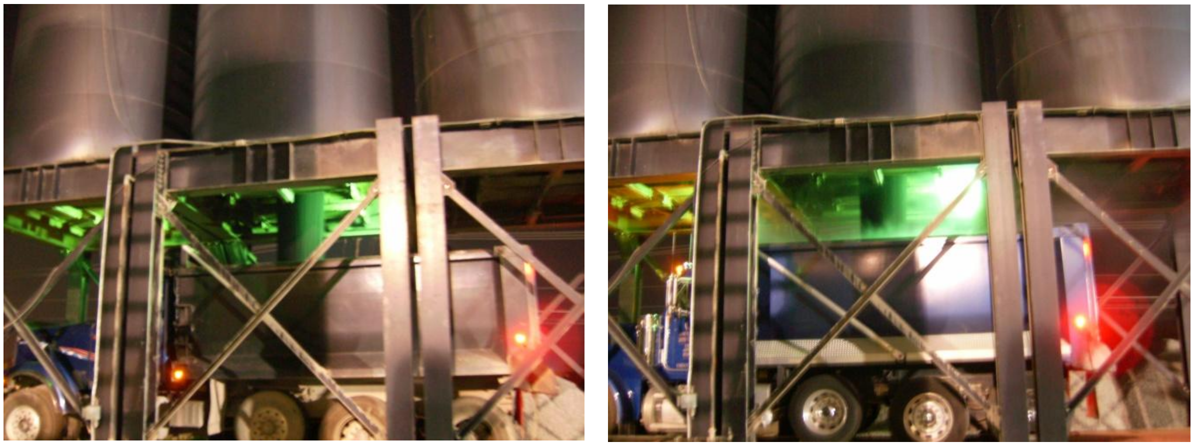

FIGURE 2 Mixture Being Loaded into Trucks -- WMA (Left) and HMA (Right)

Placement Documentation

Site Information

The site was located northwest of Tarrant City, Alabama along SR-79. The project was between milepost 14.1 and 19.5. The majority of the project was a two-lane highway. The WMA and associated HMA control mixes were placed in the southbound lane. The northbound lane was also paved, but only HMA was placed in that lane and was not documented as part of this study. FIGURE 3 illustrates the location of the site; the section of SR-79 highlighted in red is the location of the project.

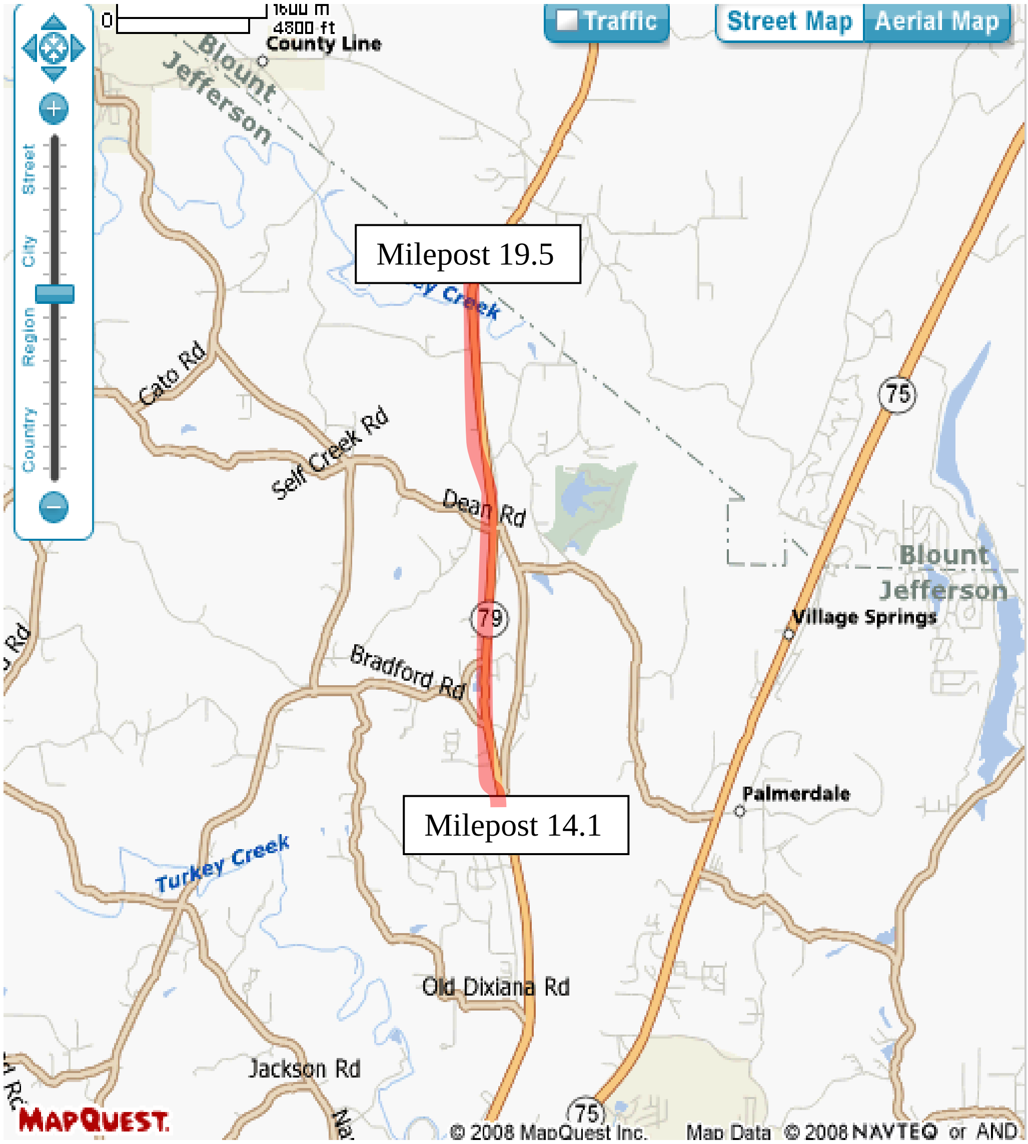

FIGURE 3 Map of Project Site

Roadway Observations

A material transfer device, RoadTec SB 2500, was used all four nights of paving. Two steel drum rollers were used for compaction.

On August 26th, the HMA test strip was constructed. Approximately 500 tons were placed. The temperature behind the screed ranged between 131 to 134°C (268 to 274°F). The rolling pattern was five passes by the breakdown roller in vibratory mode and two passes with the finishing roller in static mode. A pass was defined as a roller traversing a set spot once.

On August 27th, the WMA test strip section was placed. Approximately 500 tons of WMA was placed. The average temperature behind the screed was 104°C (220°F). The roller pattern established was the same as the one for the HMA. No smoke was observed during the compaction of the WMA, from the truck, from the material transfer device, or from the paver. The paving crew commented that the WMA mat looked more consistent than the HMA mat from the previous night.

On August 28th, approximately 1000 tons of WMA were placed. The temperature behind the screed ranged between 74 to 103°C (165 to 217°F). The average surface temperature of the milled mat was 32°C (90°F). Like the paving on the previous day, no smoke and reduced odors were noted. The rolling pattern was inconsistent throughout the night. The lute man noted that the workability of the mix was comparable to that of HMA, even at the lower temperatures. A major observation made by the screed man was that the excess material accumulated in the corners of the auger "broke free," it fell apart, and mixed well with the rest of the mix. He also commented that typical HMA would tend to stay clumped together. Overall, the attitude of the paving crew was positive.

On August 30th, HMA was placed. The temperature behind the screed ranged between 124 to 143°C (255 to 289°F). The rolling pattern was the same as was established on August 26th.

Weather

It should be noted that the weather the week of paving was wet. It rained sporadically throughout the four days of paving. TABLE 12 summarizes the weather data collected at the Birmingham airport, which was the closest weather station to the site.

TABLE 12 Summary of Weather during Construction

Day	Mix Section	Observed Inches of Precipitation	Minimum Temperature	Maximum Temperature	Average Wind Speed (MPH)
8/26/2007	HMA Test Strip	0.89	74	94	3.4
8/27/2007	WMA Test Strip	0.02	74	94	3.3
8/28/2007	WMA Paving	0.09	74	90	3.8
8/29/2007	No Paving	0.61	73	92	2.7
8/30/2007	HMA Paving	0.33	73	92	3.7

Densities

Densities were determined from measurements taken on cores extracted from the pavement. The field core data during construction was obtained from ALDOT. TABLE 13 lists the densities reported by ALDOT. The WMA test strip exhibited the highest densities, while the WMA full night of paving yielded the lowest densities. The variability of the core densities for the WMA test strip night was low, but the variability for the full night of WMA paving was more than twice that of either HMA nights of paving. It would appear that the leveling course under the WMA test strip may have had an effect on the in-place densities since it had the highest densities compared to the other sections which did not have a leveling course.

TABLE 13 Field Core Densities at the Time of Construction

Day	Mix Section	Under Pavement Surface	Density	Average Density	Density Standard Deviation
8/26/2007	HMA Test Strip	Milled	93.82%	92.89%	0.84%
			91.96%		
			92.45%		
			93.34%		
8/27/2007	WMA Test Strip	Leveling	93.44%	93.98%	0.45%
			94.18%		
			94.48%		
			93.83%		
8/28/2007	WMA Full Section	Milled	90.80%	90.59%	1.74%
			91.97%		
			88.08%		
			91.50%		
8/30/2007	HMA Full Section	Milled	92.07%	91.97%	0.74%
			92.35%		
			92.89%		
			92.33%		
			91.24%		
			90.95%		

Cooling Rate of the Mat

The temperature of the mat was monitored using an infrared temperature gun. The pavement temperature was monitored with time to evaluate the cooling of the mat during the compaction process. A spot was marked on the pavement and the temperature of that spot was recorded during the compaction process. Temperature readings were obtained approximately every 2 minutes immediately before and after each roller pass. FIGURE 4 illustrates the change in temperature with time. The WMA was delivered to the site at a lower temperature and appeared to stabilize at a lower temperature than the HMA.

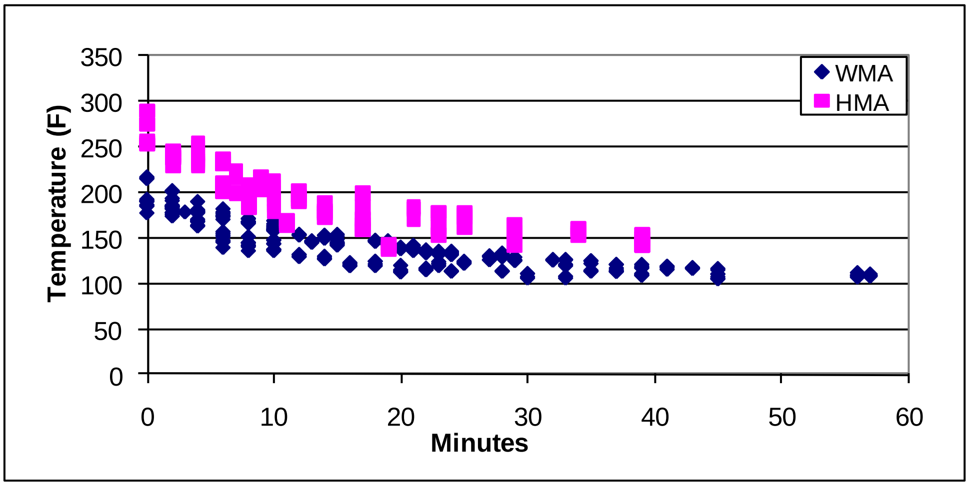

FIGURE 4 Temperature of the Mat with Time

Summary of Documentation of Production and Construction

Four nights of production and construction of HMA and WMA were documented. A test strip for each mix was constructed to establish a rolling pattern. Each mix was then produced for a full night of paving. The WMA was produced approximately 24°C (75°F) cooler than the HMA. There were no issues placing either mix. However, the densities of both HMA nights of paving and the full night of WMA paving could have been improved with an increased compaction effort and possibly a leveling course. The cooling rates of the mixes indicated that the HMA initially did cool more rapidly than the WMA.

Phase III: Laboratory Testing of Field Mix

The material collected and compacted in the field was evaluated at the main NCAT laboratory. Asphalt content, asphalt binder properties, and aggregate gradations were determined from the loose mix sampled. The specimens compacted in the field were used to evaluate the moisture susceptibility, rutting resistance, and cracking potential.

Additional specimens were compacted at the main NCAT laboratory from the plant-produced mix for dynamic modulus and flow number testing.

Extraction and Recovery Data

Asphalt was extracted and recovered from the four nights of paving. One sample was collected for each test strip. Two samples were collected for each full night of paving. The first sample was collected at the beginning of production and the second sample was obtained towards the end of production. A sample of the RAP was obtained for extraction and recoveries also.

Asphalt Content

Two asphalt contents were determined for each sample collected. The average asphalt content of the RAP was 4.07%. The average asphalt content of the HMA and WMA mixes are displayed in TABLE 14 and FIGURE 5. The error bars represent the difference between the two asphalt contents determined for each sample. The WMA consistently exhibited higher asphalt contents than the HMA. This difference in asphalt content may result in improved durability for the WMA in comparison to the HMA, but may increase the rutting potential. The asphalt contents for the full section nights for both the WMA and HMA were both lower than the two test strip nights. The difference in HMA asphalt content may be a result of the shingles not being blended homogeneously.

TABLE 14 Asphalt Contents

Mix	Sample Set Number	Average Asphalt Content	Asphalt Content Difference Between Samples
HMA (Aug. 26 -- Day 1)	1	4.90%	0.18%
WMA (Aug. 27 -- Day 2)	1	5.77%	0.01%
WMA (Aug. 28 -- Day 3)	1	4.83%	0.33%
	2	5.08%	0.08%
HMA (Aug. 30 -- Day 4)	1	4.66%	0.11%
	2	4.80%	0.08%

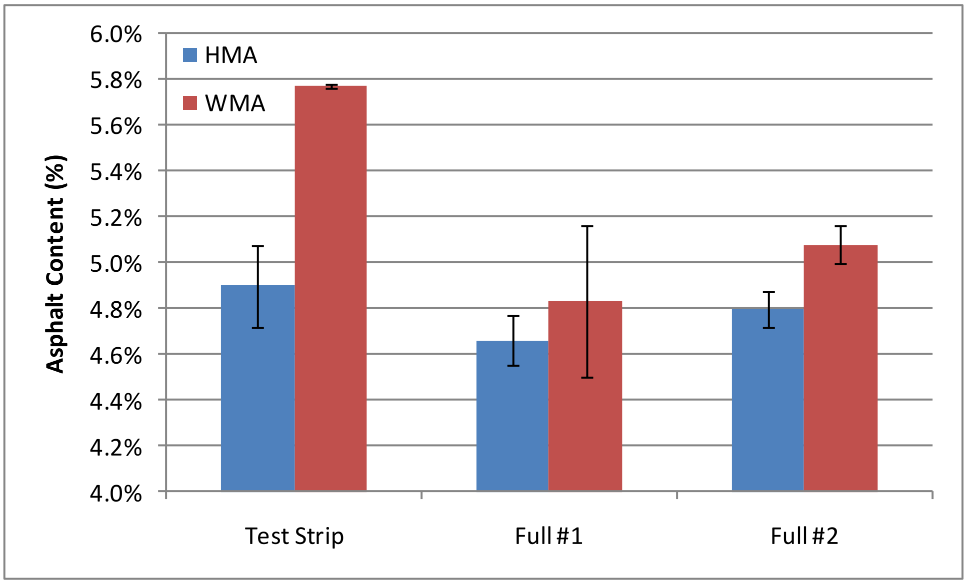

FIGURE 5 Asphalt Contents of Plant-produced Mixes

Recovered Binder

The base binder for the field mixes was the same PG 67-22 that was used in the laboratory mixes. Binder was extracted and recovered from mix and RAP samples. The extraction process was conducted in accordance with AASHTO T 319 and the recovery process followed ASTM D 5404. One set of extraction-recoveries was conducted for each test strip night and at least two sets of extraction-recoveries were conducted for each full night of paving.

The recovered binders were classified using the Superpave performance grading system in accordance with AASHTO R 29. TABLE 15 lists the binder performance grades determined for each sample. The WMA consistently graded out as a PG 70-22. The increase in stiffness from the base binder, the PG 67-22, was most likely a result of the RAP and oxidation that occurred during production. The HMA grading varied. It is hypothesized that the shingles were not blending adequately which would result in some samples with a higher percentage of shingles than other samples. Regardless of the sample, the HMA samples were stiffer on both the high and low ends of the performance grade classification than the WMA due to the higher mixing temperature and shingles.

TABLE 15 PG Binder Classification

Material	Sample Number	Continuous Binder Grade	Specification Binder Grade
HMA (Day 1)	1	90.7 -16.6	88 -16
	2	79.1 -19.4	76 -16
WMA (Day 2)	1	70.2 -26.0	70 -22
	2	70.4 -25.5	70 -22
WMA (Day 3)	1	74.2 -24.9	70 -22
	2	74.3 -24.9	70 -22
	3	73.3 -25.3	70 -22
	4	74.0 -25.6	70 -22
HMA (Day 4)	1	95.4 -13.5	94 -10
	2	81.4 -20.5	76 -16

Recovered Aggregate

Sieve analyses were conducted on the recovered aggregate for each night. The blended aggregate gradations for all of the mix samples are displayed in FIGURE 6. The aggregate gradations varied substantially. FIGURE 7 displays the blended aggregate gradations for the WMA samples and the JMF gradation. It can be seen that the test strip aggregate gradation was substantially finer than the WMA samples from the full night of paving. The samples from the full night of paving were most similar to the JMF. FIGURE 8 illustrates the aggregate gradations of the HMA samples along with the JMF. The recovered aggregate gradation of the HMA test strip night was substantially coarser than the JMF and HMA full night of paving aggregate gradations. The full night of HMA paving aggregate gradations were finer than the JMF. These substantial differences in gradation between the test strips and full nights of paving may partially explain the differences in density observed in the field.

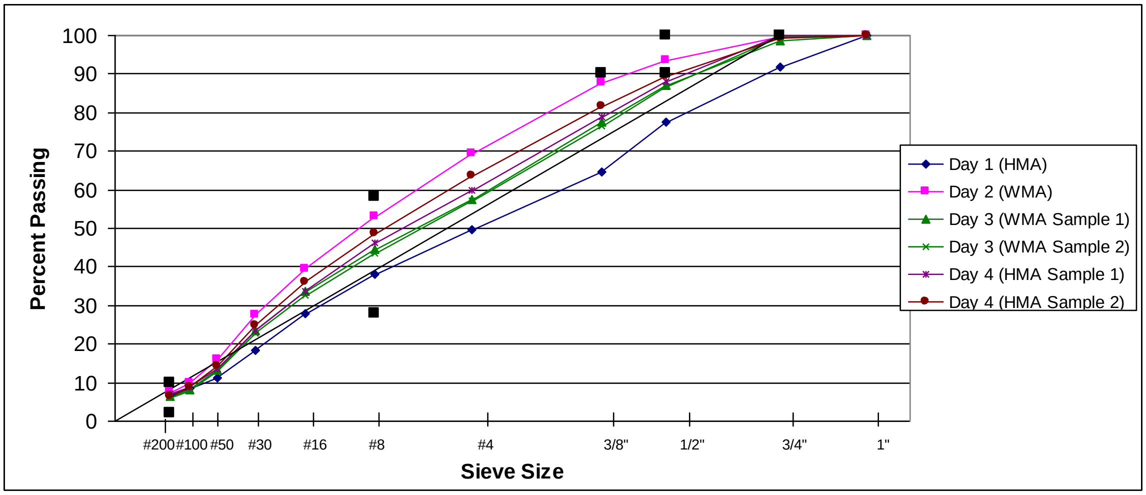

FIGURE 6 Aggregate Gradation of Recovered Aggregates

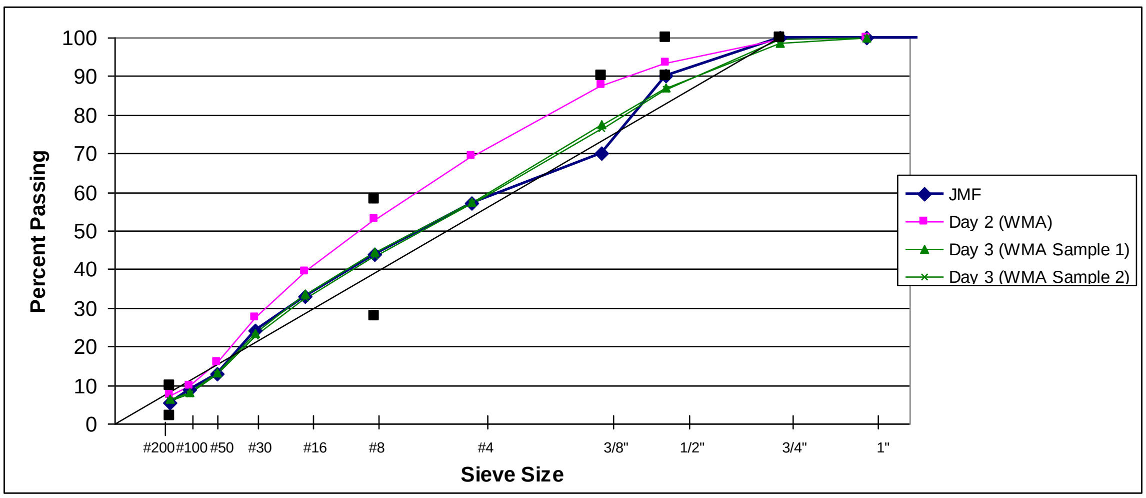

FIGURE 7 Aggregate Gradation of WMA

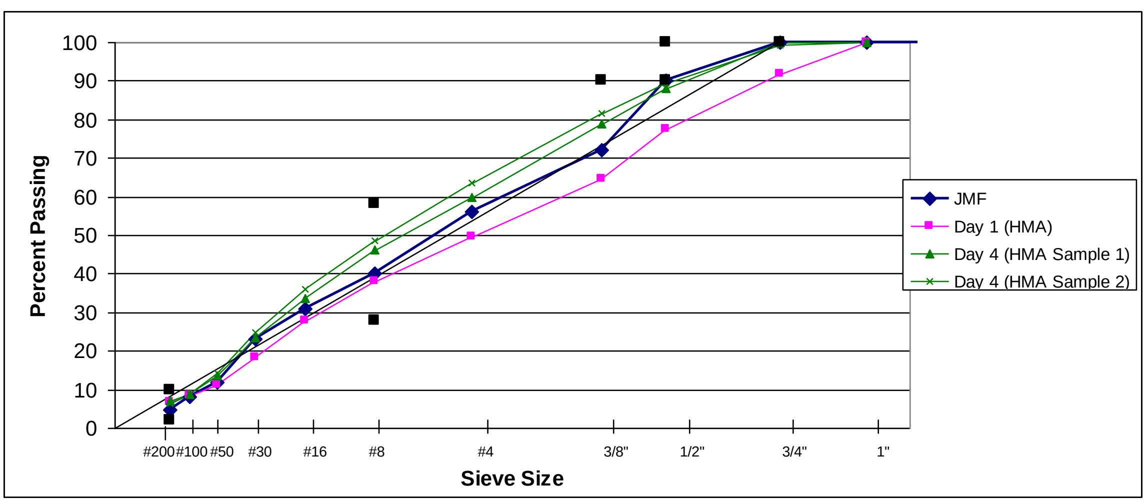

FIGURE 8 Aggregate Gradation of HMA

Moisture Susceptibility Test (ALDOT 361)

The moisture susceptibility testing was conducted in accordance with Alabama Department of Transportation specifications, ALDOT 361, *Resistance of Compacted Bituminous Mixture to Moisture Induced Damage*. The peak load measure on each specimen was recorded and used to calculate the indirect tensile strength. The average tensile strength values of the conditioned and unconditioned specimens were calculated and the ratio of average conditioned tensile strength to average unconditioned tensile strength was determined. TABLE 16 summarizes the tensile strength ratio data for the plant-produced mixes. FIGURE 9 illustrates the results reported in TABLE 14 with the error bars representing the minimum and maximum indirect tensile strength values. All but one plant-produced mix sample, WMA Day 3 Sample 1, exceeded the 80% tensile strength criterion. The indirect tensile strengths of the WMA specimens were significantly lower than those for the HMA. The differences in binder grade and fibers from the shingles in the HMA most likely were the main causes of the differences in indirect tensile strengths.

TABLE 16 Plant Mix Tensile Strength Ratios

Mix	Sample	Average Air Voids (%)	Condition	Percent Saturation	Average Tensile Strength (psi)	Average Tensile Strength Ratio
HMA (Day 1)	1	6.8	Conditioned	60.1	123.6	85%
		7.1	Unconditioned	--	145.4	
WMA (Day 2)	1	6.8	Conditioned	65.9	93.7	84%
		6.7	Unconditioned	--	111.4	
WMA (Day 3)	1	7.0	Conditioned	58.0	75.2	71%
		7.0	Unconditioned	--	105.2	
WMA (Day 3)	2	5.5	Conditioned	56.2	50	88%
		5.8	Unconditioned	--	56.6	
HMA (Day 4)	1	7.1	Conditioned	74.4	127.1	91%
		7.1	Unconditioned	--	139	
HMA (Day 4)	2	8.9	Conditioned	58.0	141.5	94%
		8.7	Unconditioned	--	151.2	

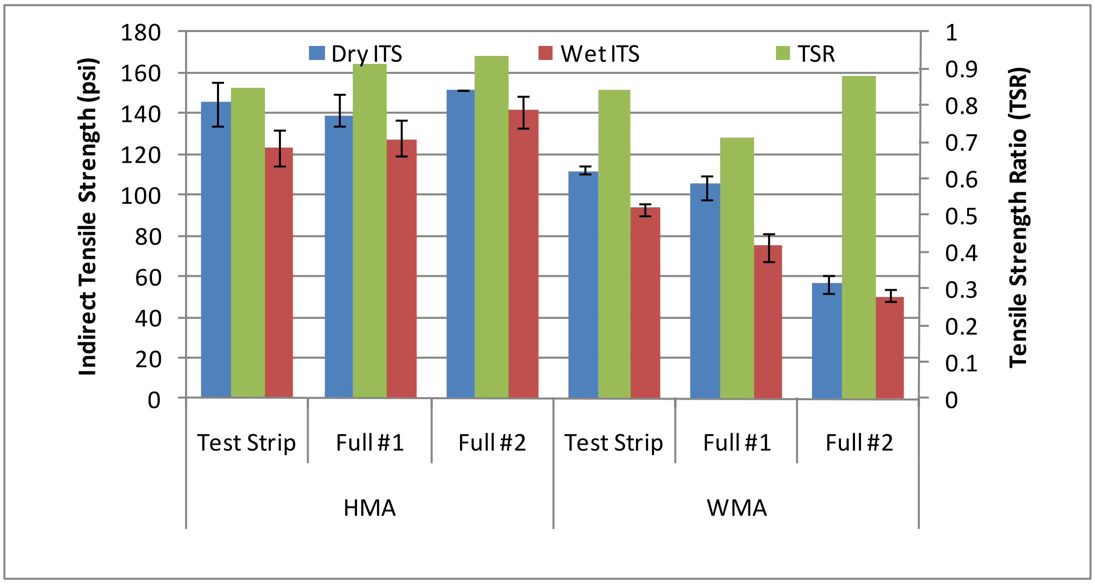

FIGURE 9 Moisture Susceptibility Results of Plant-produced Mix

Analysis of variance (ANOVA) was conducted to determine if the use of a WMA additive significantly affected the indirect tensile strength results. For both the unconditioned and conditioned specimens, producing the mix as a WMA significantly affected the indirect tensile strength results.

Asphalt Pavement Analyzer Rut Test (AASHTO TP 63)

The APA rut test is a common test conducted to evaluate the potential for a mix to rut. Rut testing was conducted primarily to determine if the WMA would be prone to rutting. TABLE 17 and FIGURE 10 summarize the APA rut depth results. The rut depths reported are from manual measurements. The whiskers in FIGURE 10 represent plus and minus one standard deviation of six samples. The WMA test strip mix rutted the most and this is most likely the result of the high asphalt content. In general, the WMA rutted slightly more than the HMA; however, the difference was minimal. The difference in rutting could be the result of higher asphalt content and softer binder in the WMA or simply test variability.

ANOVA was conducted to identify if the mix type significantly affected the variability of the APA rut depth results. The results of the ANOVA indicated that mix type did affect the APA rut depth results. Tukey's mean comparisons were conducted to determine which mixes had statistically different mean APA rut depths. The results of the mean comparison are listed in TABLE 18. The rut depths from the majority of mixes were different from one another.

TABLE 17 Average APA Rut Depths

Mix	Sample	Average Rut Depth (mm)
HMA (Day 1)	1	1.89
WMA (Day 2)	1	4.97
WMA (Day 3)	1	2.67
WMA (Day 3)	2	4.07
HMA (Day 4)	1	2.33
HMA (Day 4)	2	3.56

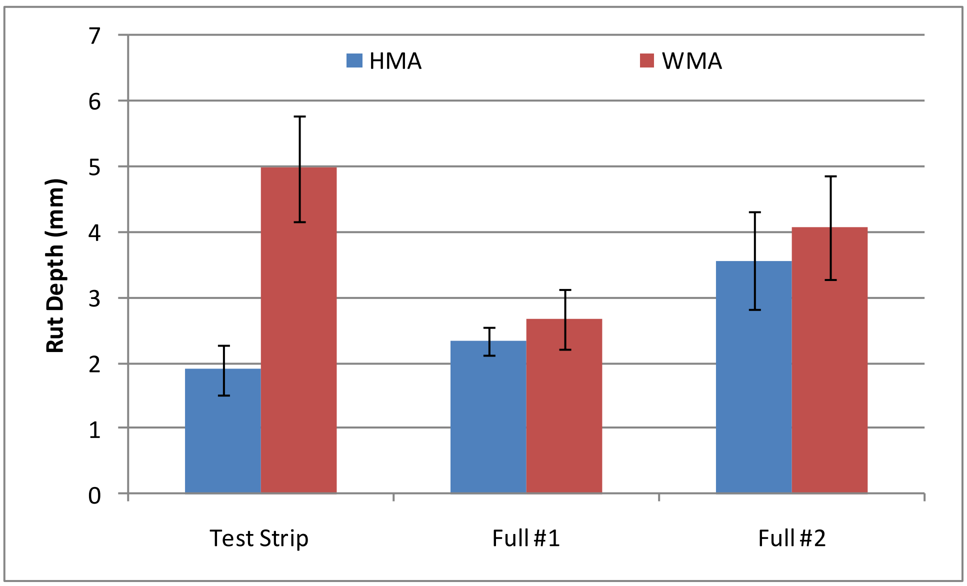

FIGURE 10 APA Results

TABLE 18 Results of Mean Comparisons of APA Rut Depths

Comparison Between	Statistically Significant Different Mean Rut Depths
Aug. 26 v. Aug. 27	✓
Aug. 26 v. Aug. 28	
Aug. 26 v. Aug. 30	✓
Aug. 27 v. Aug 28	✓
Aug 27 v. Aug 30	✓
Aug 28 v. Aug 30	

Hamburg Wheel Tracking Test (AASHTO T 324)

The Hamburg wheel tracking test was also used to evaluate the moisture susceptibility of the plant-produced mixes. TABLE 19 lists the stripping inflection points and air void contents for each mix. HMA specimens from Day 4 Sample 2 (full night of paving) were not tested due to the excessively high air voids and compacting a new set of specimens from reheated mix would have affected the results. The HMA consistently exhibited stripping inflection points that exceeded 10,000 cycles. The test stops after 10,000 cycles; therefore, the stripping inflection points for those specimens could not be determined other than to state it was greater than 10,000 cycles. The WMA stripping inflection points ranged between 5,000 and greater than 10,000 cycles. The lower stripping inflection points for the WMA may indicate that the WMA is less resistant to stripping. It should be noted that one of the specimens from WMA Day 3 did not have a stripping inflection point. A stripping inflection point of 5,000 cycles is typically considered acceptable; therefore, both mixes exhibited acceptable stripping resistance.

TABLE 19 Hamburg Stripping Inflection Points of Plant-produced Mix

Material	Sample Number	Average Air Void Content (%)	Stripping Inflection Point (Cycles)
HMA (Day 1)	1	7.1	≥10,000
WMA (Day 2)	1	6.8	6860
WMA (Day 3)	1	7.3	6050
	2	6.5	5291
HMA (Day 4)	1	7.2	≥10,000
	2	9.5	Not Tested

TABLE 20 lists the total rut depths obtained from the Hamburg wheel tracking test. The WMA resulted in greater rut depths than the HMA, which is similar to the APA rut results. The Hamburg testing suggested that the WMA is more susceptible to rutting than the HMA, this could partially be attributed to the RAS in the HMA. Both mixes passed the Hamburg total rut depth requirement of at most a 10 mm rut depth.

TABLE 20 Hamburg Total Rut Depth at 10,000 Cycles

<table>
<tr><th>Material</th><th>Sample Number</th><th>Average Air Void Content (%)</th><th>Total Rut Depth at 10,000 Cycles (mm)</th></tr>
<tr><td>HMA (Day 1)</td><td>1</td><td>7.1</td><td>1.1</td></tr>
<tr><td>WMA (Day 2)</td><td>1</td><td>6.8</td><td>15.0</td></tr>
<tr><td rowspan="2">WMA (Day 3)</td><td>1</td><td>7.3</td><td>5.2</td></tr>
<tr><td>2</td><td>6.5</td><td>5.1</td></tr>
<tr><td rowspan="2">HMA (Day 4)</td><td>1</td><td>7.2</td><td>1.7</td></tr>
<tr><td>2</td><td>9.5</td><td>N/A</td></tr>
</table>

Dynamic Modulus (AASHTO TP 62)

Dynamic modulus testing is a test that can relate the stiffness of a mix. In this study, the dynamic modulus tests were used to compare the stiffness of the WMA to the HMA. Master curves were developed to compare the response of the HMA to that of the WMA. Master curves are developed by shifting dynamic modulus test results from different testing temperatures and frequencies to form one smooth curve. A reference temperature of 21.1°C (70°F) over several frequencies was used to develop the master curves. Insufficient material was collected on the first day of testing to fabricate specimens. FIGURE 11 illustrates the results of the dynamic modulus testing for the remaining days. The HMA exhibited higher dynamic modulus values (E*), thus indicating that it was stiffer than the two days of WMA. The use of RAS in the HMA most likely was the main cause of the stiffer mix.

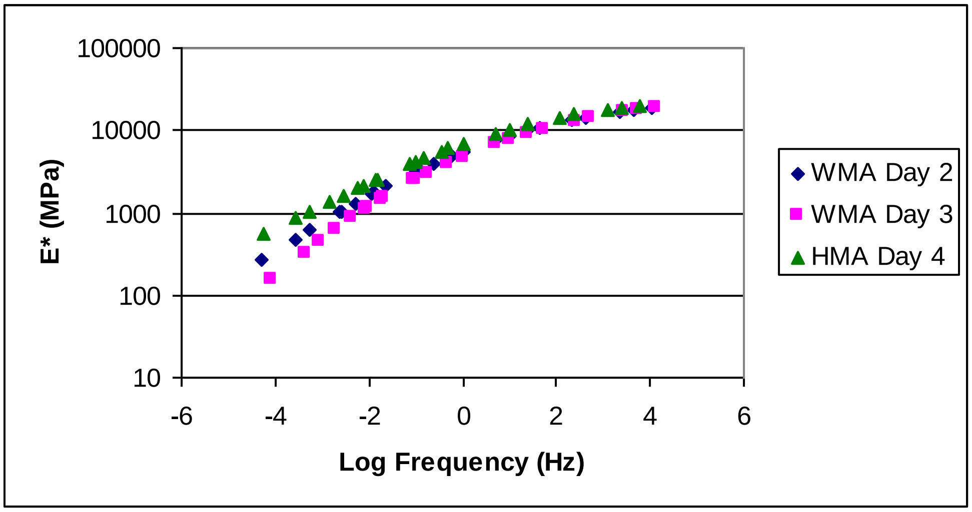

FIGURE 11 Dynamic Modulus Summary Graph

ANOVA was used to determine if mix type or day of production significantly affected the variability of the dynamic modulus results for a given temperature and frequency. The day of production was not significant for any given temperature and frequency. The mix type was significant for the intermediate temperatures for the majority of frequencies. In the cases where there was a significant difference between the WMA and HMA specimens, the HMA was significantly stiffer than the WMA.

Linear Elastic Analysis

ALDOT provided FWD deflections on the north-bound lane between these mile-points 14.1 and 19.5 as shown in Appendix A. An analysis was done by ALDOT as part of the overlay design of the resurfacing (using the DARWin Pavement Design Analysis software). The back-calculated moduli determined by ALDOT can be found in Appendix A Figure 28. It was reported by ALDOT that "minor level transverse cracking was present at intermittent locations [along the section]" and that "minor longitudinal cracking was present in the wheel paths in some areas." Based on their analysis, ALDOT recommended milling the in-place pavement to a thickness of 1.5 inches and overlaying with hot-mix asphalt (HMA).

The in-place pavement structure consisted of an 8 inch HMA pavement over a 12 inch stone aggregate base over the subgrade. The subgrade moduli reported were used for design purposes. The back-calculated subgrade moduli as calculated by ALDOT for the section range from 7,000 – 40,000 psi with an average and standard deviation of about 23,000 psi and 6,200, respectively. The FWD deflections were used as input for the back-calculation software MODCOMP 5, which employs linear elastic theory to iteratively vary layer moduli to match surface deflections. MODCOMP indicated the presence of a stiff layer in the lower subgrade at a depth of about 100 – 150 inches and estimated the subgrade moduli to be on average 25,000 psi. Using MODCOMP, the back-calculated moduli of the surface and base layers were on average in the order of 400,000 psi and 12,000 psi, respectively, with a modulus ratio (E_1/E_2) of about 33. The "stiffer" subgrade moduli relative to the base was also confirmed from surface modulus plots as shown in Appendix A Figure 29 that shows a stiffening of the pavement structure with depth. While this behavior may be attributed to a nonlinear stress-dependant response of the base and subgrade materials, the results do indicate a relatively weak base structure underlying a relatively stiff surface layer.

The back-calculation of moduli using the linear-elastic component of MODCOMP indicated that the base modulus of the section evaluated was significantly lower than the subgrade modulus. This finding was unexpected and suggested an inverted pavement structure. Typically, pavement moduli decrease with depth from the surface. The type of response has been reported previously by Dr. Dave Timm upon evaluating pavement structures at the NCAT Test Track. Indications are that the base and subgrade materials are perhaps stress-dependent and therefore cannot accurately be evaluated using linear elastic theory.

The FWD analyses do suggest, however, that the stiffness of the base layer is considerably lower than the HMA surfacing above it and could be the reason for the (fatigue) cracking observed on the surface and the need for rehabilitation of the pavement. If this is the case then it is possible that the cracking observed on the surface runs through the 8 inches of HMA that was milled for the resurfacing.

Laboratory determined dynamic moduli for an asphalt mixture are generally considerably higher than FWD back-calculated moduli for the same mixture at comparative temperatures. However, based on the dynamic modulus results obtained from the plant-produced mix, it appears that the mix placed has a stiffness that is as stiff or stiffer than the mix replaced. A much stiffer mixture would reduce deflections of the structure and consequently extend its service life. However, an overly stiff mixture may be susceptible to cracking given the weaker base and the possibility that the HMA layer beneath the 1.5 inch resurfaced mixture is already cracked.

An analysis of these results indicated that the modulus of the 12 inch base layer is considerably lower than that of the 8 inch asphalt layer above it. This difference in modulus could result in high tensile stresses developing beneath the asphalt layer and cracking observed in the wheel paths on the surface prior to rehabilitation of the section suggests that the asphalt layer is already fatigued. Dynamic modulus tests were done on the resurfacing HMA and WMA. Both resurfacing mixtures will not only be subjected to bending given the relatively weak base layer but also reflective cracking through the underlying asphalt layer that may already be fatigued.

Flow Number

Flow number testing was conducted to evaluate the susceptibility to permanent deformation of each mix. There was insufficient material to make specimens for the first day of paving. The average flow number values are illustrated in FIGURE 12. The HMA results were higher than the WMA results indicating that the WMA may be less resistant to permanent deformation. Several factors probably contributed to the difference in flow number values; such as softer asphalt in WMA, varying asphalt contents, use of RAS in the HMA, and varying aggregate structures.

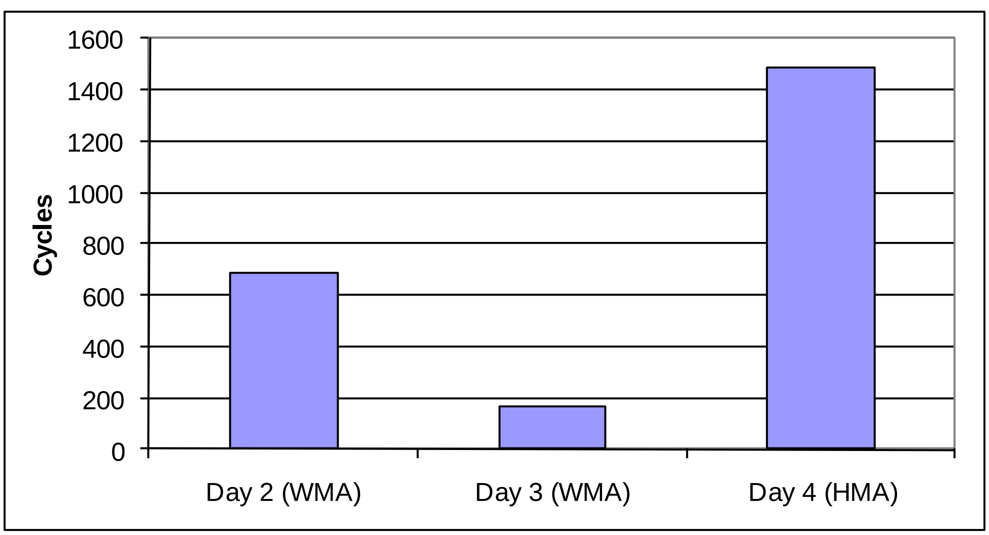

FIGURE 12 Flow Number Test Results

ANOVA was conducted to identify if mix type or day of production significantly affected the variability of the flow number results. Results showed that both did significantly affect the flow number results. The differences in asphalt contents and aggregate gradations most likely contributed to these dissimilarities. Tukey's mean comparisons were conducted to determine which days of production resulted in significantly different mean flow numbers. All days of production were identified as significantly different from one another.

IDT Creep Compliance and Strength (AASHTO T 322)

The indirect tensile creep tests were performed at three different temperatures: -20, -10, and 0°C (-4, 14, and 32°F). The rate of creep compliance was determined from the test to describe quantitatively the rate at which microcracks will develop. The rates of creep compliance were determined at the three test temperatures and are illustrated in FIGURE 13. The rates of creep compliance showed a good correlation between mixes at -10 and 0°C (-4 and 14°F). It is hypothesized that -20°C (32°F) was close to the glass transition temperature indicating that the viscoelastic response of the asphalt mixtures was not dominant at that temperature. Since the viscoelastic response was not dominant, the performance evaluation of the mixtures at that temperature may not be suitably estimated through viscoelastic fracture mechanics; therefore, the data was not included in the performance evaluation.

From the comparison of the creep rates between the mixtures at -10 and 0°C (-4 and 14°F), the rate of creep of the WMA test strip mix was the highest, while that of the HMA full night of paving mix was the lowest among the mixtures. The result indicates that the mix from the WMA test strip night is less resistant to load induced damage (e.g. fatigue cracking) compared to the other mixes placed. The HMA from the full night of paving showed the lowest creep rate among the mixtures suggesting it is the most resistant to fatigue cracking. However, the conclusion was derived from the result of the creep compliance tests alone. The fracture of asphalt mixtures are essentially governed by the rate of energy dissipation (*10* and *11*). The rate of energy dissipation is determined by the rate of creep compliance and dissipated energy threshold. The dissipated energy threshold can be determined from indirect tensile strength at -10°C (14°F). However, a software issue resulted in the indirect tensile strength values being unusable. There was insufficient mix to remake new specimens for creep compliance and strength testing.

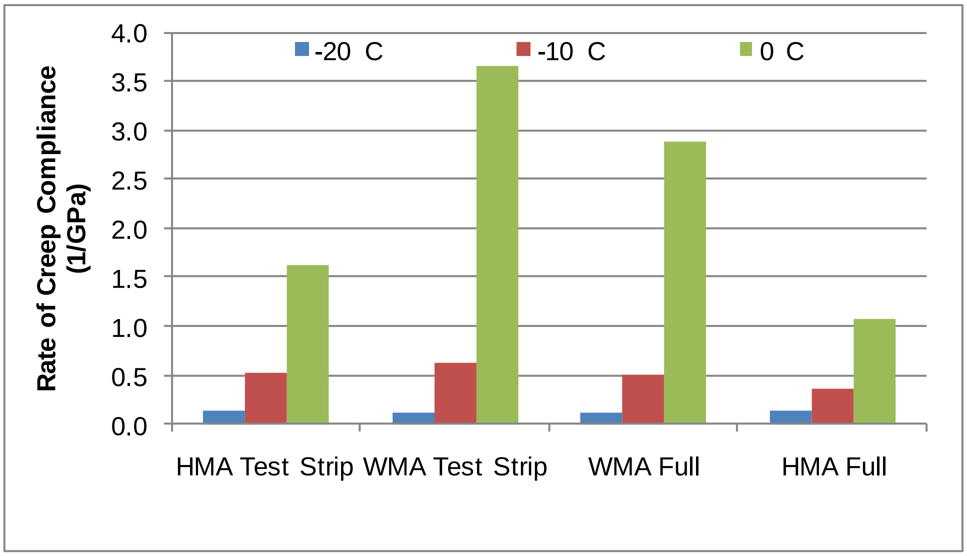

FIGURE 13 Rate of Creep Compliance

Summary of Laboratory Testing of Field Mixes

The extractions and recoveries revealed that the asphalt contents and aggregate gradations of the plant-produced mixes varied from one another. The recovered binder properties for the WMA were consistent and graded as a PG 70-22. The recovered binder properties of the HMA were inconsistent, but stiffer than the WMA in all cases. The moisture susceptibility testing indicated that the indirect tensile strengths of the WMA are lower than those of the HMA and that the WMA may be more moisture susceptible than the HMA. The APA results indicated that the WMA may be more susceptible to rutting than HMA; however, all mixes exhibited acceptable rut depths for low volume roads. The Hamburg testing confirmed that the WMA was more moisture and rut susceptible than the HMA. The dynamic modulus testing suggested the HMA was stiffer than the WMA, which was expected since the HMA contained the RAS. The flow number results confirmed the findings of the APA and Hamburg that the WMA is more susceptible to rutting. The creep compliance testing indicated that the WMA is less resistant to load-induced damage than the HMA. Overall, the WMA performed differently than the HMA; however, in most cases the WMA still met typical mix criteria for acceptance.

Phase IV: Site Revisits

Three site re-visits were made. The first and second site revisits consisted of visual inspections. The third site revisit included cutting of cores along with the visual inspections.

At the first site revisit, mix segregation was noted throughout all of the pavement sections. No cracking was observed. No other distresses were observed.

At the second site revisit distresses were observed and photographed. FIGURE 14 illustrates the mix segregation that was observed on the project. The picture is from the HMA test strip section; however, mix segregation was observed in all of the sections.

FIGURE 14 Mix Segregation in HMA

A crack was observed in the WMA full night of paving section. FIGURE 15 is a photograph of the crack observed. FIGURE 16 is a close up photograph of the crack. Soil was pumping up through the crack, suggesting that there may be a structural failure that caused the crack.

FIGURE 15 Crack in WMA Full Night of Paving Section

FIGURE 16 Close up of the Crack in the WMA Full Night of Paving Section

A second crack was observed in the WMA full night of paving section. The crack stretched across the centerline and spanned part of the WMA section and a HMA section. The HMA section was paved several days after the WMA section, but was not part of the sections included in the study. Soil was pumping up through the crack on the HMA side, suggesting there was an underlying structural issue. FIGURE 17 is a photograph of the observed crack.

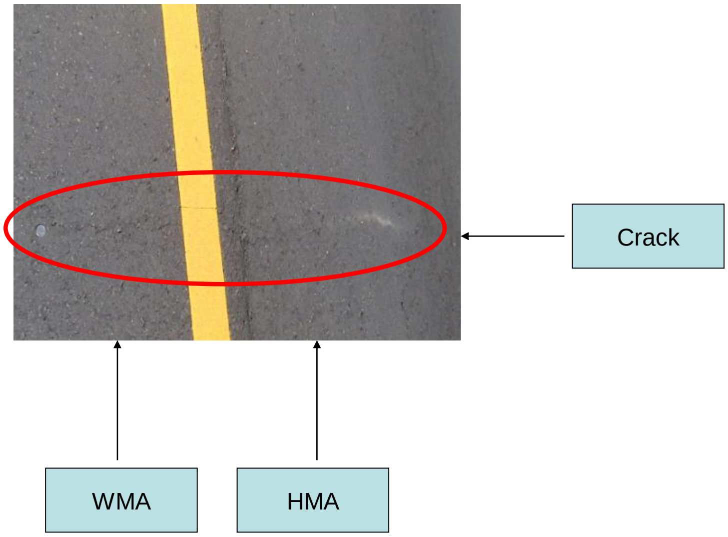

FIGURE 17 Second Crack Observed in WMA Full Paving Section

A large patch was observed in the HMA full night of paving section. FIGURE 18 displays the patch. Efforts to identify the reason for the patch were unsuccessful.

FIGURE 18 Patch in the HMA Full Night of Paving Section

The joints between the HMA and WMA sections were also documented. FIGURE 19 displays the joints. The segregation in the HMA test strip section was evident and clearly defined the end of the HMA test strip section and the start of the

WMA section. The difference between the two full nights of paving was not as apparent, which made it difficult to locate the joint.

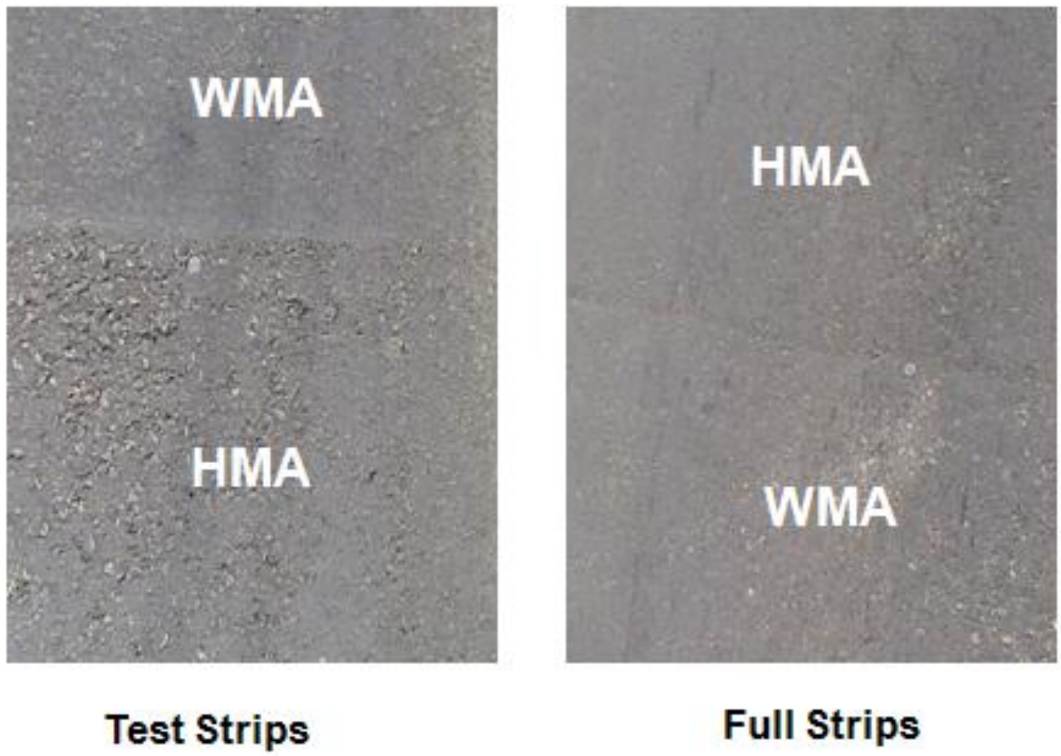

FIGURE 19 Joints Between HMA and WMA

On the third site revisit, cores were cut from each section. Six cores were obtained from each section. FIGURE 20 illustrates the aggregate gradation of the HMA test strip cores. In general, the aggregate gradation of the HMA test strip cores were similar to one another and the JMF gradation. FIGURE 21 displays the aggregate gradations for the HMA full night of paving cores. As was observed from the truck samples, the gradation of the mix placed was substantially finer than the JMF. FIGURE 22 illustrates the aggregate gradation from the WMA test strip cores. Two of the cores exhibited gradations finer than the JMF, while three of the cores were coarser than the JMF. FIGURE 23 displays the gradations of the cores from the WMA full night of paving section and it can be seen that the gradations were substantially finer than the JMF.

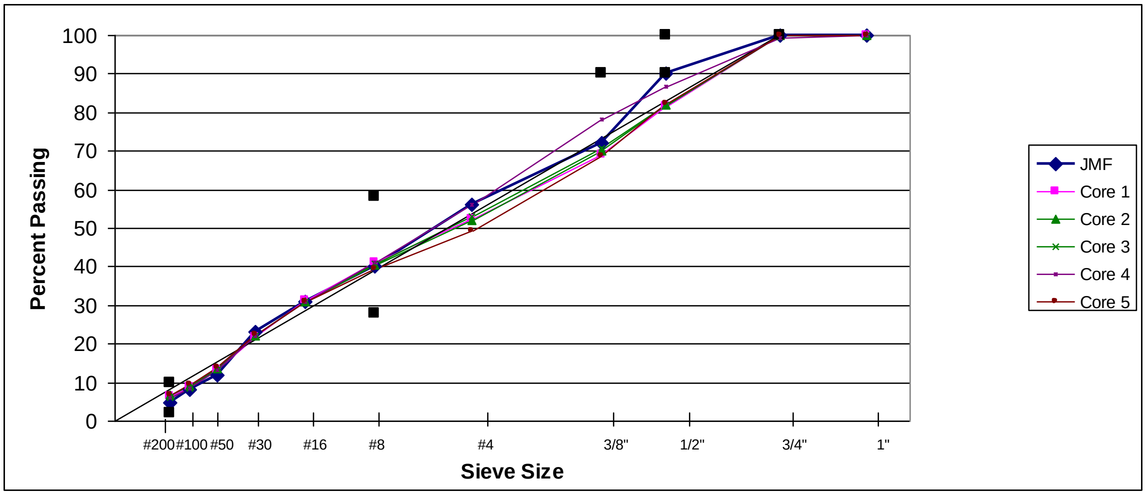

FIGURE 20 Aggregate Gradation of Cores from the HMA Test Strip

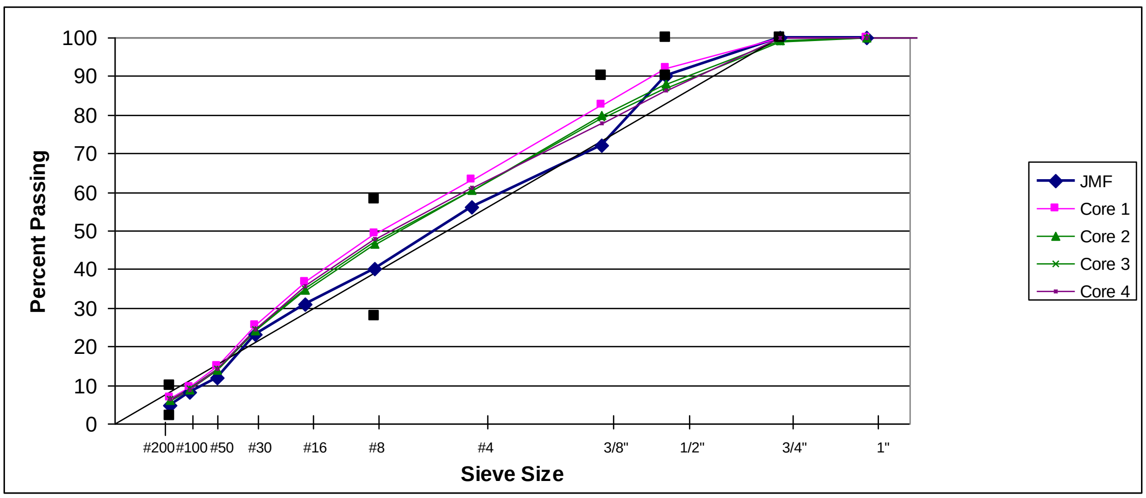

FIGURE 21 Aggregate Gradation of Cores from the HMA Full Section

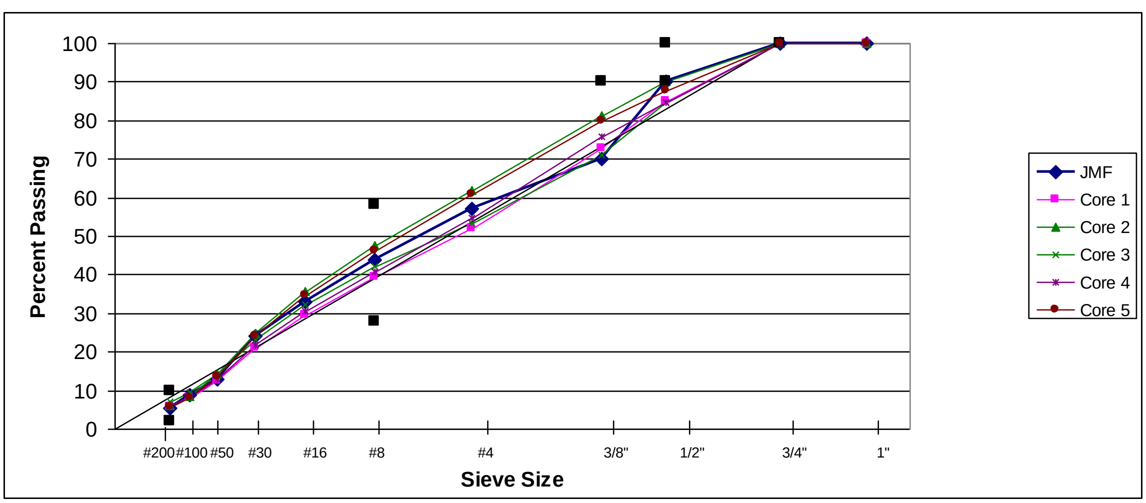

FIGURE 22 Aggregate Gradation of Cores from the WMA Test Section

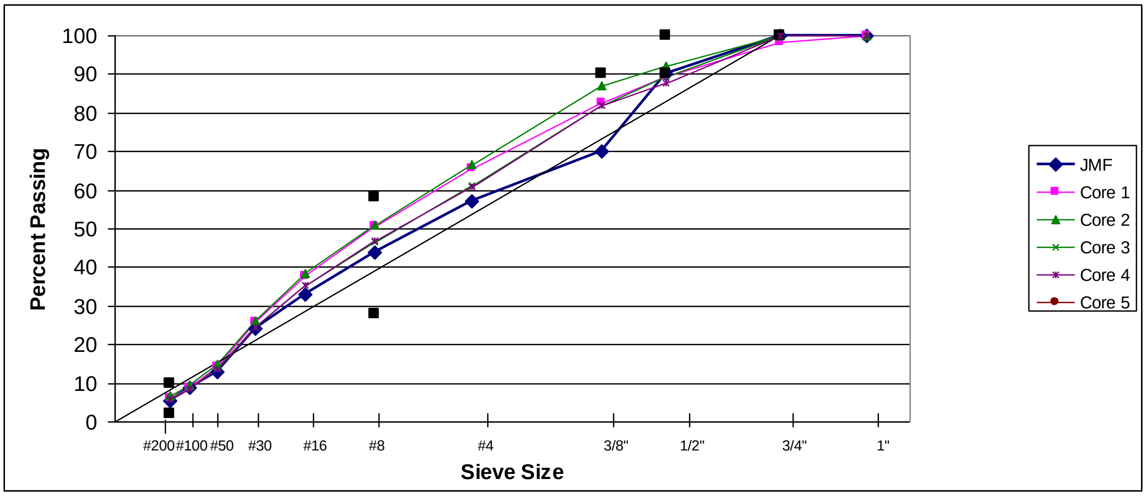

FIGURE 23 Aggregate Gradation of Cores from the WMA Full Section

TABLE 21 lists the asphalt contents of each core. FIGURE 24 illustrates the results of TABLE 21 with the whiskers representing plus and minus one standard deviation. The average asphalt content of the HMA full night of paving section was the highest, which differed from the truck samples obtained at the time of construction. The HMA test strip section had the lowest average asphalt content. The majority of the cores from that section had asphalt contents less than 5% with the exception of one core that had an asphalt content of 6.59%. The average asphalt content of the WMA sections were similar along with the variability associated with the asphalt content of those cores.

TABLE 21 Asphalt Content of Cores

Section	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6	Average
HMA Test Section	6.59	4.83	4.8	4.52	4.63	4.85	5.04
WMA Test Section	4.62	5.07	6.31	5.26	5.36	5.06	5.28
WMA Full Section	4.69	4.85	6.24	5.09	5.02	5.85	5.29
HMA Full Section	5.29	5.65	5.62	5.32	5.54	5.56	5.50

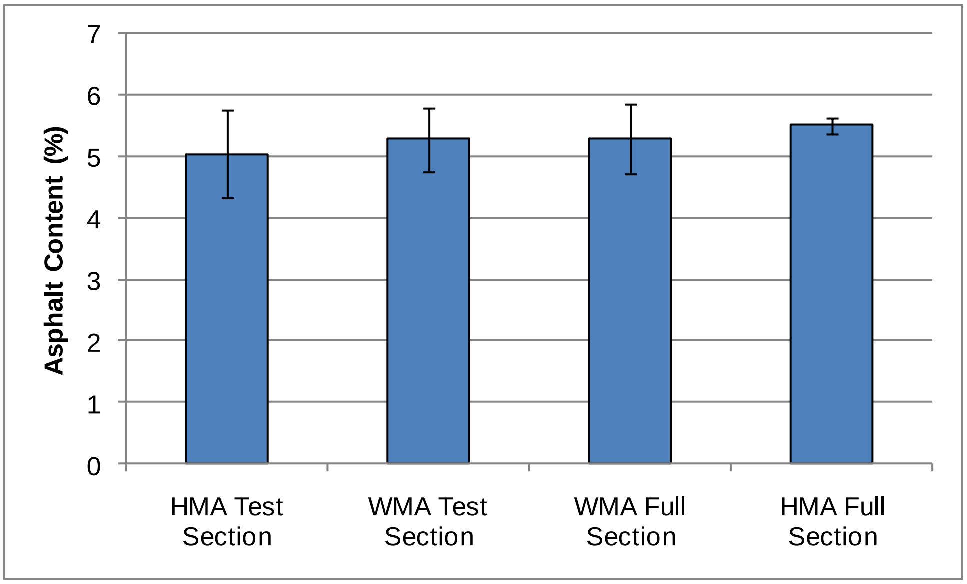

FIGURE 24 Average Asphalt Content of Cores

The air void content of the field cores was determined. FIGURE 25 illustrates the average and standard deviation of the field core air voids. As was seen at the time of construction, the WMA test strip exhibited the lowest air void content.

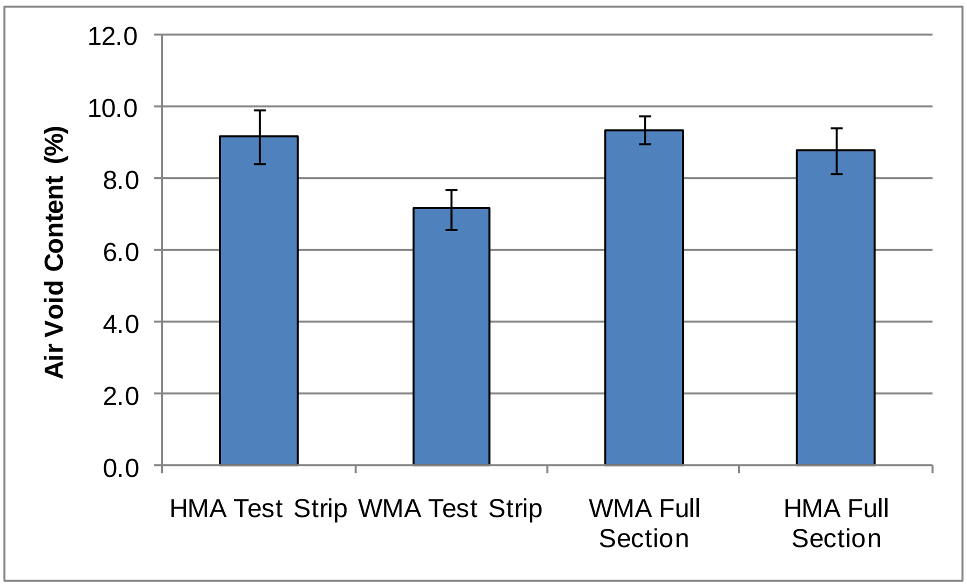

FIGURE 25 Air Void Content of Field Cores

The indirect tensile strength of the field cores was determined for three cores from each mix. The average indirect tensile strength (columns) and indirect tensile strength standard deviation (whiskers) are shown in FIGURE 26. The lowest average indirect tensile strength was the HMA test strip night, while the highest was the HMA full night of paving. These differences may partially be attributed to the asphalt content differences. Both WMA sections had indirect tensile strengths greater than the HMA test strip mix. Despite the WMA not containing RAS, the indirect tensile strengths of the WMA appear to be approaching that of the HMA full night of paving.

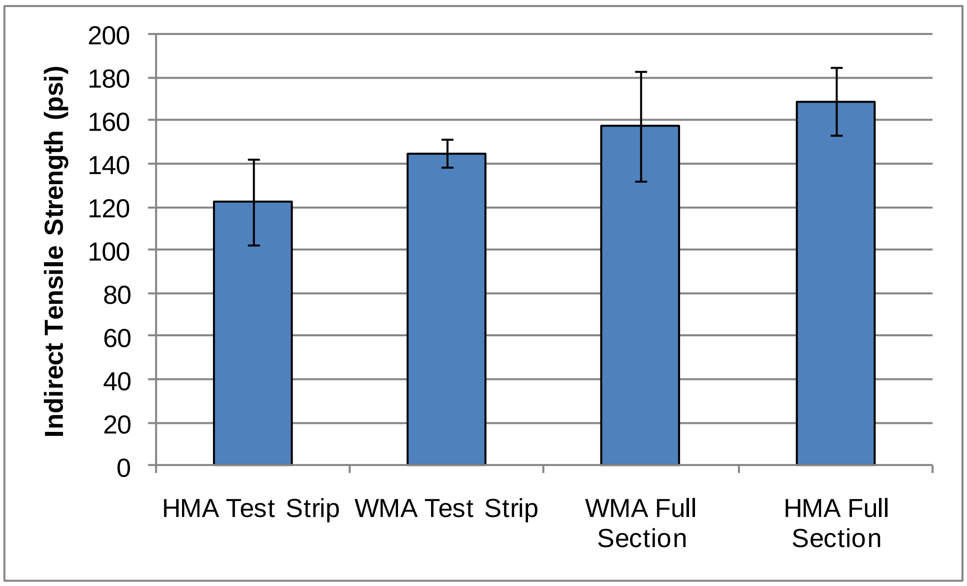

FIGURE 26 Indirect Tensile Strength of Field Cores

Bond strength testing was conducted on three field cores obtained from each pavement section to determine if the use of WMA would negatively affect the bond. TABLE 22 lists the average bond strength for the four sections constructed. FIGURE 27 illustrates the bond strength results. The whiskers represent plus and minus on standard deviation. The two WMA sections and the HMA test section all exhibited bond strength values greater than 100 psi. The two WMA sections performed similar to the first night of paving which was HMA. The last night of paving, which was HMA, exhibited lower bond strength values than the other three nights of paving. This may be an indication that that the bond between the HMA overlay and the underlying layer is not as strong as the other sections. Based on the bond strength test, the use of WMA did not negatively affect the bond.

TABLE 22 Bond Strength Results

Day	Average Bond Strength (psi)	Standard Deviation Bond Strength (psi)
Day 1 (HMA)	149.9	15.7
Day 2 (WMA)	156.4	14.6
Day 3 (WMA)	140.8	17.1
Day 4 (HMA)	84.8	45.2

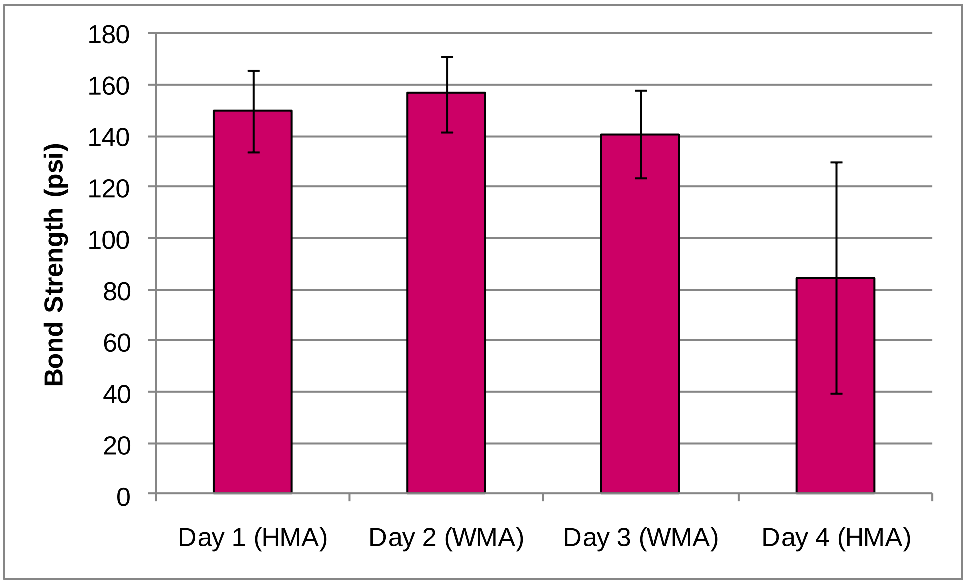

FIGURE 27 Bond Strength of Field Cores

Summary of Site Revisits

Three site revisits were conducted. Evaluations of the pavement in the daylight revealed that there were mix segregation issues throughout the pavement regardless of mix type. Asphalt contents and aggregate gradations of the field cores were determined. The asphalt contents from the field cores were closer to one another than the asphalt contents obtained from the field samples at the time of construction. The aggregate gradations however still varied similar to the samples collected at the time of construction. The indirect tensile strengths of the WMA after one year were close to approaching the indirect tensile strengths of the HMA, indicating that some "curing" had occurred. The bond strength testing of the field cores suggested that the WMA did not negatively affect the bond between pavement lifts.

CONCLUSIONS

ALDOT hosted a WMA demonstration using the WMA additive Evotherm^{TM} DAT. Two mixes were placed as part of this demonstration, a HMA containing 5% RAS and 10% RAS and a WMA containing 15% RAP.

During the production of the WMA at the Dunn asphalt plant in Tarrant City, Alabama, reduced odor and smoke were observed compared to the production of the HMA. Plant-produced mix was sampled and compacted in the NCAT mobile laboratory and then was transported to the main NCAT laboratory for testing. The laboratory testing evaluation of the plant-produced mix revealed the following:

- The asphalt content of the WMA test strip mix was substantially higher than the other mixes. The WMA asphalt content of the mix produced on the full night of paving was similar to the HMA test strip asphalt content, but greater than the HMA full night of paving asphalt content.
- The recovered asphalt binder of the WMA was more consistent than that of the HMA, which may be a result of the RAS not being properly distributed throughout the mix.
- The aggregate gradation of the WMA test strip was finer than the gradation of the JMF and the WMA full night of paving. The HMA gradations were coarser than the WMA gradations.
- The conditioned and unconditioned indirect tensile strengths of the WMAs were significantly lower than the HMA tensile strengths. All mix samples, with the exception of one from the full night of WMA paving, yielded acceptable tensile strength ratios.
- The APA rut tests results indicated that the WMA was more susceptible to rutting than the HMA. However, the higher asphalt content of the WMA may have affected the APA rutting results.
- The Hamburg wheel tracking results indicated that the WMA may by more prone to stripping than the HMA; however, all mixes passed the Hamburg stripping inflection criterion of a minimum of 5,000 cycles.

- The Hamburg rutting indicated that the WMA was more prone to rutting than the HMA; especially the WMA test strip with the higher asphalt content. However, the WMA produced on the full night of paving had total rut depths that did not exceed the maximum allowable rut depth
- The dynamic modulus results showed that the HMA was stiffer than the WMA
- The flow number testing supported the rut susceptibility findings of the APA and Hamburg
- The creep compliance testing suggested that the WMA is more susceptible to load-induced damage
- The site revisits indicated that mix segregation was an issue for both the HMA and WMA despite the use of a material transfer device
- The indirect tensile strengths of the WMA field cores was approaching that of the HMA after only 1 year indicating that WMA undergoes a type of "curing"
- The bond between pavement lifts is not affected by the use of WMA

REFERENCES

1. G. C. Hurley, "Evaluation of New Technologies for Use in Warm Mix Asphalt", MS Thesis Auburn University (2006).
2. G. C. Hurley and B. D. Prowell, "Evaluation of Evotherm for use in Warm Mix Asphalt", NCAT Report 06-02 (2006).
3. G. C. Hurley and B. D. Prowell, "Evaluation of Sasobit for use in Warm Mix Asphalt", NCAT Report 05-06 (2005).
4. G. C. Hurley and B. D. Prowell, "Evaluation of Aspha-Min Zeolite for use in Warm Mix Asphalt", NCAT Report 05-04 (2005).
5. Hurley, G., Prowell, B., and A. Kvasnak. *Ohio Field Trial of Warm Mix Asphalt Technologies: Construction Summary*. NCAT Report 09-04. Auburn, Al, 2009.
6. Hurley, G., Prowell, B., and A. Kvasnak. *Michigan Field Trial of Warm Mix Asphalt Technologies: Construction Summary*. (in review), Auburn, Al, 2009.
7. Hurley, G., Prowell, B., and A. Kvasnak. *Wisconsin Field Trial of Warm Mix Asphalt Technologies: Construction Summary*. (in review), Auburn, Al, 2009.

8. Hurley, G., Prowell, B., and A. Kvasnak. *Missouri Field Trial of Warm Mix Asphalt Technologies: Construction Summary*. (in review), Auburn, Al, 2009.
9. --, Moisture Sensitivity of Asphalt Pavements A National Seminar, Transportation Research Board, 2003.
10. Zhang, Z., R. Roque, B. Birgisson, and B. Sangpetngam. "Identification and Verification of a Suitable Crack Growth Law." *Journal of the Association of Asphalt Paving Technologists 70*, 2001: 206-241.
11. Zhang, Z., Roque R., and B. Birgisson. "Evaluation of Laboratory-Measured Crack Growth Rate for Asphalt Mixtures." *Transportation Research Record 1767*, 2001: 67-75.

APPENDIX A: FWD DATA FROM ALDOT

Table 23. FWD Test Results

			Temperature	FWD Force	FWD Deflections						
#	Direction	Marker	T(°F)	P (lb)	D0 (mils)	D1 (mils)	D2 (mils)	D3 (mils)	D4 (mils)	D5 (mils)	D6 (mils)
1	N	14.00	88	9005	12.7	8.8	5.0	2.8	1.6	1.1	0.8
2	N	14.10	88	9115	11.5	8.5	5.4	3.1	2.0	1.4	0.8
3	N	14.20	88	9137	16.4	10.5	5.6	2.7	0.9	0.6	0.2
4	N	14.40	88	9148	13.6	9.2	5.3	2.6	1.3	0.6	0.2
5	N	14.60	88	9038	15.7	10.6	5.9	3.4	2.1	1.3	1.0
6	N	14.81	88	9025	12.5	7.0	3.4	1.6	0.9	0.7	0.6
7	N	15.00	88	8882	16.7	11.7	6.1	3.1	1.1	0.4	0.0
8	N	15.20	88	8992	14.4	9.4	4.8	2.4	1.1	0.6	0.2
9	N	15.41	88	8726	28.9	22.0	14.1	8.2	4.1	2.5	1.6
10	N	15.60	88	8997	9.9	7.4	4.8	3.1	1.7	1.4	0.2
11	N	15.80	88	9014	14.7	9.8	5.2	2.6	1.2	0.9	0.8
12	N	16.01	88	9088	11.2	7.8	4.5	2.6	1.7	1.2	0.9
13	N	16.20	92	9001	13.9	9.6	5.7	2.8	1.3	0.5	0.2
14	N	16.40	92	9025	10.4	6.6	2.9	1.3	0.6	0.4	0.2
15	N	16.80	92	8989	20.1	11.1	4.3	1.6	0.5	0.4	0.4
16	N	17.00	92	8894	19.9	10.7	4.8	2.3	1.1	0.7	0.4
17	N	17.20	92	8976	13.0	8.4	4.0	2.0	1.3	0.8	0.7
18	N	17.40	92	8956	17.9	11.9	7.0	4.0	2.2	1.3	1.0
19	N	17.61	92	9014	13.7	8.9	4.3	1.9	0.8	0.6	0.4
20	N	17.80	92	8948	11.2	7.1	3.2	1.0	0.6	0.4	0.3
21	N	18.00	92	9017	13.3	7.9	4.2	2.3	1.5	1.1	0.9
22	N	18.21	92	8923	15.8	9.2	4.4	1.9	0.8	0.5	0.4
23	N	18.40	92	9055	9.1	4.9	2.2	1.0	0.6	0.3	0.3
24	N	18.60	92	8956	11.1	6.9	3.3	1.6	0.7	0.5	0.3
25	N	18.81	92	8964	10.9	7.4	4.4	2.6	1.6	1.0	0.4
26	N	19.00	92	8997	12.9	8.8	5.5	3.2	1.7	1.1	0.5
27	N	19.20	92	8669	18.9	12.5	6.8	3.1	1.4	0.6	0.2
28	N	19.41	92	8874	12.0	8.6	5.2	3.1	1.9	1.2	1.0
29	N	19.50	92	8902	14.2	8.9	4.8	2.4	1.3	0.7	0.4
		Average		8977	14.4	9.4	5.1	2.6	1.4	0.9	0.5

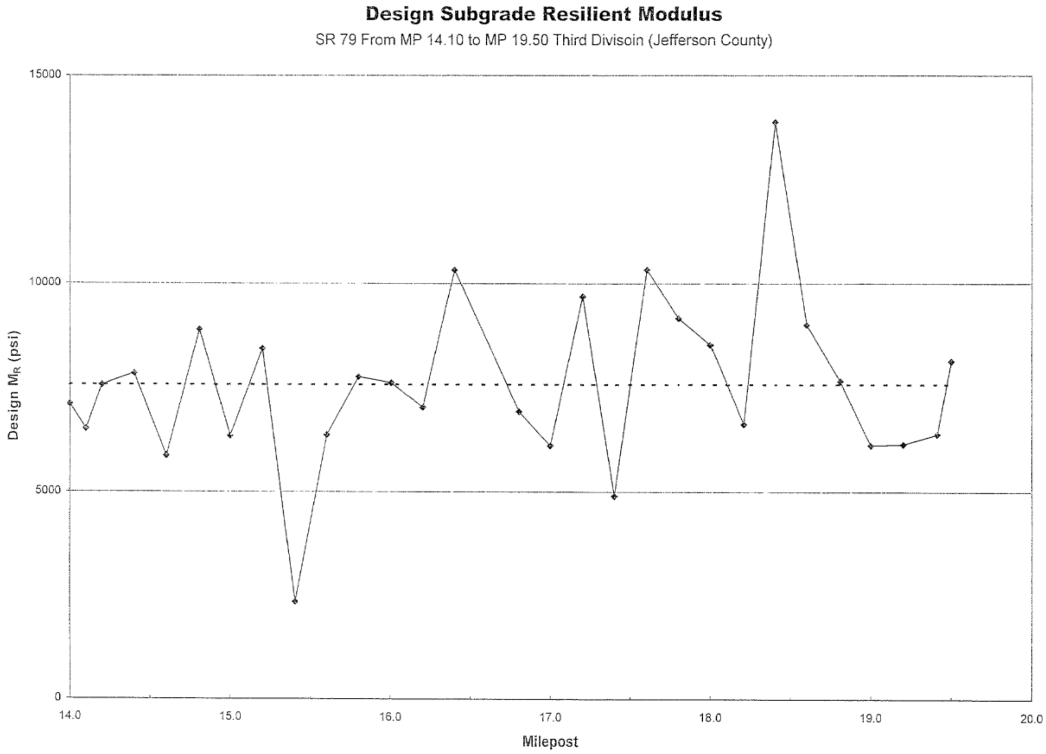

Figure 28. ALDOT Back-calculated Subgrade Moduli

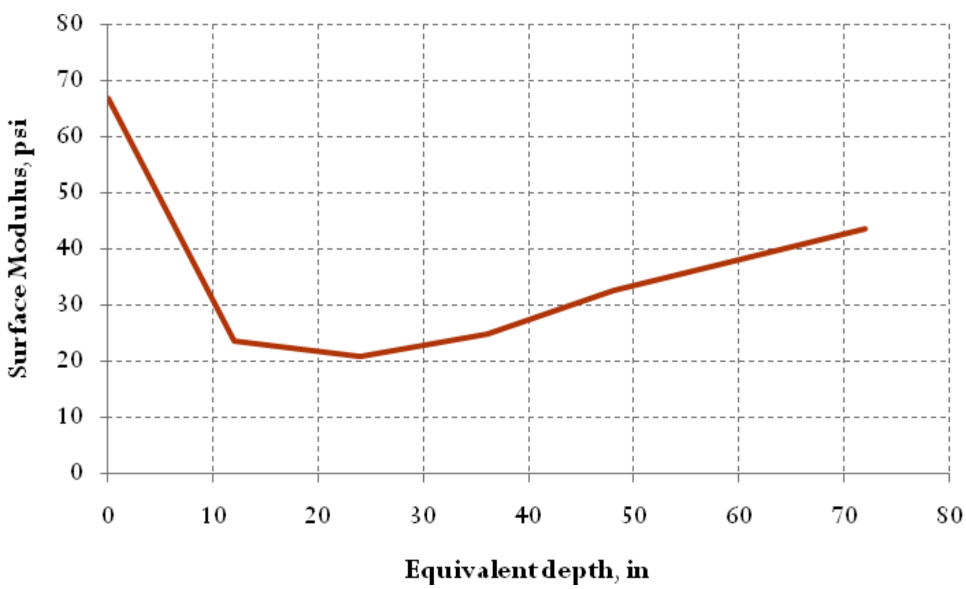

Figure 29. Surface Modulus Plot – Milepoint 14