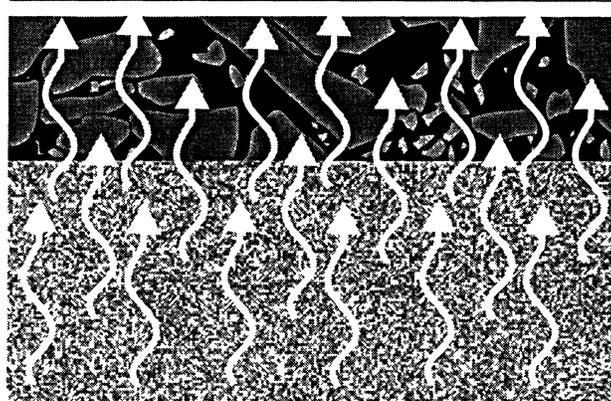


TECHNICAL ASSISTANCE
REPORT

**INVESTIGATION
OF PAVEMENT PERMEABILITY:
OLD BRIDGE ROAD**



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**Virginia Transportation Research Council
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ABSTRACT

Several instances of wet pavement and pavement icing on Old Bridge Road were reported to VDOT's Lake Ridge Area Headquarters when no new precipitation had fallen. The pavement structure appears to hold water. This water seeps to the surface at a number of isolated locations for several days after a rainfall event. The section was paved in 2000 with a Superpave 1/2 in (12.5 mm) nominal maximum aggregate size mix with a PG 70-22 binder (SM-12.5D).

Cores were taken from the right-hand travel lane in February 2001 in an attempt to identify the problem. The cores were tested for permeability, density, asphalt content, and gradation. The cores indicated that the pavement was permeable and generally failed in-place density requirements. Therefore, in April 2001, the center and left lanes were cored in a random manner to assess density and permeability. Field permeability tests were performed on site. The coring indicated that the pavement generally failed in-place density requirements. Where the pavement failed VDOT's minimum pavement density, the pavement was permeable. Areas that indicated segregation were more permeable. Although the hot-mix asphalt was a Superpave design, it was not a coarse-graded Superpave design and was not significantly different from the contractor's previous Marshall design. A strong relationship among pavement density, gradation, and permeability (in both the laboratory and the field) was developed from the data.

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INTRODUCTION

Several instances of wet pavement and pavement icing on Old Bridge Road were reported to VDOT's Lake Ridge Area Headquarters when no new precipitation had fallen. Old Bridge Road is located between Wood Hollow Drive and Occoquan Road in Prince William County. The pavement structure appears to hold water. The water seeps to the surface at a number of isolated locations for several days after a rainfall event. Several minor accidents and a fatality attributable to ice have occurred since the road was repaved in 2000. The section was repaved in 2000 with a Superpave ½ in (12.5 mm) nominal maximum aggregate size mix with a PG 70-22 binder (SM-12.5D). VDOT's Materials Division and the Virginia Transportation Research Council were asked to determine why this problem appeared after the roadway was resurfaced.

The section of Old Bridge Road is six lanes wide. The section is on a grade and has several curves. Near the curves where the worst icing has occurred, the combination of the down grade and curvature produces a cross-slope that forces water to drain across three lanes of traffic, as shown in Figure 1. Observations of the cores taken from this area indicate moisture damage in the base mix.



Figure 1. Drainage Across Old Bridge Road (After Coring)

On February 21, 2001, no wet spots were observed in the curves between Elysian Drive and Occoquan Road. Two wet areas were observed at the intersection of Wood Hollow Drive, as shown in Figure 2. Another wet area, shown in Figure 3, was observed along the edge of



Figure 2. Wet Spot on Old Bridge Road Near Wood Hollow Drive

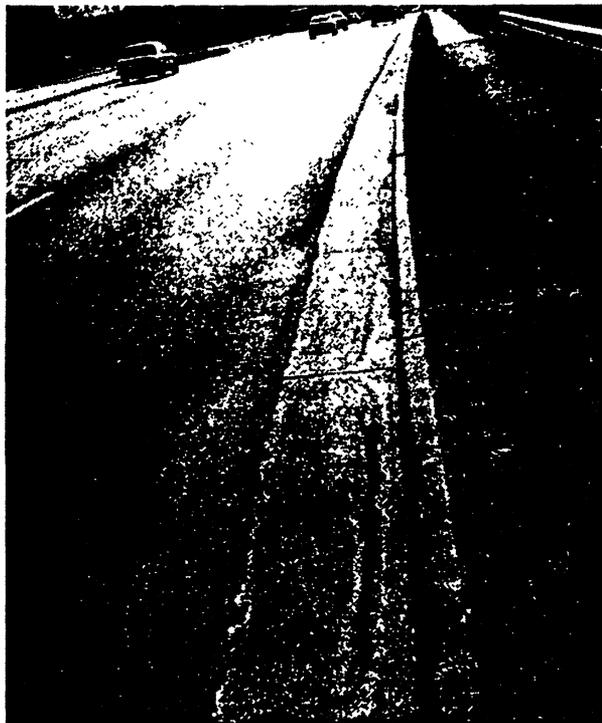


Figure 3. Wet Area at Edge of Pavement

the pavement just past Wood Hollow Drive (heading toward Occoquan Road). Based on data provided by the National Weather Service from their weather station at National Airport, no rain had fallen since February 17, 2001 (0.02 in, 0.5 mm): 0.58 in (14.7 mm) of rain fell on February 16, 2001.

Segregation at the centerline, typically caused by the auger gearbox on the paver, and segregation at the longitudinal joints (indicated by the darker areas) were observed throughout the project, as shown in Figure 4. End-of-load segregation and segregation at the transverse joints were also observed, as shown in Figure 5. Typically, segregated areas are more permeable than non-segregated areas.

The first set of cores from February 2001 indicated that portions of the right lane were permeable. There was concern that water could be entering the pavement from the two adjoining lanes. Therefore, the center and left-hand eastbound lanes were tested on April 25, 2001.

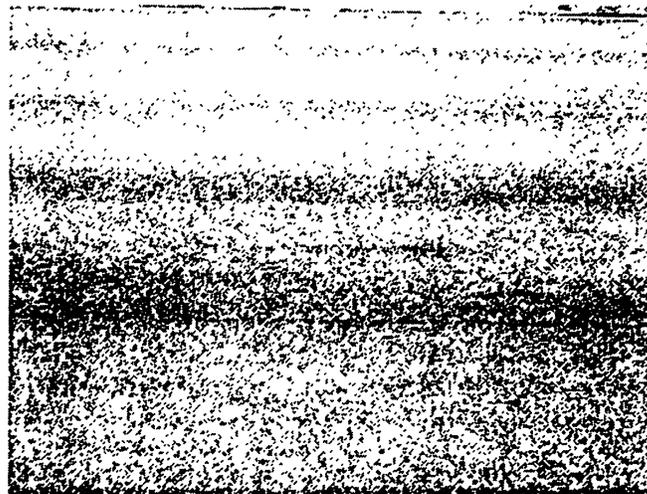


Figure 4. Centerline and Longitudinal Joint Segregation



Figure 5. Segregation at Transverse Cold Joint

PURPOSE AND SCOPE

The purpose of this investigation was to determine, if possible, where the water creating the wet spots was entering the pavement and if the pavement complied with the contract specifications and to recommend possible treatment strategies if applicable. The eastbound right-hand travel lane was sampled from Wood Hollow Drive to Occoquan Road. The eastbound center and left-hand lanes were sampled from Rolling Brook Road to Occoquan Road.

EXPERIMENTAL APPROACH

Samples were taken for analysis on February 21 and April 25, 2001. In February, 10 cores 6 in (150 mm) diameter were taken in the right-hand lane between Wood Hollow Drive and Occoquan Drive. An effort was made to take cores where the pavement appeared wet, where icing was reported to have occurred, in segregated areas, and in areas that appeared normal. The February core locations and descriptions are shown in Table 1. This initial coring effort was aimed at identifying the cause of the problem, not assessing whether the pavement complied with specifications.

Table 1. February Core Locations and Descriptions

Core No.	Location	Description
1	RWP just before intersection of Wood Hollow Drive	Dry, non-segregated
2	RWP at intersection of Wood Hollow Drive	Wet, non-segregated, grade flattens out
3	RWP at intersection of Wood Hollow Drive	Wet, transverse cold-joint segregation
4	RWP, 50 feet past Wood Hollow Drive, 15 feet before Core 5	Dry, non-segregated
5	Right edge of pavement	Wet, evidence of salt in curb pan
6	CWP at Core 5	Dry, gear-box segregation
7	CWP just past intersection with Elysian Drive	Dry, segregated
8	CWP 40 feet prior to driveway at 1635 Old Bridge Road	Dry, segregated, atypical gravel base mix
9	CWP in curve at gravel driveway near 1615 Old Bridge Road, reported as icy spot	Dry, coarse mix
10	RWP at location of Core 9	Dry, coarse mix

Note: RWP = right wheel path; CWP = center wheel path. Locations are referenced heading eastbound from Wood Hollow Drive to Occoquan Drive.

In April, coring was completed following the guidelines in VDOT's Materials Division Memorandum 232-01 for Quality Assurance of Asphalt Density. The instructions require coring locations to be randomly determined at a rate of 5 cores per 5,000 lane feet (1524 m). Nine cores were taken using the random sampling pattern. Additional cores were identified but fell outside the traffic control. These random cores could be used to assess the pavement's compliance with the specifications. An additional 5 cores were taken from specific locations (that appeared to have problems) and were designated with an *E*. The April core locations and descriptions are shown in Table 2.

Table 2. April Core Locations and Descriptions

Core No.	Distance from Rolling Brook Rd (ft)	Lane	Distance from Right Edge of Lane (ft)	Description
2-1	397	Center	10	LWP, minor segregation
2-2	620	Center	5	Edge of gear box streak, coarse
2-3	1217	Center	6	Gear box streak, medium segregation
2-4	1909	Center	8	LWP, very minor segregation
2-5	2226	Center	6	BWP, very minor segregation, adjacent to most prominent icing location
2-7	271	Left	2	Minor joint segregation
2-8	780	Left	1	Minor joint segregation, stripping in underlying layer
2-9	1093	Left	8	Edge of gear box streak, minor segregation
2-10	1246	Left	10	No segregation
E-2A	1052	Center	5.8	BWP, minor segregation
E-2B	1052	Center	2.6	RWP, no segregation
E-2C	1052	Center	8.9	LWP edge of longitudinal joint segregation
E-2D	1052	Left	5.4	BWP, minor segregation
E-4	2234	Left	3	No segregation

Note: LWP = left wheel path; RWP = right wheel path; BWP = between wheel paths. Wood Hollow Drive, Elysian Drive, the driveway at 1635, and the site of the accident are 933, 1,601, 2,122 and 2,309 feet from Rolling Brook Drive, respectively. 1 ft = 0.3048 m.

The density of the cores was determined in accordance with AASHTO T-166, Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens (AASHTO 2000). Since this method indicates higher than actual densities for permeable cores, the density was also determined with the Corelok procedure. The Corelok device vacuum seals a core in a plastic bag prior to determining its density by displacement (Buchanan 2000). Four of the 14 cores were randomly selected for determination of theoretical maximum specific gravity (Rice) in accordance with AASHTO T-209 (AASHTO 2000).

The laboratory permeability of the cores was determined in accordance with a modified version of the ASTM falling head procedure (ASTM PS 129-01). The Florida Department of Transportation (DOT) specifies that core permeability, tested by this procedure, may not exceed 0.00125 cm/s (3.54 ft/day). During the second sampling period, a field permeability test was performed using a device developed by the National Center for Asphalt Technology (NCAT) (Cooley 1998).

After permeability testing, the cores were dried to a constant mass. The core was heated, and the cut edges were carefully removed. The sample was broken up, and the asphalt content (AC%) was determined by the ignition method, VTM-102. The contractor's ignition furnace correction factor for aggregate loss, 0.20%, was used in the calculations. Gradation analysis on the recovered aggregate was performed in accordance with AASHTO T-30 (AASHTO 2000).

RESULTS AND DISCUSSION

Density and Permeability

The average thickness of the cores from the right-hand lane ranged from 2.2 to 2.7 in (55.9 to 68.6 mm). The average thickness of the cores from the center lane ranged from 1.9 to 2.3 in (48.3 to 58.4 mm). The average thickness of the cores from the left-hand lane ranged from 1.7 to 2.5 in (43.2 to 63.5 mm). The maximum specific gravity values associated with cores 2, 5, 6, and 9 were averaged to determine the density of the cores from the right lane. The average maximum specific gravity value for the right lane was 2.630. The maximum specific gravity values from cores 2-1, 2-4, and 2-8 were averaged to determine the density of the cores from the center and right lanes. The average maximum specific gravity for the center and left lanes was 2.638. The contractor's maximum specific gravity (Rice) value was 2.627. The maximum specific gravity value, obtained from analysis of the cores, for the center and left lanes was somewhat higher than the contractor's design value. This may indicate an asphalt content lower than the design value. Table 3 provides the AASHTO T-166 "SSD" air voids, Corelok air voids, and permeability values for the cores taken in February from the right-hand lane.

The minimum control strip density for an SM-12.5D mix is 92.2% density or 7.8% air voids. Only 1 of 10 cores from the right lane complied with this specification. The 98% tolerance on production density would allow density readings as low as 90.3% density or 9.7% air voids. Five of 10 right lane cores failed to meet this level. Eight of 10 right lane cores exceeded the Florida DOT's permeability specification. This indicates an opportunity for water to enter the pavement structure.

The incidence of wet spots and reported icing do not seem to correspond with the locations having the highest permeability. It is the author's belief that wet spots and icing result from a combination of pavement geometry and high air voids/permeability. For example, the wet spots in front of Wood Hollow Drive probably result from the break in grade at that location. Water traveling through the pavement structure is forced to the surface by the change in grade. In an effort to determine if the adjoining lanes were allowing water to enter the pavement, additional cores were taken on April 25, 2001, from the center and left lanes.

Table 3. Lab Permeability Results and Air Void Contents: Right-Hand Lane

Core No.	SSD Air Voids (%)	Corelok Air Voids (%)	Permeability (cm/s x 10 ⁻⁵)
1	7.0	7.8	20.1
2	7.9	NA	67.4
3	9.0	NA	183
4	9.9	11.1	268
5	8.8	9.9	173
6	12.3	14.4	1440
7	14.5	16.6	2760
8	11.7	13.0	704
9	11.5	12.4	809
10	8.6	9.7	270
Average	10.1	11.9	669

Note: 1 cm/s = 2835 ft/day.

The SSD air voids, Corelok air voids, NCAT field permeabilities, and lab permeabilities for the random and extra cores from the center and left lanes are shown in Table 4. Based on the SSD air voids, the results indicate that the average density of the random cores from the center and left lanes failed the minimum control strip density (less than 7.8% air voids) and the 98% production tolerance (less than 9.7% air voids). Both lanes still fail if the contractor's design maximum specific gravity value of 2.627 is used to calculate air voids. Only 6 of 14 cores passed the production tolerance.

An allowable specification for field permeability has not been established. However, data indicate that it produces results similar to those for the lab permeability test. The field permeability test is probably more indicative of water movement through a pavement since it allows lateral flow. Based on the Florida DOT's specification for the laboratory permeability test of a maximum permeability of 125×10^{-5} cm/sec (3.54 ft/day), four of five center lane cores and one of four left lane cores did not comply with the field permeability specification. All center lane cores and two of four left lane cores did not comply with the Florida DOT's criterion for laboratory permeability. Based on a comparison of the observed segregation for the extra cores, segregation of the mix corresponds with an increase in air voids and failing permeability. Figure 6 shows the relationship between air voids and both field and laboratory permeability. Based on the figure, the mix would need to be compacted below 9.5% air voids to meet Florida's specification. Based on the field permeability results, air voids levels less than 10.5% may be acceptable. Three additional field permeability readings, E1-A, E1-B, and E1-C, were taken from a segregated area slightly uphill from Wood Hollow Drive in the center lane. E1-A, E1-B, and E1-C had field permeability readings of 805 , 1576 , and 475×10^{-5} cm/s (22.8, 44.7 and 13.5 ft/day), respectively. All three of these areas would fail Florida DOT's specification of 125×10^{-5} cm/s (3.54 ft/day).

Table 4. Center and Left Lane Air Void Contents and Permeability Results

Core No.	SSD Air Voids, %	Corelok Air Voids, %	Field Permeability, cm/s x 10 ⁻⁵	Lab Permeability, cm/s x 10 ⁻⁵
<i>Center Lane Random Cores</i>				
2-1	10.0	11.1	195	271
2-2	10.5	11.6	97	162
2-3	17.3	20.7	Too permeable	5550
2-4	9.3	NA	143	172
2-5	11.7	13.0	391	554
Average	11.8	14.1	207	1342
<i>Left Lane Random Cores</i>				
2-7	10.6	11.6	93	201
2-8	9.5	10.5	67	106
2-9	10.9	12.7	394	464
2-10	9.6	11.2	34	98
Average	10.2	11.5	147	217
<i>Extra Cores</i>				
E2-A	13.4	14.7	545	584
E2-B	9.4	10.5	33	118
E2-C	9.3	10.4	171	168
E2-D	13.5	15.2	842	880
E4	8.2	8.9	17	29
Average	10.8	11.9	322	356

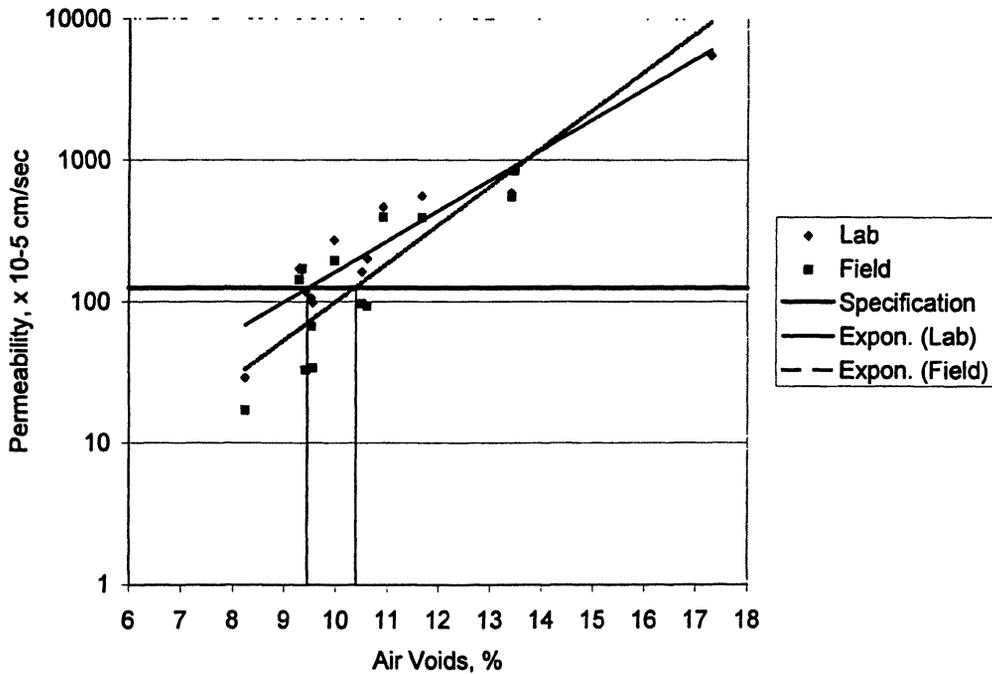


Figure 6. Permeability Versus Air Voids. Note: 1 cm/s = 2835 ft/day.

Asphalt Content and Gradation

Asphalt content and gradation analyses were performed to assess conformance with the job-mix formula and look at the degree of segregation. The results for the cores from the right-hand lane are shown in Table 5. They indicate the mix was finer than the job-mix formula. This would tend to reduce permeability. The only exception is Core 7, which appears from visual

Table 5. Right Lane Asphalt Contents and Gradations

Core No.	1	3	4	7	8	10	Job-Mix Formula
Sieve Size	Percent Passing						
19.0 mm, 3/4 in	100.0	100.0	100.0	100.0	100.0	100.0	100
12.5 mm, 1/2 in	99.1	99.2	99.4	99.6	99.0	98.5	96
9.5 mm, 3/8 i.	90.9	88.0	89.6	88.1	89.2	88.1	86.3
4.75 mm, No. 4	62.5	57.3	61.1	50.0	57.5	55.3	
2.36 mm, No. 8	44.5	39.9	43.9	34.6	40.5	38.7	36.1
1.18 mm, No. 16	32.6	29.3	32.8	26.5	29.8	28.8	
600 microns, No. 30	23.8	21.4	24.0	20.2	21.9	21.5	
300 microns, No. 50	15.7	14.1	15.7	13.6	14.4	14.4	
150 microns, No. 100	10.3	9.4	10.4	9.0	9.5	9.6	
75 microns, No. 200	6.68	6.22	6.81	5.73	6.27	6.33	5.0
AC%	5.63	5.59	5.73	NA	5.40	5.36	5.0

observations to be segregated but is actually closest to the job-mix formula. The results for Core 7 confirm the visual observation in Table 1. The Georgia DOT specifies removal and replacement for cores taken from segregated areas that are more than 10% different from the job-mix formula on a given sieve. None of the cores exceeded this level. The asphalt contents and gradations of the center lane cores are shown in Table 6. Again, the center lane gradations appear to be finer than the job-mix formula, particularly on the No. 200 sieve. The average asphalt content is also higher than design. Both deviations should have improved density and reduced permeability. Interestingly, Core 2-2, which visually appeared coarse on the surface, produced the finest gradation. The asphalt contents and gradations for the left lane are shown in Table 7.

Table 6. Center Lane Asphalt Contents and Gradations

Core No.	2-1	2-2	2-3	2-4	2-5	Average	Job-Mix Formula
Sieve Size	Percent Passing						
19.0 mm, 3/4 in	100.0	100.0	100.0	100.0	100.0	100.0	100
12.5 mm, 1/2 in	99.3	98.7	98.5	98.9	98.6	98.8	96
9.5 mm, 3/8 in	86.3	88.8	86.2	86.8	87.0	87.0	86.3
4.75 mm, No. 4	53.2	61.6	53.0	53.5	55.1	55.3	
2.36 mm, No. 8	37.8	44.6	37.6	37.2	38.2	39.1	36.1
1.18 mm, No. 16	28.4	32.7	28.2	27.9	28.3	29.1	
600 microns, No. 30	21.5	23.7	21.4	21.0	20.9	21.7	
300 microns, No. 50	14.8	16.2	14.6	14.3	14.0	14.8	
150 microns, No. 100	9.9	10.8	9.7	9.7	9.3	9.9	
75 microns, No. 200	6.4	7.1	6.3	6.5	6.1	6.5	5.0
AC%	5.16	5.60	5.21	5.19	5.26	5.28	5.0

Table 7. Left Lane Asphalt Contents and Gradations

Core No.	2-7	2-8	2-9	2-10	Average	Job-Mix Formula
Sieve Size	Percent Passing					
19.0 mm, 3/4 in	100.0	100.0	100.0	100.0	100.0	100
12.5 mm, 1/2 in	98.7	99.3	96.5	98.9	98.3	96
9.5 mm, 3/8 in	87.8	91.4	83.1	86.5	87.2	86.3
4.75 mm, No. 4	57.8	61.2	49.2	57.0	56.3	
2.36 mm, No. 8	45.3	48.0	34.9	44.8	43.3	36.1
1.18 mm, No. 16	34.4	36.1	26.4	34.3	32.8	
600 microns, No. 30	25.9	26.8	19.6	25.8	24.5	
300 microns, No. 50	17.4	18.0	13.3	17.3	16.5	
150 microns, No. 100	11.2	11.7	8.8	11.1	10.7	
75 microns, No. 200	6.8	7.2	5.8	6.7	6.6	5.0
AC%	5.57	5.72	4.98	5.44	5.43	5.0

With the exception of Core 2-9, all of the left lane cores were almost 10% finer on the No. 8 (2.36 mm) sieve than the job-mix formula. This is reflected in the permeability results for these three cores. The measured densities are also better. The asphalt contents and gradations for the extra cores are shown in Table 8. Core E-2C evidenced segregation along the longitudinal joint. Core E-2D showed minor segregation. With the exception of these two cores, the cores were almost 10% finer on the No. 8 (2.36 mm) sieve than the job-mix formula. The average asphalt content was 0.6% higher than design.

Table 8. Extra Cores Asphalt Contents and Gradations

Core Number	E-2A	E-2B	E-2C	E-2D	E-4	Average	Job-Mix Formula
Sieve Size	Percent Passing						
19.0 mm, 3/4 in	100.0	100.0	100.0	100.0	100.0	100.0	100
12.5 mm, 1/2 in	97.0	98.9	97.2	97.7	98.5	97.9	96
9.5 mm, 3/8 in	84.7	90.2	86.7	87.9	91.9	88.3	86.3
4.75 mm, No. 4	57.3	60.8	54.0	58.1	64.3	58.9	
2.36 mm, No. 8	46.2	48.7	38.5	41.0	51.2	45.1	36.1
1.18 mm, No. 16	35.0	37.0	28.4	29.9	38.2	33.7	
600 microns, No. 30	25.8	26.9	20.6	21.5	28.2	24.6	
300 microns, No. 50	17.4	18.0	13.9	14.3	18.6	16.5	
150 microns, No. 100	11.3	11.5	9.2	9.4	11.9	10.7	
75 microns, No. 200	7.0	7.1	6.0	6.2	7.2	6.7	5.0
AC%	5.48	5.74	5.45	5.56	5.91	5.63	5.0

Overall, it appears that the contractor produced a finer mix at a higher asphalt content than the job-mix formula. The gradation and binder content of this mix closely resemble the contractor's SM-2 Marshall design. SM-2 gradations are on the dense to fine side of the Superpave gradation bands. This should have made it easier to achieve density. Further, the finer mix tends to be less permeable at the same void level than a coarser mix. Even though a Blaw-Knox transfer device was used to place this mix, there was visual segregation evident throughout the section. Blaw-Knox transfer devices do not provide remixing except in the paver hopper. The visual observations of segregation do not always correspond to a coarse gradation with low asphalt content.

A multiple regression was performed using the natural log of air voids and the percent passing the No. 8 sieve (2.36 mm) as predictor variables for both the natural log of laboratory and field permeability. An R^2 of 0.97 with a standard error of 0.164 was determined for the laboratory permeability measurements. An R^2 of 0.94 with a standard error of 0.321 was determined for the field permeability measurements. Figure 7 shows the relationship between air voids and laboratory permeability for various percentages of material passing the No. 8 (2.36 mm) sieve. Figure 8 shows the relationship between air voids and field permeability for various percentages of material passing the No. 8 (2.36 mm) sieve. Both figures indicate how segregation can dramatically increase the permeability of a pavement at a given void level. In

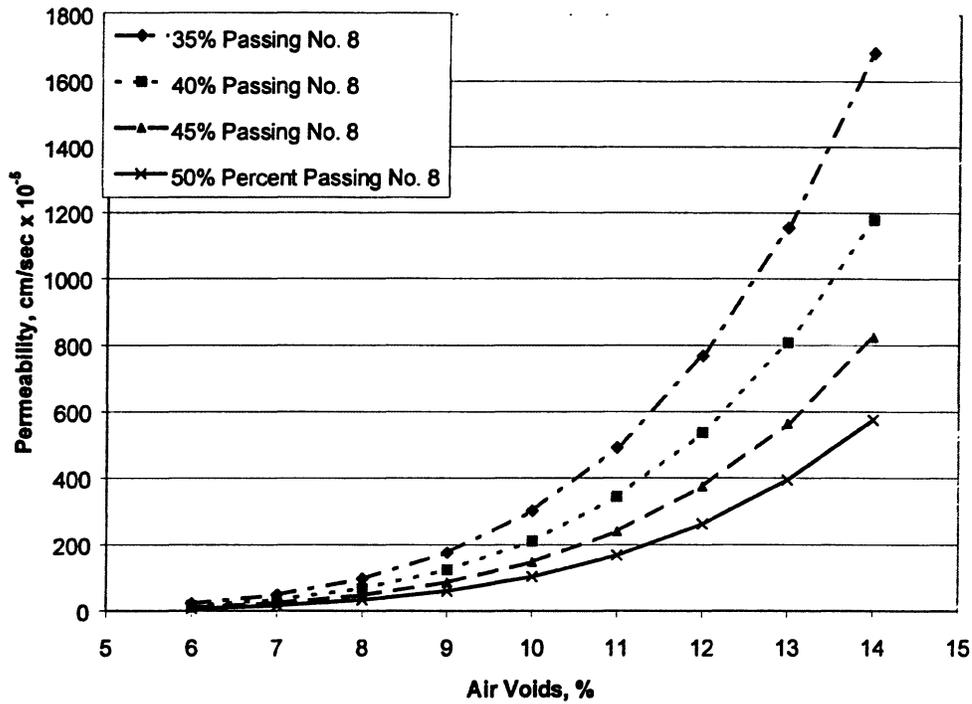


Figure 7. Laboratory Permeability as Function of Air Voids and Percent Passing No. 8 Sieve. Note: 1 cm/s = 3.54 ft/day.

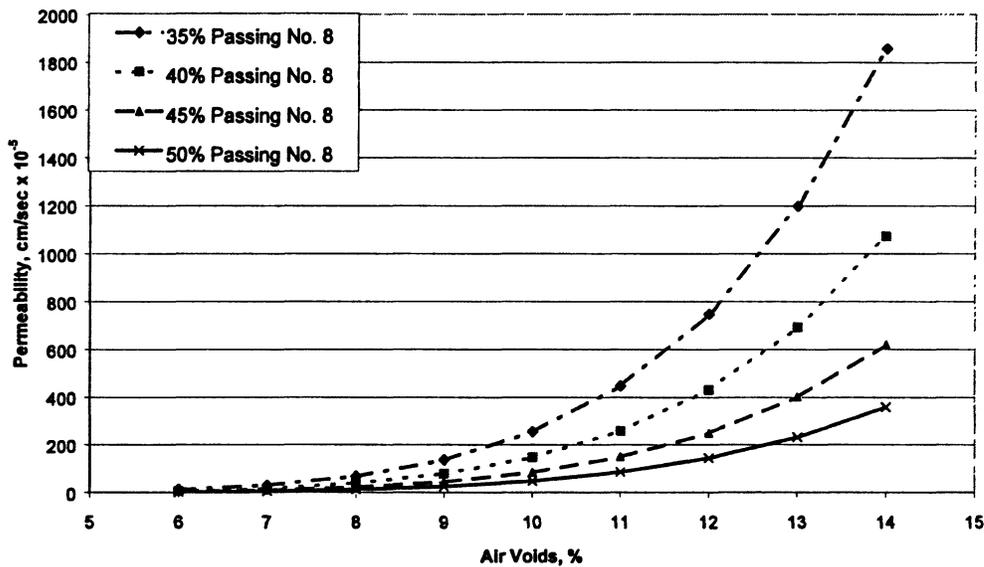


Figure 8. Field Permeability as Function of Air Voids and Percent Passing No. 8 Sieve. Note: 1 cm/s = 3.54 ft/day.

this case, the job-mix formula was set at 36% passing the No. 8 sieve (2.36 mm). If the contractor had not generally produced a finer mix, the permeability of the pavement would have been much worse. The areas with percents passing the No. 8 (2.36 mm) sieve close to those of the job-mix formula are actually considered segregated compared to the average production gradations. The figures also confirm previous research that indicates coarser mixes are more permeable at the same void level. It should be noted that the effect of the No. 8 (2.36 mm) sieve is minimal in the range of passing densities (less than 8% air voids).

Moisture Damage

Moisture damage, characterized by stripping of the coarse aggregate, was observed in the majority of the base mix cores taken between Elysian Drive and Occoquan Road. An example is shown in Figure 9. Moisture damage occurs when the asphalt separates from the aggregate in the presence of moisture. Typically, moisture damage takes a number of years to occur. This may indicate that water has been trapped in the pavement structure for some time.

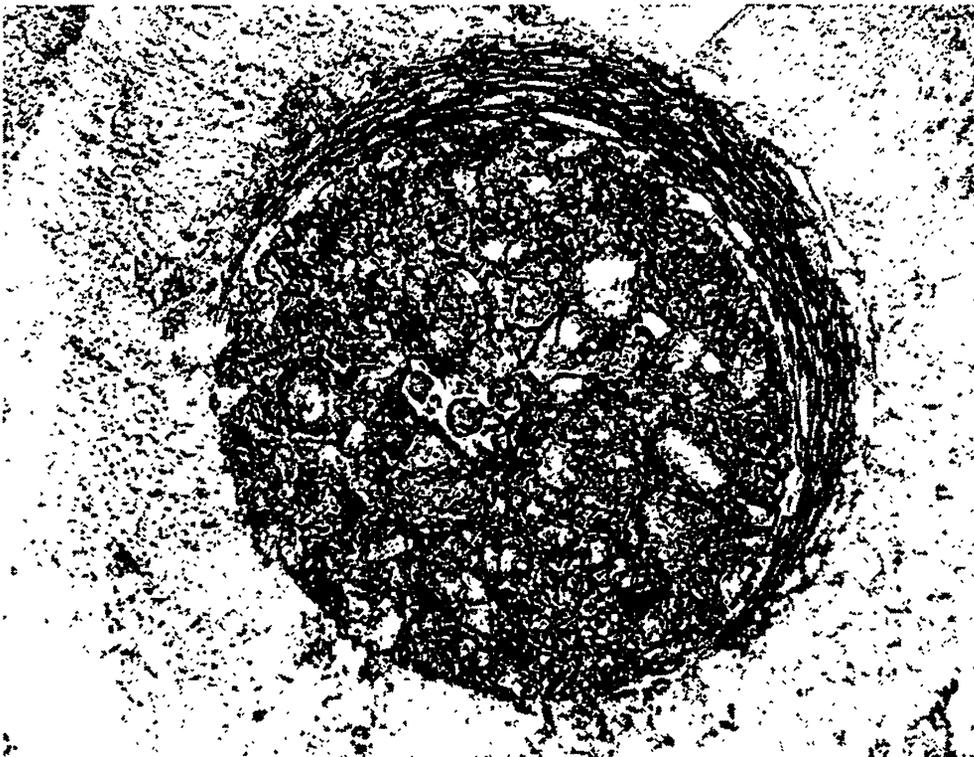


Figure 9. Typical Moisture Damage in Underlying Layer (Core 2-8)

SUMMARY

The icing conditions appear to be caused by low density and high permeability combined with drainage effects from the roadway geometry. The high permeability allows water to enter

the pavement structure. The poor roadway density is aggravated by the mat segregation. The use of a Blaw-Knox transfer device did not appear to alleviate segregation on this project. In some of the curves, the roadway geometry causes water to drain across the lanes instead of toward the gutter. In addition, in some locations, water in the pavement structure appears to be moving longitudinally downgrade (the path of least resistance) rather than toward any edge drains. The study did not investigate subsurface drainage. However, since this condition was not observed prior to the placement of the Superpave overlay, it is felt that the permeability of the overlay significantly contributes to the problem. Subsurface drainage problems may also contribute.

The design gradation is dense graded and passes through the Superpave restricted zone. Production gradations were typically finer than the design gradation. Finer gradations tend to be less permeable at a given air void level. The production asphalt content was higher than design. This would tend to improve density, but adequate density was still not achieved.

CONCLUSIONS

The following applies to the eastbound section of pavement between Wood Hollow Drive and Occoquan Road:

- The section fails pavement density.
- The contractor did not produce a coarse Superpave mix.
- The production gradations are generally finer than those of the job-mix formula.
- The production asphalt content is generally higher than the design asphalt content.
- In spite of the fine gradation and higher than design asphalt content, the pavement is permeable.
- There is a strong relationship among pavement density, gradation, and permeability.
- There is evidence of water in the asphalt layers beneath the surface mix (SM-12.5D).

RECOMMENDATIONS

To reduce water infiltration and the resulting weeping of water to the pavement surface, an impermeable pavement surface is required. However, moisture damage was present in the underlying layers throughout the section. Therefore, the treatment strategy needs to consider the potential for trapping water in the pavement structure. Doing so may increase the degree of moisture damage and lead to a pavement failure.

In some instances, a surface seal such as slurry seal or microsurfacing would be appropriate for preventing water from entering the pavement structure. However, judging from the moisture damage in the base mix near Elysian Drive, there is a significant amount of water in the pavement structure. Therefore, a surface seal may trap the moisture in the pavement, accelerating moisture damage (Aschenbrener et al., 1995). It is possible that a slurry seal or microsurfacing could be successfully used if it was placed after a lengthy period of hot, dry weather.

Another alternative would be to overlay the entire section with a fine-graded, small nominal aggregate size mix such as SM-9.0D. Since SM-9.0D can be placed at a lift thickness of 1 inch, milling the pavement would not be necessary. SM-9.0D would provide an impermeable surface but still allow the pavement to “breathe.” However, there is concern about the skid resistance of SM-9.0 mixes, particularly considering the geometry of this section.

The final alternative is to mill and replace the surface mix (SM-12.5D) for the affected area. Wet areas should be delineated a few days after a rainfall. A preliminary estimate would be to mill the center and right-hand lanes from Wood Hollow Drive to a point approximately 300 feet before the intersection of Occoquan Road. An SM-9.5D would be recommended for this application. The placement should be closely monitored to ensure that density is achieved. Based on the potential for trapping water and accelerating moisture damage, mill and replacement is probably the preferred treatment strategy.

Finally, VDOT should work to improve their density specifications and incorporate methods for measuring segregation.

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