FINAL REPORT

A CASE STUDY OF THE DESIGN, INSTALLATION, AND EARLY PERFORMANCE OF A NINETEEN-MILLIMETER SUPERPAVE MIX

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ABSTRACT

An attempt was made to design and construct a coarse matrix high binder (CMHB) mix. When the design of the CMHB mix was unsuccessful, a coarse 19.0-mm Superpave mix was substituted. This report describes the attempted design of the CMHB mix, as well as the design, construction, and early performance of the Superpave mix.

Construction problems with the Superpave mix included low gyratory voids in field samples and tenderness during the compaction process. Mix adjustments in the gradation and asphalt content did not remedy the problems. The tenderness also contributed to inadequate pavement density. Although field samples indicated low gyratory voids, no overconsolidation, as would be evidenced by bleeding, has been observed. Therefore, it appears that this mix was able to contain more asphalt than predicted by the gyratory voids. The average rut depth of all sections was about 4 mm after 27 months.

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INTRODUCTION

In 1990, a 21-member group of highway transportation engineers and officials toured six European countries to learn about the success of their asphalt pavements (AASHTO, 1990). Of particular interest was the design of asphalt wearing courses, use of asphalt modifiers, and the reported superior durability of their pavements. One of the European innovative mix designs that was brought back to the United States and implemented was the stone matrix asphalt (SMA). SMA has a strong coarse aggregate structure to resist permanent deformation, and a rich mastic fills the aggregate voids to provide maximum durability.

The SMA costs more than conventional dense-graded mixes because of special aggregate requirements, asphalt modifiers, and construction requirements. Concurrently with the development of SMA in the United States in the early 1990s, the Texas DOT was developing a similar gap-graded asphalt surface mix known as the coarse matrix high binder (CMHB) mix. This mix was often referred to as the “poor man’s SMA” because it was less expensive than SMA. Although the material requirements were not as strict as those of the SMA, claims were made that the mix had stone-to-stone contact, which produced resistance to permanent deformation (Asphalt Contractor, 1993). Also, the mix used thick asphalt films that promoted good durability.

The Virginia Department of Transportation (VDOT) let a contract to use CMHB mix to resurface approximately 4.8 km of four-lane highway. Two contractors showed interest in supplying the mix for the project, and they submitted bids. The low bidder submitted a mix design that complied with the gradation and asphalt content design limits, but they had no means to check for creep properties as required by the specification that was adapted from the Texas specification. The Virginia Transportation Research Council (VTRC) had the only lab in VDOT that could perform the creep tests. Creep test results by VTRC did not comply with design specifications, even with adjustments in the gradation. Meanwhile, a major CMHB mix project in Texas on I-20 failed soon after construction (Maghsoud Tahmoressi, personal communication, 1995). This failure coupled with the problem of obtaining passing test results on the submitted mix design prompted a look at alternative mixes that might provide the same durability benefits as those attributed to the CMHB mix.

VDOT had committed itself to implementing the Superpave design system, so the researcher decided to switch to a Superpave design for the project. It was hoped that use of a coarse mix with thick asphalt films designed with the Superpave system would retain the durability characteristics of the CMHB mix.

PURPOSE AND SCOPE

The initial purpose of this project was to design, construct, and evaluate a CMHB mix. However, when problems occurred in obtaining a satisfactory design, the mix was switched to a 19.0-mm Superpave mix. Since only two Superpave projects had been built in Virginia, this project provided additional experience for both contractor and highway personnel, especially with a 19.0-mm mix. The intent of the study was not to draw general conclusions but to provide a case study of a specific Superpave mix. It was used as a learning step in the Superpave implementation process.

METHODS

General

The first step of the study was to check the design of the CMHB mix that was not placed and then design the Superpave mix that was substituted. The second step was to design the substituted Superpave mix, observe the manufacture and construction, ascertain any problems that existed, and attempt to remedy any problems that existed. Samples were collected during construction and tested to determine compliance with the specifications. Finally, the sections were viewed to determine any evidence of premature distresses, and rut depths were measured periodically.

Test Methods

Creep Tests

Creep tests were performed on the submitted mix design for the CMHB mix in accordance with the Texas Static Creep Test (Tex 231-F). Specimens were prepared with the Marshall hammer at the design void content (4 percent). The air void content was determined, the ends of the specimens were capped with a plaster compound, and the specimens were placed in a controlled temperature chamber and maintained at 40^{o}C for 4 hours prior to the start of the test. The cylindrical specimens were then loaded in the axial direction at a pressure of 68.5 kPa for 1 hour. The load was removed, and the specimen was allowed to rebound for 10 minutes before the strain was measured. Strain was also recorded during the test so that a creep plot (strain vs. time) could be obtained.

Superpave Gyratory Volumetrics

Gyratory volumetric properties were determined for the Superpave mix design in accordance with AASHTO Provisional Standard TP4 (AASHTO, 1998). The design traffic level and design temperature selected for Route 460 was 4 million equivalent single-axle loads and 38 ^oC, resulting in 96 gyrations for N_{design}.

Asphalt Content and Gradation

Asphalt content was determined according to Virginia Test Method (VTM – 102), Determination of Asphalt Content from Asphalt Paving Mixtures by the Ignition Method (VDOT, 1995). The asphalt cement was burned from the aggregate at 538 ^oC and a sieve analysis was performed on the remaining aggregate.

Georgia Loaded-Wheel Tests

Rut prediction tests were performed with the Georgia loaded-wheel tester (GLWT) according to Georgia Test Method GDT-115 with some change (GDOT, 1997). The 75 mm x 125 mm x 300 mm beam specimens were compacted in a rolling wheel compactor using a polycarbonate compaction block to a target air void content of 7 percent. The beams were loaded with a reciprocating wheel loaded at a force of 445 N. The load was transferred to the beam through a hose with an outside diameter of 29.5 mm pressurized to 700 kPa. The test was performed for 8,000 cycles (16,000 passes) at 40 ^oC, after which rut depths were measured at three locations along the beam. Subsequently, the test method was revised to include a higher wheel load, higher hose pressure, and higher test temperature. Testing by the VTRC indicates that the more severe revised test conditions produce rut depths that are more indicative of pavement performance.

Pavement Rut Depth Measurements

Pavement rut depth measurements were made with a van equipped with a three-laser sensor road profiling system. The vehicle had three height sensors positioned above the wheel paths and center of the lane. The distance from the sensors to the pavement surface was measured, and the height of the hump in the center of the lane above the wheel paths was calculated as the rut depth. Data were collected at normal highway speeds.

Voids Determinations

The percent air voids in pavement samples that were sawed or cored from the pavement was determined according to ASTM's *Standard Test Method for Percent Air Voids in Compacted Dense and Open Bituminous Paving Mixtures*, ASTM D 3203. The voids were computed from the specimen bulk specific gravity and theoretical maximum specific gravity by the following formula:

$$\text{VTM} = 100\left(1 - \frac{\text{bulk specific gravity of core}}{\text{theoretical maximum specific gravity of mix}}\right)$$

where

VTM = percent air voids

Bulk specific gravity = bulk specific gravity of the specimen

Theoretical maximum specific gravity = theoretical maximum specific gravity of the specimen

Permeability Tests

Falling head permeability tests were performed on several cores removed from the pavement after 27 months. The tests were performed by a test method for asphalt specimens under development by an ASTM Committee D-4 task group.

The test method is based on ASTM Test Method D 5084, Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter (ASTM, 1996). A layer of petroleum jelly was applied to the outside circumferential surface before it was placed into the apparatus to prevent water from seeping between the flexible membrane and the specimen. The pressure applied to the membrane was 96 kPa. The coefficient of permeability was computed by the formula:

$$k = \frac{aL}{At}\ln\left(\frac{h_1}{h_2}\right)$$

where

k = coefficient of permeability
a = area of stand pipe
L = length of sample
A = cross-sectional area of sample
t = time over which water head is allowed to fall
h_1 = water head at the beginning of the test
h_2 = water head at the end of the test.

RESULTS AND DISCUSSION

Mix Design of CMHB

The mix was originally specified to be a CMHB mix similar to that developed by the Texas DOT. The specified gradation and the design gradation submitted by the contractor are listed in Table 1. Table 2 lists the mix proportions and source of each material.

Table 1. Mix Design of CMHB Mix

Sieve (mm)	Specification	Submitted Job Mix
19.0	100	100
12.5	94-100	95
9.5	55-75	75
4.75	30-45	39
2.36	18-28	25
0.6	8-18	16
0.3	7-15	13
0.075	6-10	8.5

Table 2. Proportions and Source of Materials for CMHB Mix

Material	Percentage	Source
No. 68	9.5	Vulcan Materials, Inc., Emporia, Va.
No. 10	25.0	Vulcan Materials, Inc., Emporia, Va.
No. 78	62.0	Vulcan Materials, Inc., Emporia, Va.
Ground lime	3.5	Germany Valley Limestone, Riverton, W.Va.
AC-30 (tested as a PG 64-22)	5.3	Koch Materials Co., Newport News, Va.

The mix was designed with a 50-blow Marshall compactive effort, which resulted in an optimum asphalt content of 5.8 percent. The specification limits adopted from Texas were less than 0.0005 for permanent strain, less than 2.41×10^{-10} MPa for slope of the steady state portion of the creep curve, and greater than 41.3 MPa for the creep stiffness. The creep tests performed on the mix failed all of these specification limits.

During the design phase of our experiment the Texas DOT had some major bleeding problems on one of their CMHB projects. At first, they thought it was caused by the asphalt binder, but this premise was later dismissed after they performed a forensic study. Because of this failure and our unfamiliarity with design process, we were somewhat concerned about the potential success of the project. Also, Superpave was becoming accepted, and there were plans to implement it almost 100 percent statewide in the future; therefore, a decision was made to switch from the CMHB design to a Superpave design using the same basic materials. The resultant mix is the only Superpave 19.0-mm surface mix that has been produced and placed in Virginia.

Mix Design Using Superpave

It was desirable to require as few changes as possible in materials so that the contractor could produce the mix for the same price as the original bid price of the CMHB mix. Since the contractor did not have a Superpave gyratory compactor, the VTRC had to perform the mix design.

Materials were obtained from the contractor that were to be used in the original CMHB mix. Also, No. 8 aggregate from the same source and natural class A sand from Glover Construction, Inc., were made available by the contractor.

A total of nine blends were used in an attempt to achieve acceptable mix properties (Table 3). One of the reasons for considering the CMHB mix was that it contained thick asphalt films for maximum durability and it was desirable to still strive for this property in the alternative Superpave mix. Mix design was an artful trial and error process striving for thick asphalt films and acceptable void properties. All of the first eight designs had asphalt contents less than 5 percent and the author wanted to keep the design asphalt content above 5 percent to strive for maximum durability. The eighth mix design was an attempt to duplicate the gradation of an intermediate mix that the contractor had produced previously; however, the blend that met the intermediate mix gradation the closest contained excess minus-0.075 mm material resulting in a low VMA and low asphalt content. On the ninth attempt a design was achieved that gave 5.2 percent asphalt content with other acceptable gyratory properties. A limitation of the design process was that the number and sizes of aggregates were confined to those made available by the contractor. It should also be noted that the coarse Superpave gradation was not as sensitive to changes as you would expect a conventional dense graded mix to be.

Construction

Approximately 4.8 km of four-lane roadway on Route 460 east of the Isle of Wight County line were resurfaced. The outside traffic lanes were composed of 230 mm of portland cement concrete covered by approximately 130 mm of asphalt surface mix overlays. The inside lanes were composed of multiple layers of asphalt surface mixes. Before 38 mm of Superpave mix was placed on all of the lanes, the asphalt overlays on top of the concrete were milled and removed and then 130 mm of BM-2 base mix was placed on top of the concrete. BM-2 is a dense-graded base mix with a 25-mm maximum nominal aggregate size. In some cases the underlying support for the concrete slabs was poor and, as expected, transverse reflection cracking has appeared in the new surface mix.

Paving of the 19.0-mm surface mix was done by Henry S. Branscome, Inc., during the period of August 12, 1996, to August 20, 1996. Weather was satisfactory, with temperatures ranging from 17 to 21^{o}C and 24 to 30^{o}C for the lows and highs, respectively. Rain stopped paving at 4:30 P.M. on August 12 and prevented paving all day on August 13. The paving site was 10 to15 minutes driving time from the asphalt plant, and temperature of the mix at the paver averaged 143 ^{o}C.

Table 3. Trial Blends and Gyratory Properties

Mix	1	2	3	4	5	6	7	8-IM	9-JM	Spec Design Limits	Restricted Zone
Percentage of Material											
No. 68	34	47	45	15	48	45	13	32	22		
No. 78	33	18	25	50	20	25	50	23	50		
No. 10	33	35	30	35	20	22	26	35	21		
Sand					12	8	11	10	7		
Sieve	**Gradation, Percent Passing**										
25.0	100	100	100	100	100	100	100	100	100*	100	
19.0	98	97	97	99	97	98	99	96	97*	90-100	
12.5	86	82	82	92	81	82	93	87	90*		
9.5	69	67	66	75	66	66	76	74	70		
4.75	40	42	38	42	40	38	44	51	38*		
2.36	26	28	25	28	28	26	31	38	25	23-49	34.6
0.6	14	16	14	15	16	14	17	21	15*		16.7-20.7
0.3	11	12	10	11	9	9	10	14	10		13.7
0.075	5.4	5.7	5.0	5.5	3.7	3.9	4.2	6.1	4.5*	2.0-8.0	
Property	**Gyratory Design Values**										
AC, %	4.5	4.2	4.7	4.4	4.2	4.4	4.2	4.0	5.2		
VMA, %	13.8	13.2	14.3	13.8	13.6	14.0	13.6	12.9	15.1	> 13.0	
VFA, %	71.1	69.8	72.0	71.1	70.5	71.5	70.6	69.1	73.6	65-75	
% G_{mm} @ N_{init}	85.2	85.8	85.2	85.4	86.9	86.3	87.3	87.9	86.2	$\leq$ 89	
% G_{mm} @ N_{max}	97.3	96.9	97.4	97.3	97.3	97.4	97.2	97.0	97.3	$\leq$ 98	

*Control sieves.

A gyratory compactor was moved into the district materials lab, and tests were performed daily by VTRC personnel to determine whether satisfactory properties were being achieved. After paving the first day, the tests showed that the mix was too dense (low voids) by Superpave standards and it tended to push under the breakdown roller after two passes. Normally, void levels below the minimum allowable level would result in the pavement developing bleeding and possibly rutting. Although the mix contained low voids, VDOT's prior experience with a Superpave mix in 1995 indicated that Superpave mixes could probably tolerate extra asphalt cement and still function well. Therefore, there was not major concern over the low voids. But, to be safe, another lab mix design was done the next day during the rain delay to try to correct the problem of low voids and tenderness.

Several changes in the blends and asphalt contents were made during the project in an attempt to refine the mix design, which resulted in the production of eight mixes (see Figure 1).

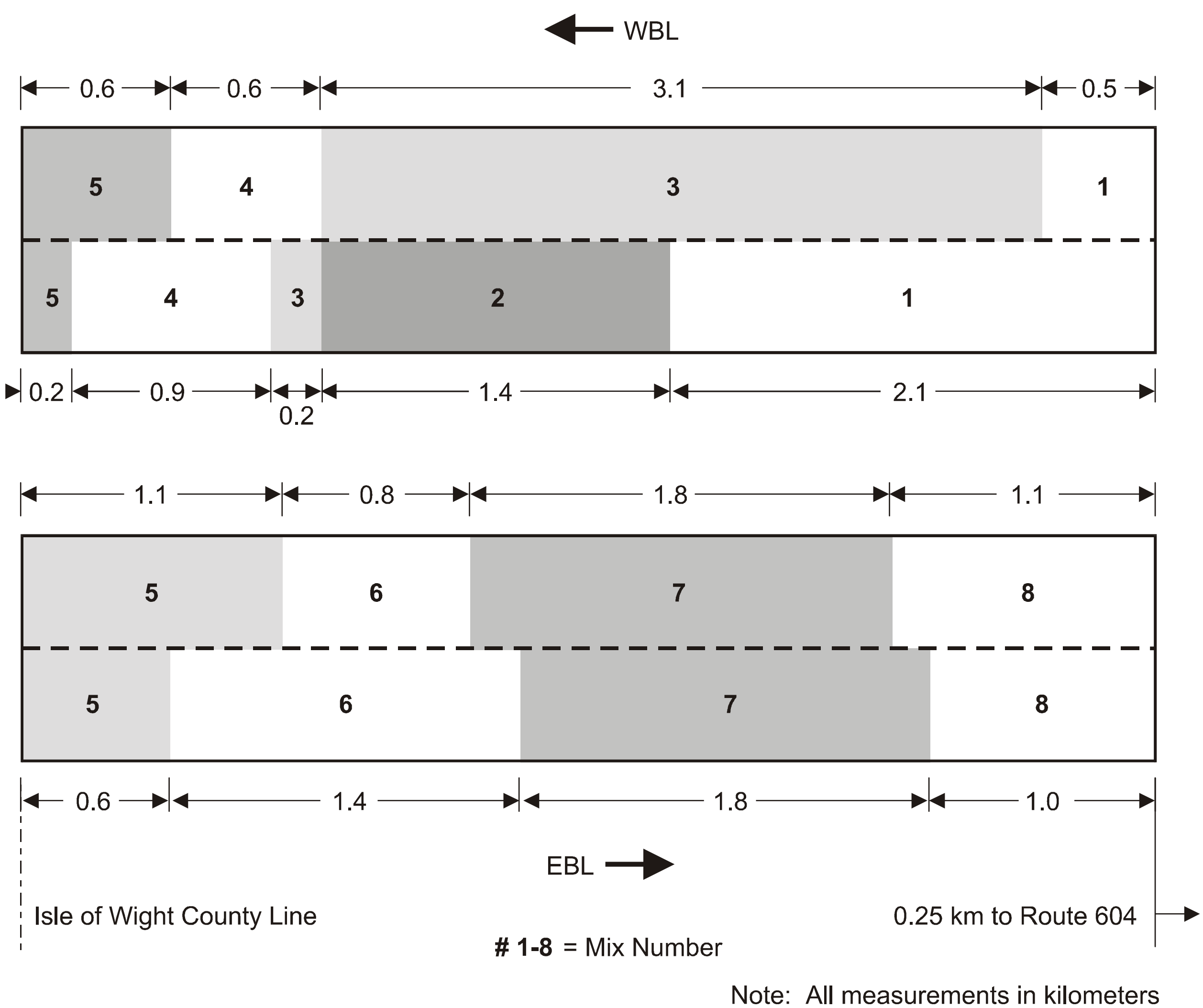

Figure 1. Test Section, Route 460

However, the problems with low voids in lab specimens and tenderness during field compaction were not solved. Texas also reported tenderness problems, especially with 19.0-mm mixes (Tahmoressi, 1997), and other states have reported at unrecorded meetings that some 19.0-mm mixes are tender during compaction. It has been observed by others that a temperature zone exists where the mix is tender during compaction. They found it necessary to roll the mix while it was hot, stop rolling as the mix temperature was in the tenderness zone, and complete the rolling when the mix cooled below the tenderness zone (Deahl, 1999). Although the tenderness zone was observed on the Route 460 project, the rolling technique described was not known at that time and, therefore, was not used.

Lab Tests

Table 4 shows input blends for each of the field mixes and the test results of asphalt content and gradation tests. Table 5 shows the results of the gyratory tests, 75-blow Marshall tests, and GLWT rutting tests. The value listed for each mix is an average of tests on two to five samples taken during production. The number of samples per mix varied because the quantities of the mixes were different. Table 6 summarizes some test results and placement observations.

After problems with low lab voids and tenderness during rolling with the first mix, the blend of the second mix was adjusted to be coarser. However, the blend appeared to be too coarse on the road and was still unstable under the rollers; therefore, the contractor attempted to adjust the blends again for mix 3. The tested gradations for mix 3 did not show an appreciable change from those for mix 2; however, the tenderness was not as severe. The contractor was able to apply two static passes and one vibratory pass with the breakdown roller and two or three passes with the finish roller after the mix had cooled to 110 ^{o}C before tenderness was evident. The blend was changed slightly for mix 4, but the mix was not as stable during rolling even though the tested gradation changed only slightly from that of mix 3.

The asphalt content was decreased to 5.0 percent for mix 5 in an attempt to increase gyratory voids and decrease tenderness. The mix was still unstable during the rolling operation, and it would tolerate only one or two passes with the breakdown roller. The mix had to cool more before it was rolled with the finish roller. For mix 6 the blend was switched back to the same blend as used in mix 3 since that mix seemed to have slightly better resistance to the tenderness problem. The change seemed to make the mix more stable under the rolling process. This aggregate blend was maintained for the remainder of the project. The asphalt content was lowered to 4.8 percent for mix 7. When the contractor obtained 4.5 percent VTM for a 75-blow Marshall test, which was considered high, and the mix looked dry, the asphalt content was increased to 5.0 percent for mix 8.

Different aggregate blends were used for mix 1; mix 2; mixes 3, 6, 7 and 8; and mixes 4 and 5. The blend corresponding to that used in mixes 3, 6, 7, and 8 seemed to be less tender during the rolling process. However, when the range of gradation and asphalt content test values achieved for all of the mixes was examined, only mix 1 would be near the outer tolerance limits for a single mix. In other words, with the possible exception of mix 1, the other mixes were within the normal production tolerance range for a single mix and probably should not produce

Table 4. Test Results of Field Samples

Mix No.	1	2	3	4	5	6	7	8	Job Mix Design	Restricted Zone
	Mix Inputs									
No. 68	22	42	35	33	33	35	35	35	22	
No. 78	50	33	40	40	40	40	40	40	50	
No. 10	21	18	18	20	20	18	18	18	21	
Sand	7	7	7	7	7	7	7	7	7	
AC, %	5.2	5.2	5.2	5.2	5.0	5.0	4.8	5.0	5.2 $\pm$0.3	
Sieve, mm	**Gradation**									
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100*	
19.0	99.7	98.2	98.1	98.5	98.7	97.5	98.6	98.6	97 $\pm$4*	
12.5	91.5	86.8	85.7	86.7	85.1	86.7	87.4	84.5	90 $\pm$4	
9.5	78.0	72.2	72.2	72.5	71.2	73.5	74.2	70.4	70 $\pm$4	
4.75	44.1	39.6	40.7	40.2	38.7	42.6	42.6	40.4	38 $\pm$4	
2.36	28.6	26.2	27.1	26.2	25.3	28.5	28.6	27.9	25 $\pm$4*	34.6
0.6	17.1	16.1	16.5	15.5	15.2	16.7	16.9	17.2	15 $\pm$3	16.7-20.7
0.3	12.4	11.8	12.2	11.5	11.3	12.3	12.5	12.8	10$\pm$2.5	13.7
0.075	5.2	5.3	5.5	5.3	5.1	5.5	5.6	5.8	4.5 $\pm$1*	
AC, %	5.4	5.4	5.3	5	5.1	5.2	5	5.2	5.2 $\pm$0.3	

Note: Values in shaded cells are outside the spec limits. Tolerance limits for job mix are for average of 4 tests.
*Design sieves.

Table 5. Gyratory, Marshall, and GLWT (Rutting) Results for Field Samples

Mix No.	1	2	3	4	5	6	7	8	Spec Design Limits
Gyratory Volumetric Properties									
% G_{mm} @N_{init}	90.1	87.4	88.0	87.1	87.1	88.4	87.7	88.5	$\leq$ 89
% G_{mm} @ N_{des}	98.0	97.3	97.7	96.4	96.8	97.9	97.3	98.2	96
% G_{mm} @ N_{max}	99.0	98.6	99.0	97.6	98.0	99.2	98.5	99.3	$\leq$ 98
% VTM @ N_{des}	2.0	2.7	2.3	3.6	3.2	2.1	2.7	1.8	4.0 $\pm$ 1.5
% VFA @ N_{des}	84.6	83.1	83.4	75.8	77.5	84.8	80.0	86.1	65-75 **
%VMA @ N_{des}	13.3	14.0	13.8	14.5	14.4	13.8	13.5	13.2	> 13.0
75-blow Marshall Volumetric Properties									
%VTM	2.5	2.8	2.4	3.2	3.5	2.6	3.1	2.4	3-6 *
%VMA	13.7	14.0	13.9	14.1	14.6	14.1	13.9	13.8	> 14.0 *
%VFA	82.1	80.6	82.7	77.6	76.2	81.6	77.6	82.2	65-80 *
GLWT Rut Depth									
%VTM	6.8	6.7	6.5	6.5	7.2	6.2	5.8	5.9	
Rutting, mm	3.9	4	3.9	2.8	3.4	3.2	1.9	3.0	< 8.0 **

*Previous VDOT design values, but not in effect for this project.
**Georgia guideline.
***Based on design. No field tolerance assigned.
Note: Values in shaded cells are outside spec limits.

Table 6. Field Samples Mix Properties and Construction Observations

Mix	Gradation*	Lab Voids		Fines/asphalt ratio**	Film thickness, μm	Comments
		Gyratory	Marshall			
1	A	Low	Low	1.09	8.3	Tender
2	B	Low	Low	1.10	8.7	Tender
3	C	Low	Low	1.12	8.8	Less tender
4	D	OK	OK	1.14	9.1	Tender
5	D	OK	OK	1.04	9.5	Tender
6	C	Low	Low	1.12	9.0	Less tender
7	C	Low	OK	1.21	8.2	Less tender
8	C	Low	Low	1.20	8.2	Less tender

*Each letter signifies a similar gradation.
**Maximum allowable = 1.20.

significantly different properties. It was not possible to duplicate the gradation of the original job mix; but mix 5 was the closest to it. Mixes 4 and 5 produced gyratory and Marshall volumetric properties that were the closest to being acceptable, but they displayed more tenderness during compaction.

As mentioned previously, it was desirable from a durability perspective to incorporate as much asphalt cement as possible and still maintain a rutting resistant surface. The GLWT tests were performed under test conditions that were believed to indicate performance, but the conditions have been changed to predict potential rutting problems more closely. Even though Georgia was using 8 mm as the maximum rut depth allowable under the test, it is possible that the limit should have been somewhat lower with the test conditions being used at that time. The first three mixes had the highest rut depths in the tests, but mix 5 produced the most pavement rutting.

Properties that should be associated with durability are asphalt film thickness and the fines-asphalt (F/A) ratio. The F/A ratio is the ratio of the amount of -75-micron aggregate to the amount of effective asphalt cement. The asphalt film thickness and fines-asphalt ratio calculated for the field mixes are listed in Table 6. A laboratory study indicated that asphalt aged excessively if the film thickness was less that 9 to 10 microns (Kandhal, 1996). A second study by Kandhal, Foo, and Mallick (1998) cited a recommended minimum film thickness of 8 μm. Mixes 2 through 6 should have the best resistance to aging because the film thickness was very close or within the 9 to 10 micron range. It is also interesting to note that mix 5, which had the highest pavement rutting, also had the lowest F/A ratio and highest asphalt film thickness. The low F/A ratio may have resulted in low stiffness, and when combined with thick asphalt films, it could have produced more rutting.

Field Tests

Density (voids) and rut depth field tests were performed. The results are shown in Table 7. Also, the change of rut depth with time is shown in Figure 2. Rutting has progressed approximately at a linear rate since construction with a total average rut depth of approximately 4 mm after slightly more than 2 years. Although the total rut depth is not excessive, it is hoped

Table 7. Pavement Voids and Rut Depth in Traffic Lanes

Mix No.	VTM of Road Cores, %		Rut Depth, mm		
	0 months	28 months	2 months	17 months	27 months
1	9.6	8.8	0	2.5	3.0
2	9.1	None in traffic lane			
3	9.9	8.4	-0.3	2.3	3.8
4	9.3	6.6	-0.3	2.5	4.6
5	9.4	7.6	0.3	4.1	4.6
6	8.0	8.2	0.3	2.0	3.3
7	10.3	9.8	0.5	2.0	3.8
8	9.0	6.8	1.0	2.5	4.1
Average	9.2	8.0	0.2	2.6	3.9

Note: Maximum allowable VTM at 0 months = 8.0%.

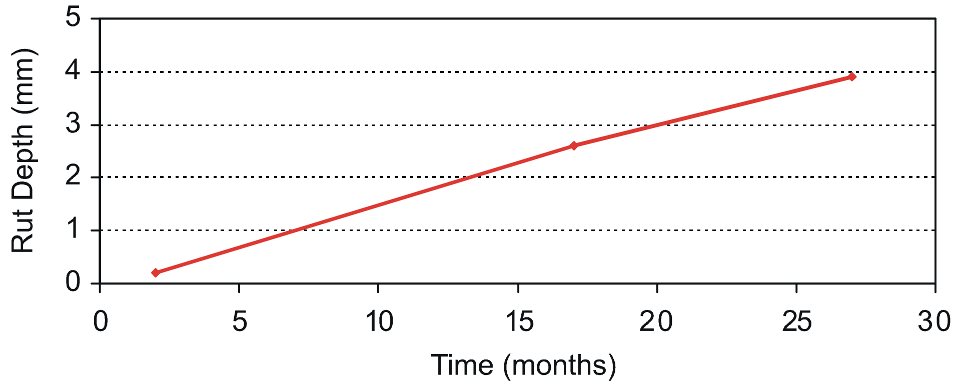

Figure 2. Average Rut Depth of Mixes in Outside Traffic Lanes

that the rate of development will decrease. Some of the rutting can be attributed to consolidation, but calculations for the 1.2 percent decrease in pavement voids can account for only about 0.5 mm of the rutting. Most of the rutting is probably the result of aggregate particles reorienting themselves to a more stable state. If adequate density could have been achieved during construction, the particles would have been in a more stable position and this initial rutting should have been less.

Mix 5 was placed in the eastbound and westbound traffic lanes (see Figure 1). The average rut depth of 3.3 mm and 5.8 mm in the eastbound and westbound traffic lanes, respectively, was significantly different when analyzed with a *t* test at 95 percent confidence limits. The test results (volumetrics and gradation) of samples from both sections revealed no obvious reasons for the difference. Also, the rut depth of other mixes placed in the westbound lane was obviously not larger than that of mixes in the eastbound lane, which does not indicate a difference in traffic loading in the two directions.

Only the pavement voids of mix 6 met the specification's allowable maximum of 8.0 percent. Failure to meet the density (voids) requirement was probably primarily caused by the tenderness problem. Falling head permeability tests were performed on five cores taken throughout the project after 27 months. Four of the five test results were less than 235 x 10^{-5} cm/sec, which is very good in comparison to the results of many tests performed on samples of other mixes taken over the last year. Although the permeability is slightly higher than a current Florida DOT allowable value of 125 x 10^{-5} cm/sec, it is felt that the mix is relatively impermeable, which should deter the entrance of surface water. If surface water was allowed to penetrate the pavement structure, it would deteriorate the asphalt and concrete supporting base material. Although it is probably too early to form substantive conclusions regarding durability, drive-through visual evaluations indicate no extraordinary distresses after 27 months.

CONCLUSIONS

Upon placement, the Superpave 19.0-mm mix was too dense and became tender during compaction. Laboratory tests on field samples also indicated that the mix was too dense, although no bleeding has been observed. Adjustments to the aggregate blend and asphalt content helped but did not eliminate the tenderness or laboratory density problems. Based upon our experience and that of other states, it should be recognized that the 19.0-mm mix may become tender during compaction.

Experience was also gained regarding placing a mix that was too dense according to lab tests. Perhaps coarse Superpave mixes are more tolerant to high density (low voids) in laboratory tests, which would allow more asphalt cement to be tolerated than is allowed by the suggested Superpave specifications. Recent unpublished work by Brian Prowell indicates that the specified number of gyrations may be too high, resulting in laboratory voids too low for properly proportioned mixes.

The average pavement rut depth for all of the sections was 3.9 mm, which is acceptable. However, it is hoped that the rate of rutting will decrease. If rutting is contained, the mix has an opportunity to be durable because of its relatively thick asphalt films. The performance should continue to be monitored periodically.

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REFERENCES

American Association of State Highway and Transportation Officials. (1990). *Report on the 1990 European Asphalt Study Tour*. Washington, DC: Author.

American Association of State Highway and Transportation Officials. (1998). *AASHTO Provisional Standards*. Washington, DC: Author.

American Society of Testing and Materials. (1996). *Annual Book of ASTM Standards, Soil and Rock (II)* (Volume 04.09). West Conshohocken, PA: Author.

Big Rock Contact Carries the Heavy Load. (1993, December). *Asphalt Contractor*, p. 20.

Deahl, C. (1999, February). Superpave Mixes Present Ongoing Compaction Challenges. *Asphalt Contractor*, p. 92.

Georgia Department of Transportation. (1997). *Method of Test for Determining Rutting Susceptibility Using the Asphalt Pavement Analyzer*, GDT-115. Forest Park: Author.

Kandhal, P. S. and Chakraborty, S. (1996, January). *Effect of Asphalt Film Thickness on Short and Long Term Aging of Asphalt Mixtures* (NCAT Report No. 96-1). Auburn University: National Center for Asphalt Technology.

Kandhal, P. S., Foo, K. Y., & Mallick, R. B. (1998). *Critical Review of Voids in Mineral Aggregate Requirements in Superpave* (TRR No. 1609). Washington, DC: Transportation Research Board.

Tahmoressi, M. *Summary of TxDOT Superpave Projects Constructed in 1997*. Internet site: www.utexas.edu/research/superpave

Virginia Department of Transportation. (1995, July). *Virginia Test Method for Determination of Asphalt Content From Asphalt Paving Mixtures by the Ignition Method*, VTM-102. Richmond: Author.