Performance of Foamed Warm Mix Asphalt in Virginia Over Four to Six Years

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16. Abstract:

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Six mixtures produced using plant foaming technologies and placed between 2008 and 2010 were identified and subjected to field coring and laboratory testing to provide insight as to the performance of foamed WMA mixtures. All coring was performed in 2014, which resulted in pavement ages ranging from 4 to 6 years. Three comparable hot mix asphalt (HMA) mixtures were cored and evaluated to provide average values for comparison. All cores were tested to determine air-void contents and permeability and were subjected to dynamic modulus, repeated load permanent deformation, and Texas overlay testing. In addition, binder was extracted and recovered for performance grading.

Test results found similar properties for the WMA and HMA mixtures evaluated. One WMA mixture exhibited high dynamic modulus and binder stiffness, but overlay testing did not indicate any tendency toward premature cracking. All binders were found to have aged between two and three performance grades above that which was specified at construction. WMA binders and one HMA binder aged two grades higher, and the remaining two HMA binders aged three grades, indicating a likely influence on aging from the reduced temperatures at which the early foamed mixtures were typically produced. Overall results indicated that foamed WMA should be expected to perform similarly to HMA.

WMA has been fully adopted by VDOT as an alternative to HMA since 2008; however, at the time of implementation of foamed WMA, no studies had been conducted by VDOT to assess the performance of WMA relative to that of HMA. This study validated the assumption that the properties and performance of foamed WMA are similar to those of HMA.

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OVER FOUR TO SIX YEARS**

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ABSTRACT

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INTRODUCTION

Warm mix asphalt (WMA) is produced by incorporating additives into asphalt mixtures or by foaming asphalt binder to allow production and placement of the mixture at temperatures below the production temperatures of conventional hot mix asphalt (HMA). Benefits such as reduced plant emissions, improved compaction in the field, extension of the paving season into colder weather, longer haul distances, reduced opening time to traffic, and reduced energy consumption at the plant may be realized with different applications.

Research on the uses and benefits of WMA has been promising, and many states, including Virginia, now allow several WMA technologies. However, despite recent gains in the understanding of WMA and predictions of its performance, knowledge of the long-term impact of these technologies on mixture performance is very limited. NCHRP Project 09-49, Performance of WMA Technologies: Phase I—Moisture Susceptibility (Martin et al., 2014), found no evidence from in-service pavements to indicate that WMA is more susceptible to moisture damage than HMA, although laboratory test results can indicate potential issues. NCHRP Project 09-49A, Performance of WMA Technologies: Phase II—Long Term Performance (Washington State University et al., 2017), was recently completed and found that pavements containing various WMA technologies exhibited long-term performance comparable with that of the companion HMA pavement sharing similar pavement structure, climate, and traffic conditions. It benefits the Virginia Department of Transportation (VDOT) to be proactive in evaluating the lifetime performance of in-state WMA installations to verify their performance compared to that of conventional HMA, as the use of some form of WMA has become nearly ubiquitous in the state, with more than 75% of asphalt mixture production reported in 2014 being WMA (Hansen and Copeland, 2015).

In the past several years, more research has been initiated to investigate the performance of WMA. Although the majority of these studies were laboratory studies, the purpose of a select few was to evaluate the performance of WMA in the field. Most of the studies were limited to initial performance, as WMA has not been in use long enough for true lifetime performance to be evaluated.

VDOT's initial WMA trials were constructed in 2006 and documented in several reports (Diefenderfer and Hearon, 2008, 2010; Diefenderfer et al., 2007). These trials assessed the Sasobit additive and Evotherm DAT technology and determined that WMA should perform similarly to HMA, based on initial construction and early performance data. After the trials, VDOT developed a special provision for the use of WMA and in 2009 incorporated the use of

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approved WMA technologies in their specifications. At the time the trials were constructed, plant foamed WMA had not been introduced; however, by 2010, plant foaming was increasingly used to produce WMA because of its cost-competitiveness with HMA.

Wielinski et al. (2009) documented the production and construction of two foamed WMA demonstration test sections in 2008 and considered initial performance up to approximately 5 months of service. The projects were deemed a success, as design and construction criteria were met, constructability was good, and the initial performance was excellent.

Texas began investigating WMA use around the same time as Virginia, and initial work resulted in the statewide implementation of Special Provision 341-020 in January 2009, which allowed WMA to be used at the contractor's option (Button et al., 2007; Estakhri et al., 2010). A subsequent study monitored more than 10 WMA projects for approximately 3 years while also validating procedures for determining mixture volumetrics and performance properties (Estakhri, 2012). The findings of the study indicated that the performance of WMA was comparable to that of HMA and supported the Texas laboratory curing time and temperature selection procedures.

Florida constructed its first WMA pavement in 2006 and began to allow WMA use in 2010 (Sholar et al., 2009). A study evaluating six of the WMA pavements at ages ranging from 2 to 6 years found that the field performance of the WMA was comparable to that of HMA (Choubane et al., 2014). Foaming was used in the production of four of these pavements, the oldest of which was 5 years of age at the time of evaluation. The study noted that no instances of significant deterioration or poor performance were identified in the six WMA pavements, nor were any practical differences seen between the various WMA technologies used in the projects.

The National Center for Asphalt Technology (NCAT) reported results from WMA trials conducted in several states (Hurley et al., 2009a,b; 2010a,b; Kvasnak et al., 2010); this work focused primarily on construction, although performance was detailed for 2 years for the trial located in Missouri (Hurley et al., 2010a). Additional work was conducted in NCHRP Project 09-47A, Engineering Properties, Emissions, and Field Performance of Warm Mix Asphalt Technologies, as one of the first large-scale national studies of WMA performance (West et al., 2014). The study investigated 14 projects, including 12 WMA technologies. Projects included new construction and in-service projects that were 3 to 5 years old. The in-service projects included only two foamed WMA mixtures. In all projects, the in-service performance of WMA and HMA was virtually identical. Little to no rutting was observed, no evidence of moisture damage was found, and very little indication of transverse or longitudinal cracking was seen.

Additional assessments of performance were conducted at the NCAT test track from 2009 to 2011 when several test sections that included WMA technology were placed. One mixture in the experiment with a high recycled asphalt pavement (RAP) content was produced using the Astec Double-Barrel Green System foaming technology. In addition, a WMA experiment was conducted; this experiment used WMA mixtures for all pavement layers produced using two technologies: the Astec Double-Barrel Green System and the Evotherm DAT chemical additive. Very little difference was seen in the field for these mixtures as compared to control mixtures. Rutting in WMA experiment sections was minimal but was slightly increased in the two WMA sections over that in the control section.

The general conclusions from each of these studies supported the concept that WMA performance should be expected to be similar to that of HMA. However, there is a distinct lack of long-term field performance data available to validate expectations for foamed WMA mixtures; results of performance assessments after 5 years in service were found for only six foamed WMA field sites nationwide.

PURPOSE AND SCOPE

The purpose of this study was to document the material properties of early foamed WMA sections constructed in Virginia and investigate the long-term performance of the sections to validate the expectation of equivalent performance. Six mixtures produced using plant foaming technologies and placed between 2008 and 2010 were identified and subjected to field coring and laboratory testing to provide insight as to the performance of foamed WMA mixtures. All coring was performed in 2014, which resulted in pavement ages ranging from 4 to 6 years.

Information collected included the documentation of locations constructed with plant foaming technologies. Locations were visited and general condition was observed to assess performance. Cores were collected to verify mixture properties such as in-place density and binder grade and content. These cores were also tested to determine modulus values and evaluate resistance to rutting.

No "control" HMA pavements were constructed for direct comparison to the foamed WMA pavements. For the purpose of comparison, three HMA mixtures with known properties and performance were cored at 5 years of age to provide average age reference points for material properties.

METHODS

Site Evaluation

Locations

This study evaluated mixtures constructed from 2008 to 2010 using plant foaming WMA technologies; no companion HMA mixtures were constructed. These mixtures encompass examples of some of the earliest experiences with plant foaming techniques for the production of WMA. Sections were cored and evaluated in 2014 for the current work.

As no companion HMA pavement sections were constructed to serve as comparison mixtures, three surrogate mixtures were selected to use as comparison mixtures. These mixtures were constructed in 2006 and were cored at an age of 5 years in 2011.

Location information and the designation for each foamed WMA mixture are provided in Table 1. Table 2 shows the information for each HMA mixture. Mixture design information is shown in Table 3 for all mixtures. Figure 1 shows the design gradation of each mixture.

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Table 1. Plant Foamed WMA Mixtures

Mixture	Year	VDOT District	County	Route	Mileposts	Length	Mixture Type	Contractor
A	2008	Lynchburg	Pittsylvania	US 29 SBL	0.97-4.45	3.48 mi	SM-9.5D	Piedmont Asphalt
B	2009	Staunton	Frederick	US 522 NBL	0.0-1.52	1.52 mi	SM-12.5D	W-L Construction
C	2010	Staunton	Frederick	SR 7 WBL	0.0-3.64	3.64 mi	SM-12.5D	W-L Construction
D	2010	Staunton	Rockingham	US 33 WBL	30.01-37.71	7.70 mi	SM-12.5A	Adams Construction
E	2010	Lynchburg	Campbell	SR 128	0.16-0.69	0.53 mi	SM-9.5D	Templeton Paving
F	2010	Lynchburg	Pittsylvania	US 29 NBL	4.42-7.72	3.30 mi	SM-9.5D	Piedmont Asphalt

WMA = warm mix asphalt; VDOT = Virginia Department of Transportation; SBL = southbound lane; NBL = northbound lane.

Table 2. HMA Mixtures Produced in 2006

Mixture	VDOT District	County	Route	Mixture Type	Contractor
HMA 1	Culpeper	Rappahannock	SR 211	SM-9.5A	Superior Paving Co.
HMA 2	Staunton	Highland	US 220	SM-12.5A	B&S Construction Inc.
HMA 3	Hampton Roads	York	SR 143	SM-9.5D	Branscome, Inc.

HMA = hot mix asphalt; VDOT = Virginia Department of Transportation.

Table 3. Mix Design Information for Plant Foamed WMA and HMA Mixtures

Mix Design	Mixture								
	A	B	C	D	E	F	HMA 1	HMA 2	HMA 3
Sieve Size, % Passing									
19 mm	100	100	100	100	100	100	100	100	100
12.5 mm	100	96	96	96	100	100	100	96	100
9.5 mm	94	84	84	86	96	93.5	92	86	93
4.75 mm	64	-	-	-	64	58	60	-	61
2.36 mm	46	40	38	35	-	43	43	34	44
0.075 mm	5.8	6.5	6.5	5.6	5.8	5.2	5.7	6	5
Volumetric Properties									
VTM	3.5	4.0	-	3.2	4.0	3.8	-	-	-
VFA	78	-	-	-	-	-	-	-	-
Asphalt content	5.8	5.5	5.5	5.7	5.8	5.6	5.5	5.2	5.7
VMA	15.9	-	-	-	-	-	-	-	-
FA ratio	1.0	-	-	-	-	-	-	-	-
G_{mm}	2.436	2.613	-	2.423	2.420	2.529	-	-	-
Other Properties									
RAP content	25	25	25	15	26	26	12	10	20
Binder grade	PG 64-22	PG 64-22	PG 64-22	PG 64-22	PG 64-22	PG 64-22	PG 64-22	PG 64-22	PG 70-22

WMA = warm mix asphalt; HMA hot mix asphalt; - = data not available; VTM = voids in total mix; VFA = voids filled with asphalt; VMA = voids in mineral aggregate; FA ratio = fines to aggregate ratio; G_{mm} = maximum mixture theoretical specific gravity; RAP = recycled asphalt pavement.
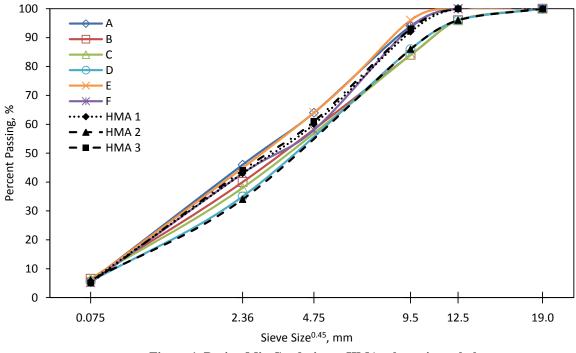
Figure 1. Design Mix Gradations. HMA = hot mix asphalt.

Coring

Ten cores 6 in (150 mm) in diameter were taken from each foamed WMA pavement section for evaluation. Six cores 6 in (150 mm) in diameter were taken from each HMA pavement section for evaluation. Core locations were randomized along the length and width of the pavement section.

VDOT Pavement Management System (PMS) Data

Distress data for the foamed WMA sites were extracted from VDOT's PMS. VDOT's Maintenance Division acquires and maintains the results of an annual condition survey of all interstates, all primaries, and approximately 20% of secondary pavements. The survey collects and summarizes detailed distress data for each 0.1 mile of right-lane or principal direction pavement surface. Condition is reported on a scale from 0 to 100, completely failed to new or like new, respectively. The overall section rating, the composite condition index (CCI), is the lower of two ratings that summarize the load-related and non–load related distresses for a pavement.

Laboratory Evaluation

Core Air Voids

Air-void contents were determined in accordance with AASHTO T 269, Percent Air Voids in Compacted Dense and Open Asphalt Mixtures (American Association of State Highway and Transportation Officials [AASHTO], 2014).

Permeability

Permeability testing was performed on cores in accordance with Virginia Test Method 120, Method of Test for Measurement of Permeability of Bituminous Paving Mixtures Using a Flexible Wall Permeameter (VDOT, 2014).

Dynamic Modulus Test

Dynamic modulus tests were performed using an Asphalt Mixture Performance Tester (AMPT) with a 25 to 100 kN loading capacity in accordance with AASHTO T 342, Standard Method of Test for Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures (AASHTO, 2014). Tests were performed on 38-mm-diameter by 110-mm-tall specimens cored horizontally from field cores (Bowers et al., 2015; Diefenderfer et al., 2015). Three testing temperatures (4.4°C, 21.1°C, and 37.8°C) and six testing frequencies ranging from 0.1 to 25 Hz were used. Tests were conducted starting from the coldest temperatures to the warmest temperatures. In addition, at each test temperature, the tests were performed starting from the highest to the lowest frequency. Load levels were selected in such a way that at each temperature-frequency combination, the applied strain was in the range of 75 to 125 microstrain. All tests were conducted in the uniaxial mode without confinement. Stress versus strain values were captured continuously and used to calculate dynamic modulus. Dynamic modulus was computed automatically using IPC |E*| software. Results at each temperature-frequency combination for each mixture type are reported for three replicate specimens.

Repeated Load Permanent Deformation Test

The repeated load permanent deformation (RLPD) test is used to evaluate the rutting resistance of asphalt mixtures. An AMPT with a 25 to 100 kN loading capacity was used to conduct the tests. Testing was performed on 38-mm-diameter by 110-mm-tall specimens cored horizontally from field cores. All RLPD testing was conducted on specimens previously tested for dynamic modulus. Tests were conducted at 54°C based on LTPPBind software that represents the 50% reliability maximum high pavement temperature at locations in central Virginia. A repeated haversine axial compressive load pulse of 0.1 s every 1.0 s was applied to the specimens. The tests were performed in the confined mode using a confining stress of 10 psi (68.9 kPa) and a deviator stress of 70 psi (483 kPa). The tests were continued for 10,000 cycles or a permanent strain of 10%, whichever came first. During the test, permanent strain (ε_p) versus the number of loading cycles was recorded automatically, and the results were used to estimate the flow number. The flow number was determined numerically as the cycle number at which the strain rate is at a minimum based on the Francken model.

In addition to calculations of the flow number, the slope and intercepts of the secondary portion of the cycle vs. permanent strain curve were estimated in log-log space by applying a power-law function:

$$\varepsilon_p = aN^b \qquad\qquad\qquad \text{[Eq. 1]}$$

where

ε_p = permanent strain
a = intercept in log-log space
N = cycle number
b = slope in log-log space.

The slope of this line is indicative of the plastic strain rate of the mixture. A higher slope relative to other mixtures indicates a mixture that is likely more susceptible to rutting. The intercept of the line is indicative of the plastic strain at cycle N = 1 (Khosravifar et al., 2015; Von Quintus et al., 2012).

No preconditioning load was used during testing; however, Gibson and Li (2013) reported that initial RLPD testing performed in the laboratory testing for NCHRP Project 09-30A used a preconditioning stage consisting of the application of 100 cycles of 10% of the deviator stress. Gibson and Li performed analyses to determine a method wherein virtual preconditioning could be applied to test results obtained without preconditioning to approximate the change in response predicated by the actual preconditioning cycles. This virtual preconditioning consisted of the removal of the first 51 cycles of data and the subsequent re-zeroing of the 52nd cycle to approximate the response curve after preconditioning cycles have been performed. All analyses in this study except the determination of flow number were conducted after virtual preconditioning was applied.

Overlay Test

The Texas overlay test was performed to assess the susceptibility of each mixture to cracking. Testing of field cores having a 150-mm diameter and varying thicknesses was performed generally in accordance with TX-248-F, Test Procedure for Overlay Test (Texas Department of Transportation, 2009), using a universal testing machine with a loading capacity of 25 to100 kN. Testing was performed at a temperature of 25 ± 0.5°C. Loading was applied for a total of 1,200 cycles or until a reduction of 93% or more of the maximum load was reached.

Binder Extraction and Recovery

Extraction of binder from cores was performed in accordance with AASHTO T 164, Quantitative Extraction of Asphalt Binder from Hot Mix Asphalt (HMA), Method A (AASHTO, 2014), using n-propyl bromide as the solvent. Binder was recovered from the solvent using the Rotavap recovery procedure specified in AASHTO T 319, Quantitative Extraction and Recovery of Asphalt Binder from Asphalt Mixtures (AASHTO, 2014).

Binder Testing

Binder grading was performed in accordance with AASHTO M 320, Performance-Graded Asphalt Binder (AASHTO, 2014).

RESULTS AND DISCUSSION

Visual Assessment

Visual assessment of the foamed WMA mixtures indicated no issues with performance. Mixture B had been overlaid at an age of 6 years during the summer prior to coring and thus the condition could not be seen. HMA mixtures were generally performing as expected.

PMS Data

Distress data were extracted from VDOT's PMS for sections containing the foamed WMA mixtures and are summarized in Table 4. Section B is shown to have had a CCI rating of 59 prior to placement of the overlay in 2015. Sections A and F had the next lowest CCI ratings: 69 and 72 at ages 8 and 6 years, respectively. All other sections had CCI ratings over 80. Ride quality for all sections was acceptable, with four of the six sections having an International Roughness Index (IRI) below 95 and the remaining two sections having an IRI below 115.

Table 4. PMS Distress Data for Foamed WMA Sections

Data	Section					
	A	B[a]	C	D	E[b]	F
Year Paved	2008	2009	2010	2010	2010	2010
Year Evaluated	2016	2014	2016	2016	2015	2016
Segment Length, mi	2.99	1.56	1.81	7.65	0.28	3.03
Average IRI, Left Wheel Path	77	112	78	78	112	80
Average IRI, Right Wheel Path	91	111	81	85	116	74
Average IRI	*84*	*112*	*80*	*82*	*114*	*77*
Transverse Cracking Severity 1, avg. ft/mi	5176.2	0	16.6	442.0	146.4	871.3
Transverse Cracking Severity 2, avg. ft/mi	114.7	0	0	4.3	0	0
Longitudinal Cracking Severity 1, avg. ft/mi	254.4	0	0	83.7	3.6	11.9
Longitudinal Cracking Severity 2, avg. ft/mi	10.0	0	0	2.6	0	0
Longitudinal Joint Cracking Severity 1, avg. ft/mi	292.5	0	286.9	1637.4	0	271.2
Longitudinal Joint Cracking Severity 2, avg. ft/mi	0	0	0	59.6	0	0
Alligator Cracking Severity 1, avg. ft^2/mi	4516.5	37.3	59.1	1124.0	42.9	1070.3
Alligator Cracking Severity 2, avg. ft^2/mi	1326.0	33.4	1.7	189.4	0	14.2
Alligator Cracking Severity 3 , avg. ft^2/mi	0	0	0	0.3	0	0
Patching Area, Wheel Path, avg. ft^2/mi	25.4	0	3.32	0	0	0
Patching Area, Non–Wheel Path, avg. ft^2/mi	0.7	0	0	0	0	0
Delamination Area, avg. ft^2/mi	7.4	0	0	0	0	0
Bleeding Severity 1, avg. ft^2/mi	0	30.8	0	0	0	0
Bleeding Severity 2, avg. ft^2/mi	0	62.9	0	0	0	0
Average Rut Depth, in	0.10	0.29	0.16	0.12	0.17	0.08
Total Average Rutting per Mile, in/mi	1.03	2.97	1.71	1.22	1.79	0.87
Average Load Distress Rating	*69*	*59*	*94*	*87*	*94*	*75*
Average Non Load Distress Rating	*79*	*100*	*98*	*85*	*98*	*73*
Average Composite Condition Index	*69*	*59*	*93*	*82*	*94*	*72*

PMS = Virginia Department of Transportation Pavement Management System; IRI = International Roughness Index; avg. = average.
[a] Section B was overlaid in 2015 at an age of 6 years.
[b] Section E condition data were available only from 2015. These data were collected in the westbound travel lane; coring of Section E was performed in the eastbound passing lane.

Core Air Voids

Air voids were measured for all cores from all foamed WMA and control mixtures. Table 5 details the results. Figure 2 summarizes the average and standard deviation of air-void contents. Cores from Mixtures B and D had very low average void contents, at 2.97% and 1.94%, respectively, and the remaining mixtures had average void contents in the range of 5% to 9%.

Table 5. Road Core Air Voids, %

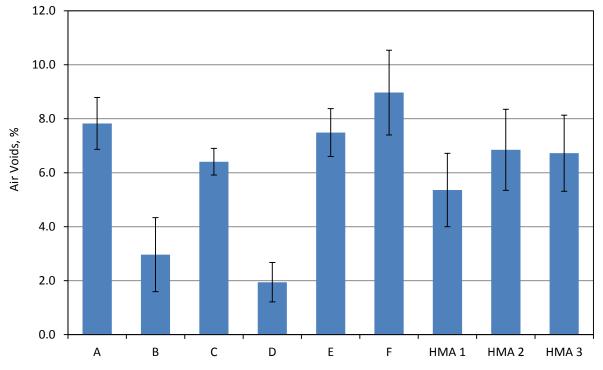| Specimen	Mixture								
	A	B	C	D	E	F	HMA 1	HMA 2	HMA 3
1	8.4	5.5	6.7	3.1	8.3	10.3	5.6	7.1	6.4
2	8.2	3.7	6.5	1.7	7.6	9.9	5.9	6.9	4.4
3	7.9	4.8	6.7	2.1	8.6	11.3	3.3	6.1	6.1
4	8.1	1.8	6.6	1.2	8.5	10.2	6.2	9.4	8.2
5	7.9	2.2	5.1	1.5	7.0	9.6	4.2	6.8	7.2
6	6.6	1.5	6.5	2.2	7.0	9.1	7.0	4.8	8.0
7	7.9	1.7	6.4	2.0	6.0	8.3			
8	8.2	2.9	6.8	1.1	6.3	6.4			
9	5.8	2.3	6.4	3.1	7.6	7.1			
10	9.2	3.4	6.3	1.5	8.0	7.6			
Average	7.83	2.97	6.41	1.94	7.49	8.97	5.36	6.85	6.72
Std. Deviation	0.96	1.37	0.49	0.73	0.88	1.57	1.36	1.50	1.41

HMA = hot mix asphalt.

Figure 2. Average and Standard Deviation of Road Core Air-Void Contents. HMA = hot mix asphalt.

Permeability results for all tested cores are shown in Table 6. Cores from mixtures HMA 1 and HMA 2 were not tested. A plot of the permeability results as shown in Figure 3 indicates that all results with the exception of those for Mixture D were below the specification requirement of 150 x 10^{-5} cm/s at predicted permeability at 7.5% air voids. A number of specimens are not shown on the plot in Figure 3 as the permeability values were zero. A summary of the relationships between permeability and void contents is shown in Table 7.

Table 6. Permeability of Road Cores (permeability x 10^{-5} cm/s)

Specimen	Mixture								
	A	B	C	D	E	F	HMA 1	HMA 2	HMA 3
1	17.34	1.10	6.82	8.68	1.30	3.80	N/A	N/A	0.00
2	12.21	0.00	8.28	27.90	3.03	6.24			0.00
3	4.29	1.39	2.44	6.53	1.49	5.83			0.00
4	3.03	0.00	4.23	1.62	1.52	1.57			6.30
5	14.30	0.00	5.68	1.47	2.80	3.93			0.00
6	1.21	0.00	4.31	7.31	2.00	1.60			0.89
7	8.90	0.00	4.33	4.24	0.57	0.51			
8	4.27	0.00	2.06	0.00	0.83	0.00			
9	0.43	0.00	6.46	19.13	0.80	1.64			
10	25.36	0.00	6.36	2.63	2.01	0.00			

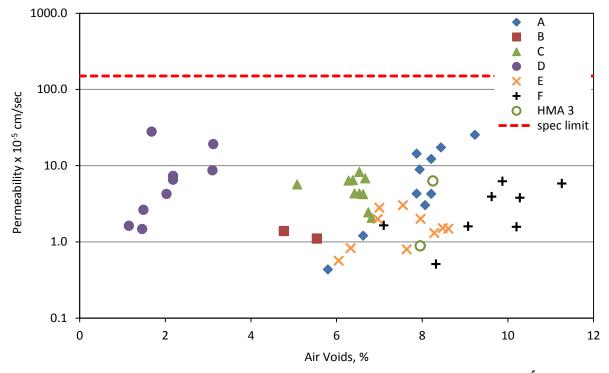HMA = hot mix asphalt; N/A = data not available.

Figure 3. Road Core Permeability Results. The specification requires a maximum of 150 x 10^{-5} cm/s permeability at 7.5% air voids. HMA = hot mix asphalt.

Table 7. Permeability of Road Cores (permeability x 10^{-5} cm/s)

Mixture	Exponential Trendline	Predicted Permeability x 10^{-5} cm/s at 7.5% Air Voids
A	$y = 0.0004e^{1.205x}$	3.365088
B	[a]	-
C	[b]	-
D	$y = 0.8672e^{0.9173x}$	843.2507
E	$y = 0.2599e^{0.2285x}$	1.442376
F	$y = 0.0439e^{0.4234x}$	1.050913
HMA 3	[a]	-

HMA = hot mix asphalt.
[a] Insufficient data points with permeability $\neq 0$ to fit trendline.
[b] Air voids do not span sufficient range to fit trendline appropriately.

Dynamic Modulus Test

Dynamic modulus testing was performed on small-scale specimens cored horizontally from the road cores. Table 8 provides details of the individual specimen air voids. Ideally, specimen sets should be ±0.5% of the target air-void content, which for this study was the average air void content of the cores. However, because of the inherent variability of cored specimens, this was not always possible. An effort was made to choose cores with air-void contents as similar as possible for use in preparing the dynamic modulus specimens; despite this, there was variability in air voids within some specimen sets that was greater than desired. Table 9 summarizes some of the mixture properties that influence the dynamic modulus.

Table 8. Dynamic Modulus and RLPD Specimen Void Contents

Specimen	Mixture								
	A	B	C	D	E	F	HMA 1	HMA 2	HMA 3
1	7.9	5.5	6.4	1.7	7.6	8.3	5.6	6.9	6.4
2	7.9	3.7	6.4	2.2	7.0	7.1	5.9	6.1	6.1
3	6.6	4.8	6.3	2.2	7.6	7.6	4.2	6.8	7.2
Average	7.45	4.67	6.36	2.01	7.40	7.66	5.22	6.60	6.57
Standard Deviation	0.72	0.93	0.07	0.29	0.35	0.62	0.91	0.43	0.57
All Cores Average	7.83	2.97	6.41	1.94	7.49	8.97	5.36	6.85	6.72

RLPD = repeated load permanent deformation. HMA = hot mix asphalt.

Table 9. Summary of Mixture Properties and Average Specimen Set Void Contents for Dynamic Modulus and RLPD Testing

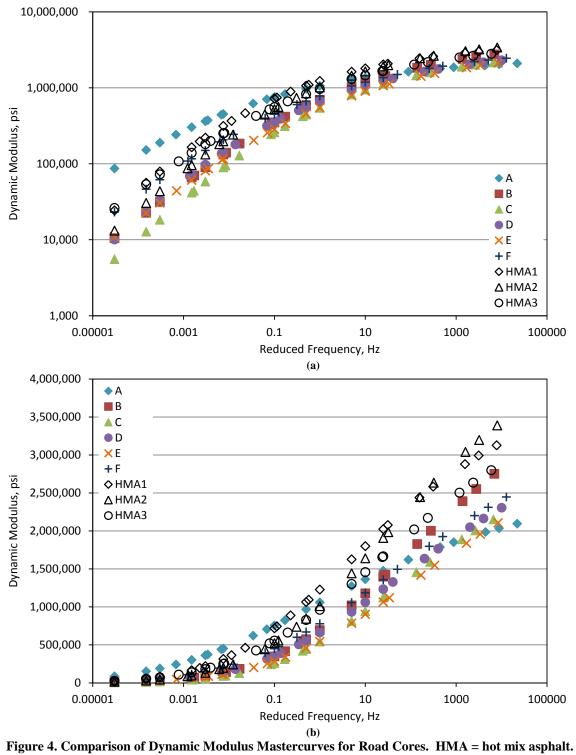| Mixture	Mixture Type	Binder Type	Asphalt Content, %	RAP Content, %	Specimen Set Average Voids, %
A	SM-9.5D	PG 64-22	5.8	25	7.45
B	SM-12.5D	PG 64-22	5.5	25	4.67
C	SM-12.5D	PG 64-22	5.5	25	6.36
D	SM-12.5A	PG 64-22	5.7	15	2.01
E	SM-9.5D	PG 64-22	5.8	26	7.40
F	SM-9.5D	PG 64-22	5.6	26	7.66
HMA 1	SM-9.5A	PG 64-22	5.5	12	5.22
HMA 2	SM-12.5A	PG 64-22	5.2	10	6.60
HMA 3	SM-9.5D	PG 70-22	5.7	20	6.57

RLPD = repeated load permanent deformation; RAP = recycled asphalt pavement.

Dynamic modulus mastercurves, created using a reference temperature of 70°F (21°C), are shown in Figure 4 for all mixtures. Figure 4(a) shows results on a log-log scale, which emphasizes the differences in modulus for mixtures below approximately 0.1 Hz, corresponding to slow loading frequencies, which are analogous to higher temperatures. Mixture A is shown to be the stiffest mixture at frequencies below 0.1 Hz, and Mixture C is the least stiff at frequencies below 0.1 Hz. Figure 4(b) presents results using a log-linear scale to emphasize differences in modulus at higher frequencies that are masked by a log-log scale. This indicates that HMA 1 and HMA 2 are stiffer at frequencies above 0.1 Hz and that Mixtures C and E are least stiff. Of interest, Mixture A, although being stiffer at lower frequencies, had a considerably lower slope than the other mixtures, and so its stiffness was less relative to the other mixtures with increasing frequency of load application. Practically speaking, this indicates a stiffer mixture at higher temperatures that is also less stiff at low temperatures. All other mixtures maintained similar rankings in terms of stiffness throughout the range of test frequencies.

Black space diagrams plot dynamic modulus versus phase angle and enable the assessment of how stiffness and elasticity are related in a material. In the case of asphalt materials, the black space plot shows a peak phase angle at intermediate stiffness that is due to the interaction of binder and aggregate. At higher temperatures, because of the viscous flow and low stiffness of the binder, the aggregate structure begins to dominate material response. At lower temperatures, the mixture volumetrics and binder stiffness control behavior.

From the black space graph shown in Figure 5, the interaction of stiffness and phase angle can be investigated. At low test temperatures, the mixtures respond in an elastic manner and have the highest modulus values, shown in the upper left portion of the curve. As the temperature increases, the binders soften and become more viscous, causing the modulus to decrease as the phase angle increases. At the highest test temperature, 37.8°C for this study, the aggregate structure can be seen to begin to dominate the response in many of the mixtures as the phase angle peaks and then begins to decrease.

In Figure 5, Mixture A has lower phase angles compared to the other mixtures and does not reach the peak phase angle seen in the other mixtures during testing. This indicates that the binder had likely not reached viscous flow in the course of the temperatures used during the test and may explain the higher stiffness seen at the lower reduced frequencies in Figure 4a. In comparison, Mixtures E and F show pronounced decreases in phase angle and modulus with increased temperature. The remaining mixtures were more consistent in behavior. Interestingly, Mixtures A, E, and F were SM-9.5 mixtures, which may indicate that the aggregate structures could be playing a role in the behavior; however, there is no particular gradation indication among the three mixtures to pinpoint specifically. Mixtures A and E were very close in design gradation on all sieves yet showed very different behavior. Mixture F indicated behavior similar to that of Mixture E but was coarser on the 4.75 mm sieve.
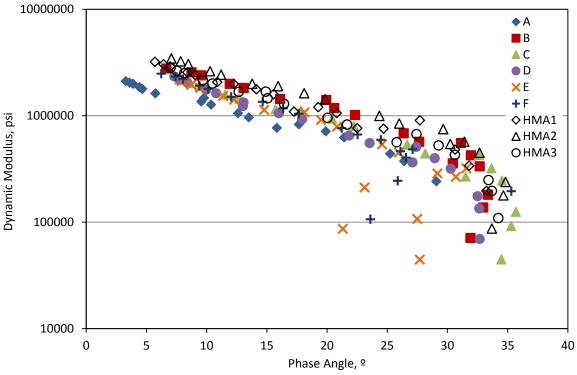
(a)

(b)

Figure 4. Comparison of Dynamic Modulus Mastercurves for Road Cores. HMA = hot mix asphalt.

Figure 5. Comparison of Black Space Plots for Road Cores. HMA = hot mix asphalt.

RLPD Test

The RLPD test was performed to assess the rutting potential of the mixtures. The test was performed in the confined mode, using a confining stress of 10 psi (68.9 kPa) and a deviator stress of 70 psi (483 kPa). Specimens were loaded until 10,000 cycles or 10% strain, whichever came first. No preconditioning load was used; however, Gibson and Li (2013) reported that initial RLPD testing performed in the laboratory testing in NCHRP Project 09-30A used a preconditioning stage consisting of the application of 100 cycles of 10% of the deviator stress. Gibson and Li performed an analysis to determine a method whereby virtual preconditioning could be applied to test results obtained without preconditioning to approximate the change in response predicated by the actual preconditioning cycles. This virtual preconditioning consists of the removal of the first 51 cycles of data and the subsequent re-zeroing of the 52nd cycle to approximate the response curve after preconditioning cycles have been performed. Figure 6a shows all RLPD data after virtual preconditioning was applied. All further analysis was conducted after virtual preconditioning was applied.

Figure 6b clearly shows grouping among the average response of the mixtures. Mixture A accumulated the lowest strain among all mixtures. Mixtures B, F, and HMA 3 were grouped as were Mixtures C, D, E, HMA 1, and HMA 2. Groupings were not found to be influenced by volumetric properties.
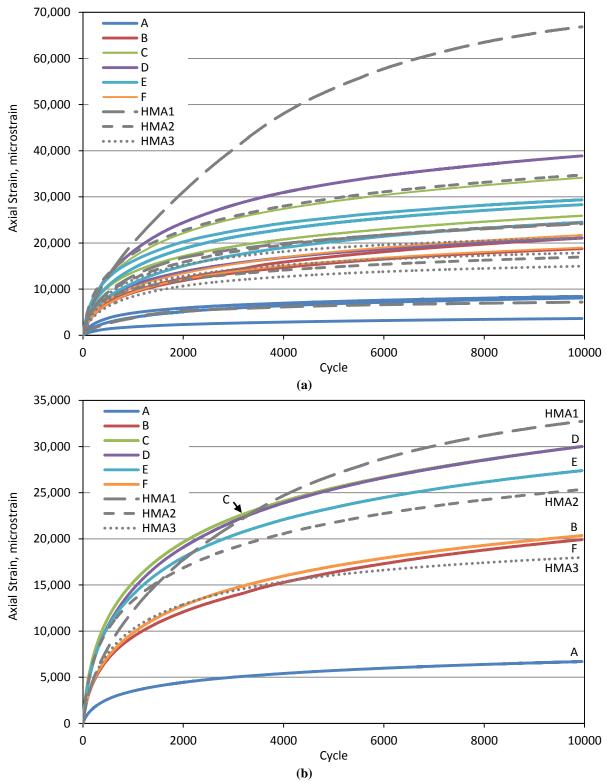
Figure 6. RLPD Results After Virtual Preconditioning for (a) All Specimens That Did Not Prematurely Fail and (b) Mixture Averages. HMA = hot mix asphalt.

Several alternate methods for analysis of RLPD data were used. These included the analysis of slope-intercept values, measured and predicted strain at 10,000 cycles, and slope at 2% strain. To determine the slope and intercept of the strain response, power law models were applied to the averaged RLPD data for each mixture and each individual specimen response curve. Table 10 summarizes average values of slope and intercept for each mixture; the average measured and predicted strains at 10,000 cycles; and the slope of the power law model at 2% strain. Figures 7 and 8 show the measured and predicted strains at 10,000 cycles, respectively.

Table 10. RLPD Analysis Values for Each Mixture

Mixture	Slope	Intercept	Measured Strain at 10,000 Cycles, με	Predicted Strain at 10,000 Cycles, με	Slope at 2% Strain
A	0.3674	248.64	8758	7331	0.048
B	0.3973	551.82	26041	21429	0.945
C	0.3737	1049.7	39714	32799	2.808
D	0.3977	847.65	38662	33038	2.810
E	0.3735	962.96	36280	30034	2.219
F	0.3945	585.96	27899	22175	1.025
HMA 1	0.5108	336.22	38335	37138	3.432
HMA 2	0.3566	1035.3	35777	27635	1.766
HMA 3	0.3474	822.25	23893	20165	0.711

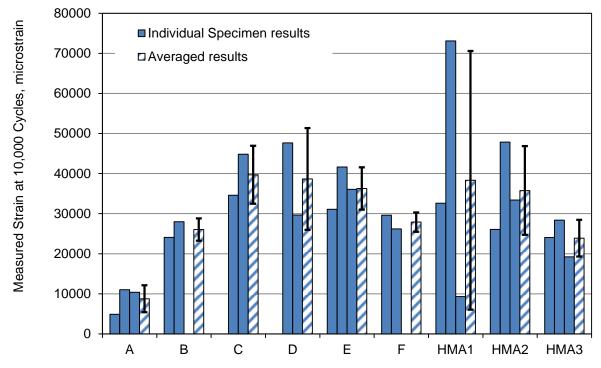RLPD = repeated load permanent deformation; HMA = hot mix asphalt.

Figure 7. Summary of Measured Strain at 10,000 Cycles. I-bars indicate the standard deviation of averaged results. HMA = hot mix asphalt.
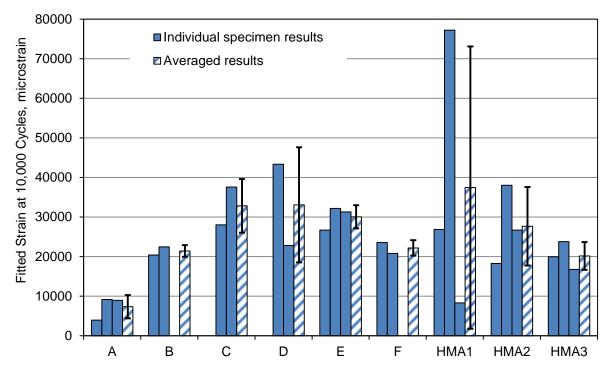
Figure 8. Summary of Fitted Strain at 10,000 Cycles Using Power Law Models. I-bars indicate the standard deviation of averaged results. HMA = hot mix asphalt.

Dongré et al. (2009) found that slope at a strain value of 2% correlated well with the flow number and suggested that this may be a robust indicator of rut resistance. They found that increased slope values may indicate an increased potential for the mixtures to rut. Figure 9 presents the slope of the fitted response curve at 2% strain, and Table 11 shows the flow number results. Examination of the results for these specimens did not indicate any correlation between the slope and flow number.

Mixtures were ranked in order of potential for rutting sensitivity based on each of the analysis methods described; these results are presented in Table 12: a ranking of 1 indicates the least susceptible mixture, and a ranking of 10 indicates the most susceptible mixture. Specific maximum values for each measure were not suggested. Table 12 indicates that, overall, Mixtures A, HMA 3, and B should be expected to perform better in rutting than the remaining mixtures based on intercept values, strains at 10,000 cycles, and the slope at 2% strain. Rankings were generally consistent among the intercept value, measured and predicted strains at 10,000 cycles, and slope at 2% strain. Slope rankings varied somewhat from other rankings, although the mixtures ranking first and third in the slope analysis (Mixtures HMA 3 and A, respectively) were the two highest ranked mixtures overall for the other analysis methods.
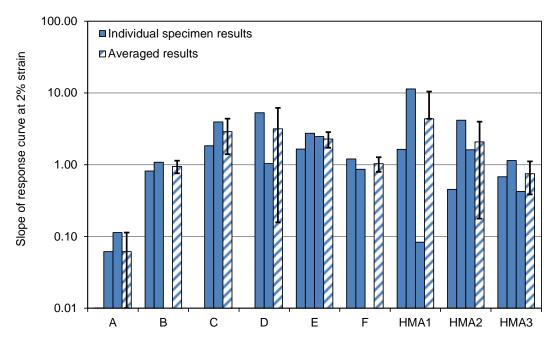
17

Figure 9. Slope of Fitted Response Curve at 2% Strain. I-bars indicate the standard deviation of averaged results. Note that missing specimens for Mixtures B, C, D, and F are caused by specimens failing prior to 2% strain. HMA = hot mix asphalt.

Table 11. Summary of Flow Number Results

Specimen	Mixture								
	A	**B**	**C**	**D**	**E**	**F**	**HMA 1**	**HMA 2**	**HMA 3**
1	10000	10000	328	547	10000	9949	-	10000	-
2	10000	10000	10000	10000	10000	10000	-	10000	10000
3	10000	977	10000	10000	10000	2512	10000	10000	10000
Average Flow No.	10000	6992	6776	6849	10000	7487	10000	10000	10000
Standard Deviation	0.0	5209.4	5584.1	5457.7	0.0	4308.6	-	0.0	-

HMA = hot mix asphalt; - = specimen did not reach flow during test.

Table 12. Mixture Ranking for Rutting Sensitivity by RLPD Analysis Method[a]

Mixture	Slope	Intercept	Measured Strain at 10,000 Cycles	Predicted Strain at 10,000 Cycles	Slope at 2% Strain
A	3	1	1	1	1
B	7	3	3	3	3
C	5	9	9	7	7
D	8	6	8	8	8
E	4	7	6	6	6
F	6	4	4	4	4
HMA 1	9	2	7	9	9
HMA 2	2	8	5	5	5
HMA 3	1	5	2	2	2

RLPD = repeated load permanent deformation; HMA = hot mix asphalt.
[a]Least susceptible to rutting sensitivity mixture ranking is 1.

Overlay Test

The Texas overlay test was conducted to assess the cracking potential of the foamed WMA mixtures. Replicate tests were performed on each mixture, with five replicate cores tested for each mixture. Loads were applied until failure or until 1,200 cycles were reached in accordance with TX-248-F (Texas Department of Transportation, 2009). Outlier analysis was applied such that any result falling outside one standard deviation of the average test result was discarded. Only two test results were discarded: one core each from Sections A and C were found to be outliers. The test results indicated that all foamed WMA mixtures should perform well, as all mixtures reached 1,200 load applications with the exception of Mixture C, which exceeded 1,000 cycles (see Figure 10).
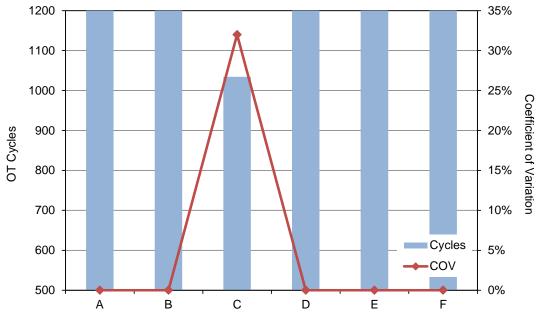

Figure 10. Overlay Test Results for WMA Cores. OT = overlay test; COV = coefficient of variation.

Binder Testing

Binders were extracted and recovered from cores collected for each mixture. Binders were tested after recovery and after aging in a pressure aging vessel (PAV). Table 13 summarizes the performance grading parameters for each binder from the foamed WMA sites. Data were not available for Mixture F. Table 14 summarizes the performance grading parameters for each control binder.

Table 13. Performance-Graded (PG) Binder Properties of Foamed WMA Sites

Property	Site					
	A	B	C	D	E	F
Dynamic Shear, 10 rad/sec, specification: G*/sin delta > 2.20 kPa						
Recovery G*/sin delta, 76ºC		2.222	2.358	2.753	4.294	N/A
Recovery G*/sin delta, 82ºC	9.27	1.098	1.144	1.321	2.037	
Recovery G*/sin delta, 88ºC	4.464					
Recovery G*/sin delta, 94ºC	2.213					
Recovery G*/sin delta, 100ºC	1.112					
Recovery G*, 76ºC		2.207	2.344	2.738	4.252	
Recovery G*, 82ºC	9.132	1.095	1.14	1.318	6.026	
Recovery G*, 88ºC	4.426					
Recovery G*, 94ºC	2.203					
Recovery G*, 100ºC	1.109					
Recovery phase angle, 76ºC		83.39	83.83	84.12	81.95	
Recovery phase angle, 82ºC	80.09	85.30	85.67	85.92	84.16	
Recovery phase angle, 88ºC	82.45					
Recovery phase angle, 94ºC	84.43					
Recovery phase angle, 100ºC	85.93					
Recovery failure temperature, ºC	*94.12*	*76.08*	*76.57*	*77.83*	*81.38*	
Dynamic Shear, 10 rad/sec, specification: G* sin delta < 5000 kPa						
PAV G* sin delta, 25.0ºC		6066	6381			N/A
PAV G* sin delta, 28.0ºC		4312	4495	5593	6589	
PAV G* sin delta, 31.0ºC			3112	4146	4712	
PAV G* sin delta, 40.0ºC	7294					
PAV G* sin delta, 45.0ºC	4105					
PAV G*, 25.0ºC		9.09E+06	9.44E+06			
PAV G*, 28.0ºC		6.14E+06	6.30E+06	8.28E+06	9.98E+06	
PAV G*, 31.0ºC			4.16E+06	5.88E+06	6.75E+06	
PAV G*, 40.0ºC	1.01E+07					
PAV G*, 45.0ºC	5.33E+07					
PAV phase angle, 25.0ºC		41.84	42.54			
PAV phase angle, 28.0ºC		44.60	45.49	42.51	41.30	
PAV phase angle, 31.0ºC			48.40	44.81	44.26	
PAV phase angle, 40.0ºC	46.26					
PAV phase angle, 45.0ºC	50.37					
PAV failure temperature, ºC	*43.29*	*26.07*	*27.06*	*29.12*	*30.47*	
Creep Stiffness, 60 sec, specification: Stiffness < 300 MPa and m-value > 0.300						
Stiffness, 0ºC	262					N/A
M-value, 0ºC	0.250					
Stiffness, -6ºC		117	126	140	168	
M-value, -6ºC		0.338	0.318	0.313	0.304	
Stiffness, -12ºC	900	251	262	288	313	
M-value, -12ºC	0.164	0.294	0.287	0.276	0.263	
ΔT_c, ºC	8.3	2.2	3.6	4.2	5.0	
Stiffness failure temperature, ºC	*-11.3*	*-23.4*	*-23.1*	*-22.3*	*-21.6*	
M-value failure temperature, ºC	*-3.0*	*-21.2*	*-19.5*	*-18.1*	*-16.6*	
PG Grade	**-**	**76-16**	**76-16**	**76-16**	**76-16**	

WMA = warm mix asphalt; N/A = data not available; - = grade does not conform to AASHTO M320 (AASHTO, 2014).

Table 14. Performance-Graded (PG) Binder Properties of HMA Sites

Site	HMA 1	HMA 2	HMA 3
Dynamic Shear, 10 rad/sec, specification: G*/sin delta > 2.20 kPa			
Recovery G*/sin delta, 76ºC	6.408	4.410	5.832
Recovery G*/sin delta, 82ºC	3.056	2.061	2.809
Recovery G*/sin delta, 88ºC	1.510		1.396
Recovery G*, 76ºC	6.308	4.371	5.737
Recovery G*, 82ºC	3.029	2.051	2.780
Recovery G*, 88ºC	1.503		1.389
Recovery phase angle, 76ºC	79.90	82.37	79.59
Recovery phase angle, 82ºC	82.37	84.49	82.03
Recovery phase angle, 88ºC	84.43		84.11
Recovery failure temperature, ºC	*84.83*	*81.48*	*84.14*
Dynamic Shear, 10 rad/sec, specification: G* sin delta < 5000 kPa			
PAV G* sin delta, 28.0ºC		6454	5197
PAV G* sin delta, 31.0ºC	5418	4590	3776
PAV G* sin delta, 34.0ºC	3868	3203	
PAV G*, 28.0ºC		9.61E+06	7.53E+06
PAV G*, 31.0ºC	7.90E+06	6.47E+06	5.26E+06
PAV G*, 34.0ºC	2.38E+06	4.30E+06	
PAV phase angle, 28.0ºC		42.18	43.65
PAV phase angle, 31.0ºC	43.33	45.18	45.89
PAV phase angle, 34.0ºC	45.99	48.15	
PAV failure temperature, ºC	*31.72*	*30.21*	*28.36*
Creep Stiffness, 60 sec, specification: Stiffness < 300 MPa and m-value > 0.300			
Stiffness, -6ºC	179	177	148
M-value, -6ºC	0.313	0.308	0.327
Stiffness, -12ºC	362	349	294
M-value, -12ºC	0.264	0.255	0.278
ΔT_c, ºC	-2.8	-3.8	-2.9
Stiffness failure temperature, ºC	*-20.4*	*-20.7*	*-22.2*
M-value failure temperature, ºC	*-17.6*	*-16.9*	*-19.3*
PG Grade	**82-16**	**76-16**	**82-16**

HMA = hot mix asphalt.

With the exception of Binder A, all foamed binders graded to a PG 76-16 binder. It is clear that in-service aging is not well simulated by laboratory aging, as all of the foamed WMA mixtures were produced with virgin PG 64-22 binders and varying RAP contents from 15% to 26%. Even Binder D, which was produced with PG 64-22 binder and 15% RAP, aged to a PG 76-16 binder after 5 years in service.

Binder A was exceptionally stiff with a high failure temperature of 94.1ºC and a low failure temperature of -3.0ºC; the failure temperatures fall outside those established in AASHTO M 320, so this binder did not meet any performance grade. It is unclear why Binder A exhibited such stiffness; RAP content may have been an influence, but this is unlikely as four additional foamed WMA mixtures were produced using PG 64-22 binder and RAP contents of 25% to 26% and only Mixture A exhibited the drastic increase in stiffness. The behavior seen in recovered Binder A was also supported by the mixture testing, as Mixture A showed generally higher dynamic modulus across most of the range of testing and resistance to deformation in the RLPD analysis. Interestingly, Mixture A did not show any increased tendency toward cracking in the overlay test that would be expected from a binder this stiff. This is particularly surprising given

the ΔT_c value for Binder A in Table 13. ΔT_c is calculated as the difference between the critical temperature for stiffness and the m-value for a binder and has been found to be an indicator of non–load-related cracking susceptibility (Anderson et al., 2011).

Control binders HMA 1 and HMA 3 graded to a PG 82-16 binder; HMA 2 graded to a PG 76-16 binder but was nearly a PG 82-16 binder. In general, the decreased aging of the foamed binders may be due to slightly reduced production temperatures.

SUMMARY OF FINDINGS

- Foamed WMA mixtures were found to have varying air void contents and permeability, similar to HMA mixtures.

- Dynamic modulus results indicated that most foamed WMA mixtures compared well, with the exception of Mixture A, which exhibited a different behavior trend.

- Black space analysis generally indicated that foamed WMA mixtures and HMA mixtures should respond similarly.

- RLPD test results were found to require virtual preconditioning, as preconditioning was not performed on the specimens in this study. In NCHRP Project 09-30A, a non-reported preconditioning stage was used wherein 100 cycles at a level of 10% of the deviator stress was applied (Gibson and Li, 2013); this was replicated with the removal of 51 cycles of test data and the re-zeroing of the remaining data.

- RLPD test results were used with various methods of analysis to rank mixtures. Intercept, measured strain at 10,000 cycles, predicted strain at 10,000 cycles, and slope at 2% strain generally ranked mixtures similarly. Slope analysis resulted in changes to the ranking trend, although the mixtures ranked first and third were the two highest ranked mixtures using all other analysis methods.

- Overlay test results indicated similar performance, with all mixtures reaching 1,200 cycles except Mixture C, which exceeded 1,000 cycles.

- Binder grading indicated that foamed WMA mixtures aged less than HMA mixtures, likely because of the slightly lowered production temperatures. However, all mixtures, regardless of virgin binder grade and RAP content, aged to at least a PG 76-16 binder, one to two grades higher than the specified binder.

CONCLUSIONS

- *The study verified that foamed WMA mixtures should be expected to perform similarly to HMA mixtures.*

- *RLPD testing should be modified such that preconditioning is applied for confined specimens.*

- *Binder aging is causing a significant change in binder grade in-service from that which was specified at construction.*

RECOMMENDATIONS

1. *VDOT's Materials Division should continue to allow foamed WMA as currently permitted in the specifications.* Foamed WMA appears to perform similarly to HMA and offers no impediment to continued use.

2. *The Virginia Transportation Research Council (VTRC) should modify internal RLPD testing protocols to provide specimen preconditioning, especially in the case of confined specimen testing.* This will provide consistency with the analysis used in NCHRP Project 09-30A (Von Quintus et al., 2012) that can be incorporated into rutting analysis of specific mixtures in place of the universal rutting model.

3. *VTRC should continue to monitor the performance of foamed WMA sections to assess their lifetime performance.* This study evaluated 4 to 6 years of performance; however, pavement surfaces are expected to remain in service considerably longer than that period. Validating that WMA does not shorten the lifetime of the pavement surface is a worthwhile effort.

4. *VTRC should continue to investigate the impact of aging on binder grade and performance.* Binder aging can significantly affect the performance of asphalt mixtures and unless addressed may compromise the lifespan of Virginia mixtures. Further work is needed to validate the impact of aging and consider means to mitigate it if necessary.

BENEFITS AND IMPLEMENTATION

Benefits

The benefit of implementing Recommendation 1 is that the use of foamed WMA will continue as currently permitted in the specifications. No changes to practice are necessary.

The benefit of implementing Recommendation 2 is that VTRC test results will support the use of rutting analysis for specific mixtures instead of the application of the universal rutting model. This will allow for more accurate analysis of rutting susceptibility in design and may improve the ability of analysts to determine the best among competing designs for improved efficiency.

The benefit of implementing Recommendation 3 is that it will provide confidence in assessing the lifetime performance of WMA mixtures. There is always the potential for differences in performance to occur throughout the life of a mixture; validation across a lifetime will ensure that performance can be expected to be equivalent.

The benefit of implementing Recommendation 4 is that if aging can be addressed or mitigated such that the mixture lifespan is increased, there is potential for cost savings through optimized maintenance treatments and delayed rehabilitation needs.

Implementation

With regard to Recommendation 1, VDOT's Materials Division will make no changes to VDOT's specifications regarding the use of foamed WMA based on the outcomes of this study. This study validated the assumption that the properties and performance of foamed WMA are similar to those of HMA. However, discussion during the review of this work identified concerns beyond the scope of the study about allowable WMA mixture temperatures at the time of placement. Section 315.04 of VDOT's *Road and Bridge Specifications* (VDOT, 2016) allows placement of WMA at mixture temperatures above 200°F. This minimum mixture temperature was determined as being nondiscriminatory to specific WMA technologies, as it is the minimum applicable for WMA produced using additives; however, it is not practically appropriate for WMA produced using foaming technologies. The discussion suggested that WMA temperatures should be monitored during construction to determine if reduced temperatures during compaction are leading to poor in-place density, indicating a need to revisit the specification limits. This issue will be addressed through the submission by VTRC and VDOT's Materials Division of a Research Needs Statement to VTRC's Asphalt Research Advisory Committee at the next meeting to initiate discussion of the topic and assess the need for a research study.

With regard to Recommendation 2, VTRC has updated internal test procedures for the RLPD test to include specimen preconditioning. This will provide more reliable data for use in Pavement ME Design software.

With regard to Recommendation 3, VTRC will continue to monitor the performance of the mixtures and sites evaluated in this study to determine if outcomes require further implementation efforts. This will be accomplished according to the following plan:

- Revisit the sites immediately prior to surface replacement.

- Collect pavement cores for laboratory testing. Pavement performance data will be gathered from VDOT's PMS.

- Evaluate changes in mixture properties and pavement performance to assess the evolution of properties and associated performance over time.

With regard to Recommendation 4, VTRC will continue to focus analysis efforts on identifying the factors that influence binder aging and its impact on mixture performance. This will be accomplished initially within the scope of other current projects underway that involve binder testing and mixture performance analyses. If these projects do not provide sufficient data to allow conclusions, additional work will be proposed through VTRC's Asphalt Research Advisory Committee.

ACKNOWLEDGMENTS

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