Standard Title Page - Report on State Project

<table>
<tr><td>Report No.

VTRC 06-R9</td><td>Report Date

October 2005</td><td>No. Pages

30</td><td>Type Report:

Final</td><td>Project No.
76634
Contract No.</td></tr>
<tr><td colspan="4">Title:
Forensic Investigation of Concrete Pavement: U.S. 460, Appomattox Bypass</td><td rowspan="3">Key Words:
Concrete pavement, Appomattox Bypass, pavement failure, drainage</td></tr>
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Abstract

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A forensic investigation was conducted to identify the causes of failure and determine reasonable remedial measures. The investigation included a visual condition survey, non-destructive testing, coring through pavement layers, and slab removal.

Drainage problems, construction issues, and other pavement component issues were found with regard to this rigid pavement and led to the failure. Water trapped within the open-graded drainage layer aggravated the failure rate of the drainage layer abrading the underlying soil cement layer.

As a remedial measure, the researchers suggested that the drainage function be restored and the damaged pavement layers be repaired.

FINAL REPORT

FORENSIC INVESTIGATION OF CONCRETE PAVEMENT: U.S. 460, APPOMATTOX BYPASS

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Virginia Transportation Research Council
(A Cooperative Organization Sponsored Jointly by the
Virginia Department of Transportation and
the University of Virginia)

Charlottesville, Virginia

October 2005
VTRC 06-R9

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INTRODUCTION

The Appomattox Bypass was built during the 1993 and 1994 construction season along U.S. 460 in Appomattox County, Virginia. The bypass is a four-lane divided primary highway located on the north side of the town of Appomattox. An approximately 2.8-mile-long section of the bypass has failed prematurely at several locations and was the subject of the forensic investigation reported herein. The pavement structure consisted of 9 inches of doweled jointed plain concrete pavement (JPCP) with a 4-inch cement-stabilized open-graded drainage layer (OGDL) over 6 inches of soil cement.

This pavement was designed for a 30-year life but has shown signs of fatigue for the last few years; and as early as 1998. Although the pavement is only 11 years old, mid-slab cracks and settlement have propagated through both the eastbound and westbound travel lanes. The traffic count as of 2003 varied from 11,000 average daily traffic (ADT) to 15,000 ADT with 14% and 6% truck traffic, respectively.

Several investigations of this pavement have been conducted, but they did not uncover a definitive failure mechanism to explain the premature failure. The Virginia Department of Transportation's (VDOT) Appomattox Residency initiated a contract to replace 137 slabs, which provided a golden opportunity to do an in-depth forensic investigation of the pavement.

PURPOSE AND SCOPE

The purpose of this forensic investigation was to review previously performed investigations of the pavement, identify the failure mechanism, find the possible causes of the premature failure, and recommend reasonable remedial measures.

METHODS

A typical section with both damaged and undamaged slabs was selected for evaluation. The forensic investigation was focused on the section, and the researchers assumed that the reason for its premature failure would be similar to those for other sections of the bypass. In this case, the selected section was approximately located from mile post (MP) 8.10 to MP 8.35 on the eastbound lane.

The historical records, including a preliminary engineering report, design details, and construction history, were reviewed for possible clues to failure. The pavement was cored at different locations within the selected section to ascertain the condition of the different layers of the pavement, including the subgrade. The cored samples were also tested in the laboratory to determine the quality of the material in the respective layers. The investigation was extended during the slab removal, which was a part of an ongoing rehabilitation effort by the residency. In order to assess the overall condition of the pavement, a visual condition survey along with two other non-destructive tests was performed. The falling weight deflectometer (FWD) test was performed to assess the load transfer efficiency, and a profiler was run to measure the ride quality of the entire section.

RESULTS

Records Review

A preliminary engineering and soil survey was conducted in 1974 for this project (Hayden, 1974). The material encountered was predominately red clay and tan silt. The in-situ soil was quite wet (above optimum) in some areas (for at least 50% of the subgrade) at the time of drilling. Therefore, a 2-foot undercut was recommended for several sections on the eastbound and westbound lanes. One of the sections for which an undercut was recommended was EBL station 19+00 to station 24+25, which coincides with the section selected for investigation (approximately station 21+00 to 30+00). The soil from this section was classified as A-7-5(15) (USCS classification MH, elastic silt) with a soaked California bearing ratio (CBR) of 9.0% and a swelling of 0.2%. Most other soil samples were classified as A-4 or A-5 with varying numbers of group index values (3 to 10). CBR values for these soils varied from 4.0% to 18.0% with a respective swelling of 4.35% to 0.77%. The undercut materials were recommended to be replaced with suitable materials having a minimum CBR of 9 and a swell no greater than 2.25%. Stabilization of the top 6 inches of subgrade with 10% cement by volume was also recommended.

Pavement Design

The pavement design on this project was based on the American Association of State Highway and Transportation Officials' (AASHTO) 1986 Design Method for Rigid Pavement (AASHTO, 1986). The following input parameters were used:

- Projected Equivalent Single-Axle Loads (ESAL) for 30 years: 8,000,000
- Reliability level (%): 95
- Overall standard deviation: 0.35
- 28-day mean modulus of rupture for portland cement concrete: 650.00 psi
- 28-day mean modulus of elasticity for portland cement concrete: 3,705,000 psi
- Load transfer coefficient, J factor: 3.20
- Modulus of subgrade reaction (K value): 193 psi/in
- Overall drainage coefficient (C_d): 1.20
- Initial serviceability: 4.5
- Terminal serviceability: 2.5.

The recommended pavement typical section was as follows:

1. 9.0 inches doweled JPCP slab, 15-foot spacing
2. 4.0 inches cement-stabilized OGDL
3. 6.0 inches cement-treated soil, using 10% hydraulic cement by volume
4. 9.0 to 6.0 inches variable depth jointed concrete un-doweled tied shoulder
5. 4.0 inches aggregate base material, Type I, Size 21A (shoulder)
6. pavement edge drain UD-4 in accordance with VDOT standard pavement edge drain (VDOT, 1991).

Based on the soil survey (Hayden, 1974), the subgrade soil was identified as micaceous silt, with an AASHTO classification of A-7-5, A-4, and A-5 and a CBR ranging between 4% and 8 %.

The drainage coefficient used in the design was 1.2 based on the inclusion of functional OGDL and pavement edge drain. This led to a relatively thinner slab thickness, compared with the worse case scenario where the drainage features failed to function properly and a lower drainage factor of 0.7 would have been applicable.

Previous Investigative Reports

Three investigations have been conducted to understand the premature failure mechanism of the Appomattox Bypass. They were conducted by the Virginia Transportation Research Council and VDOT's Materials Division.

In 1994, VTRC (Lane, 1994) examined two cores, taken for a depth check, to evaluate consolidation in concrete pavements. Lane stated that excessive entrapped void content was found in the top half of the cores, indicating a more general problem with consolidation on this project. The poor consolidation may have resulted from one or more factors: insufficient vibrator frequency, one or more malfunctioning vibrators, excessive speed of the paving machine, and stiff concrete.

In 1998, the Pavement Design and Evaluation Section of VDOT's Materials Division conducted load transfer (LT) and mid-slab testing on the same project (Elfino, 1998). All the joints, 933 joints in the westbound and 918 joints in the eastbound direction, were tested for LT. Mid-slab testing was done at a 50-foot spacing throughout the project in both directions. It was reported that 9% (88 joints) of the westbound joints had a load transfer efficiency (LTE) below 50%, considered poor. In the eastbound direction, 18% of the joints (161 joints) had a LTE below 50%. Mid-slab testing showed generally acceptable deflection under the plate (D1), but the eastbound direction was worse than the westbound. The testing was performed during March and April of 1998, where the temperature was 30 to 35 degrees Fahrenheit for the eastbound (joints are open) and 50 to 70 degrees Fahrenheit for the westbound (joints are relatively closed) direction. In addition, the edge drain was inspected by camera, which showed that the majority of the edge drain was clear of clogging or broken pipes.

The preliminary findings from another investigation (Clark, 2000) included low-severity mid-slab cracks along with settlement (as high as 0.5 inch to 1 inch) for several slabs. A good ride quality was reported for both eastbound and westbound lanes with an average International Roughness Index (IRI) value of less than 90 inches/mile, although the majority of the values were 50 to 70 inches/mile. It was also reported that, in general, the load transfer efficiency of the joints was good, with the average being above 80%. Only seven joints in the eastbound lane were below 50%. The deterioration of OGDL and soil cement was speculated to be the probable cause for premature failure. High traffic, slab curling, and poor joint seals were mentioned among other possible contributing factors. According to the 1999 data published by VDOT's Traffic Engineering Division, annual average daily traffic (AADT) was 11,000, with 8% truck traffic. Corner breaks were speculated to be due to the slab curling and loss of support. Further in-depth investigation was recommended to identify the actual causes of failure. This investigation tested only selected joints and mid-slab points in the travel lanes of both directions while the temperature was between 40 and 50 degrees Fahrenheit. One observation to note on the LTE data is that it included LTE greater than 100%, which is not possible and can lead to a higher LTE average. It was noted that the LTE exceeding 100% had a very high deflection up to 37 mils. This indicates very poor load transfer and curling up at these joints.

Existing Condition of Pavement

Field Distress Survey

A visual distress inspection was performed for the entire bypass through a windshield survey in December 2004 and was compared with the detailed condition survey conducted by the Appomattox Residency in July 2004. It was obvious from the field visit that travel lanes were the only distressed lanes throughout the bypass. The residency's condition survey is summarized in Tables 1 and 2 for the eastbound and westbound travel lanes, respectively. Overall, about 24% of the slabs in the eastbound lane showed distresses. On the other hand, about 12% of the slabs in the westbound lane showed distresses. The predominant types of distresses were mid-slab cracks and slab settlement. As an example, Figure 1 shows one of the settled slabs. During the field visit, a poor sealant condition was also observed. One of the damaged seals is shown in Figure 2. During the visit, it was also speculated that heavily loaded truck traffic might have contributed to these distresses. A traffic count provided by the Lynchburg District confirmed the presence of high truck traffic. Detailed traffic data are presented in Table 3. By comparing

Table 1. Results of Visual Distress Survey Conducted by Appomattox Residency (Eastbound Lane)

Mile Post (MP 7.92 as 0)		No. Damaged Slabs (of 35 slabs/ 0.1 mi)		% Slab Damaged	Remarks
From	To	Cracked Slab	Settled Slab		
0	0.1	2	3	1	
0.1	0.2	0	4	11	
0.2	0.3	8	13	37	
0.3	0.4	6	14	40	5 loose slabs
0.4	0.5	11	15	49	
0.5	0.6	7	19	54	7 loose slabs
0.6	0.7	7	11	51	5 loose slabs
0.7	0.8	0	1	3	
0.8	0.9	12	15	54	2 loose slabs
0.9	1.0	18	16	69	3 loose slabs
1.0	1.1	3	9	29	
1.1	1.2	0	2	6	
1.2	1.3	2	0	6	
1.3	1.4	1	3	9	
1.4	1.5	5	3	14	
1.5	1.6	2	12	37	
1.6	1.7	7	3	26	2 loose slabs, spalling
1.7	1.8	0	0	0	
1.8	1.9	0	5	14	at least ¼-inch settlement
1.9	2.0	0	0	0	
2.0	2.1	3	7	23	Some map cracking, high settlement, diagonal cracks
2.1	2.2	8	9	37	
2.2	2.3	8	17	63	
2.3	2.4	0	0	0	
2.4	2.5	0	0	0	Poor surface finish
2.5	2.6	4	0	11	
2.6	2.7	0	0	0	
Overall damage (average)				24	

Tables 1 and 2 it may be seen that more distresses were observed on the eastbound lane, and the traffic data in Table 3 also show a higher truck traffic on the eastbound lane. That the high truck traffic and travel lane was the only distressed lane indicates traffic-related distresses.

Functional Evaluation

VDOT's Asset Management Division also conducts yearly distress surveys, which included roughness/ profile measurements. The IRI values were obtained from the division for 2003 and 2005. Average IRI values are presented and plotted in Figure 3. Results of tests conducted in 2003 for the eastbound lane had an average IRI of 87, a standard deviation (SD) of 13, and a coefficient of variation (COV) of 15; in 2005, the average was 116, the SD was 34, and the COV was 29. This indicates progressive deterioration in the ride quality. Tests conducted in 2003 for the westbound travel lane revealed an average IRI value of 71, a SD of 15, and a COV of 21; in 2005, the average was 83, the SD was 30, and the COV was 36. This also is an indication of progressive deterioration in the ride quality. It is to be noted that the ride quality is

Table 2. Results of Visual Distress Survey Conducted by Appomattox Residency (Westbound Lane)

Mile Post (MP 10.64 as 0)		No. Damaged Slabs (of 35 slabs/ 0.1 mile)		% Slab Damaged	Remarks
From	To	Cracked Slab	Settled Slab		
0	0.1	1	0	3	
0.1	0.2	0	0	0	
0.2	0.3	3	0	9	longitudinal hairline cracks
0.3	0.4	0	0	0	
0.4	0.5	0	0	0	diagonal cracks
0.5	0.6	1	0	3	
0.6	0.7	0	0	0	
0.7	0.8	0	0	0	
0.8	0.9	0	0	0	
0.9	1.0	0	0	0	
1.0	1.1	0	1	3	map cracks (hairline)
1.1	1.2	0	4	11	map cracks (hairline)
1.2	1.3	0	0	0	
1.3	1.4	0	0	0	
1.4	1.5	0	0	0	
1.5	1.6	0	6	17	
1.6	1.7	0	0	0	
1.7	1.8	0	4	11	
1.8	1.9	0	1	3	
1.9	2.0	0	0	0	
2.0	2.1	0	1	3	
2.1	2.2	0	7	20	
2.2	2.3	2	25	71	
2.3	2.4	19	26	77	3 loose slabs and ½- to 1-inch settlement, spalling
2.4	2.5	1	8	23	
2.5	2.6	8	0	23	
2.6	2.7	3	12	34	
Overall damage (average)				12	

worse in the eastbound lane, which coincides with the structural condition being worse in the eastbound lane. Obviously, the residency also replaced more slabs in the eastbound lane. It is also noted that the area under investigation had a relatively high IRI because of faulting and mid-slab cracks.

Structural Evaluation

The structural condition and integrity of the existing concrete pavement was evaluated using a FWD. Data have been collected over the last few years for the Appomattox Bypass. Both mid-slab and joint load transfer tests were included into the testing program. The LTE is defined as the ratio between the deflection of the unloaded slab and the loaded slab times 100. The LTEs are presented in Figure 4 for the years 2001 and 2005. Several years of LTE statistics are presented in Table 4. In order to investigate the gradual progression of load transfer failure, data were grouped according to low (< 50%), medium (51%-75%), and high (> 75%) and presented in Figure 5. It is important to note that only 15% of the total joints were tested every year except for 1998, when all the joints were tested. The testing load was different

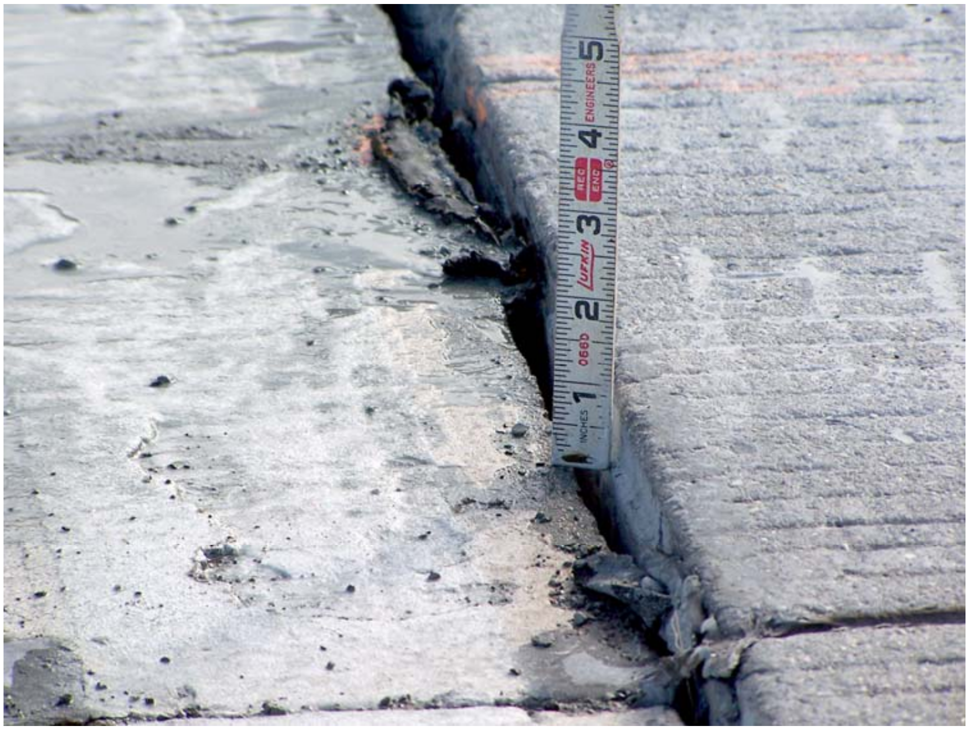

Figure 1. Differential Slab Movement at Longitudinal Joint

Figure 2. Broken Sealant at Transverse Joint

Table 3. Traffic Count According to 2003 Survey

Location/MP	ADT (vehicles/day)	% Truck (no. truck)	% Axle – Distribution (no. truck)				Directional Factor
			Bus	Two axles	3+ axles	Trailer	
West end MP 7.92	15,000	6 (900)	0	1 (150)	1 (150)	4 (600)	0.574
East end MP 10.64	11,000	14 (1540)	1 (110)	1 (110)	1 (110)	11 (1210)	0.503

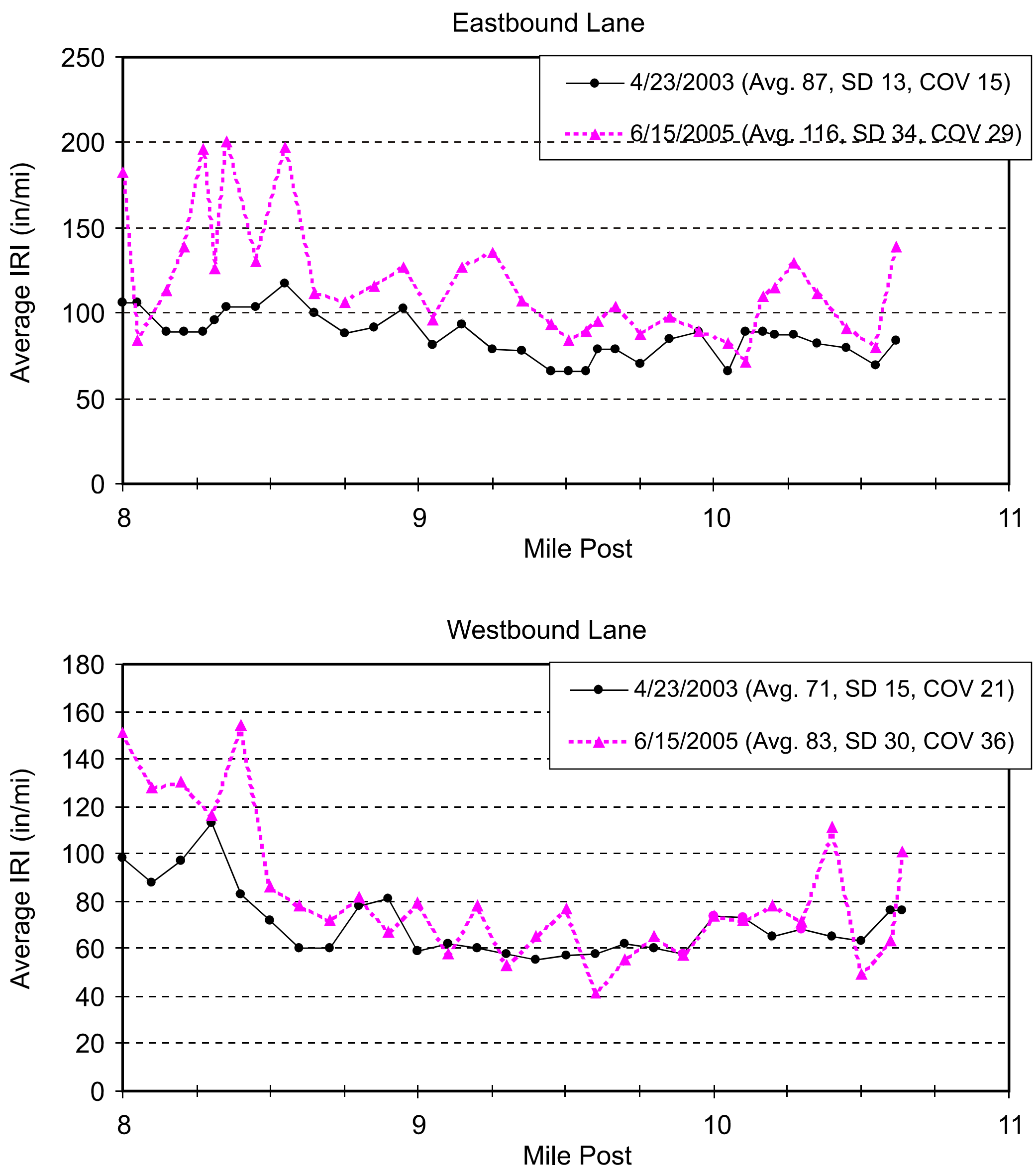

Figure 3. IRI Values from Profiler Testing: Eastbound and Westbound Travel Lanes

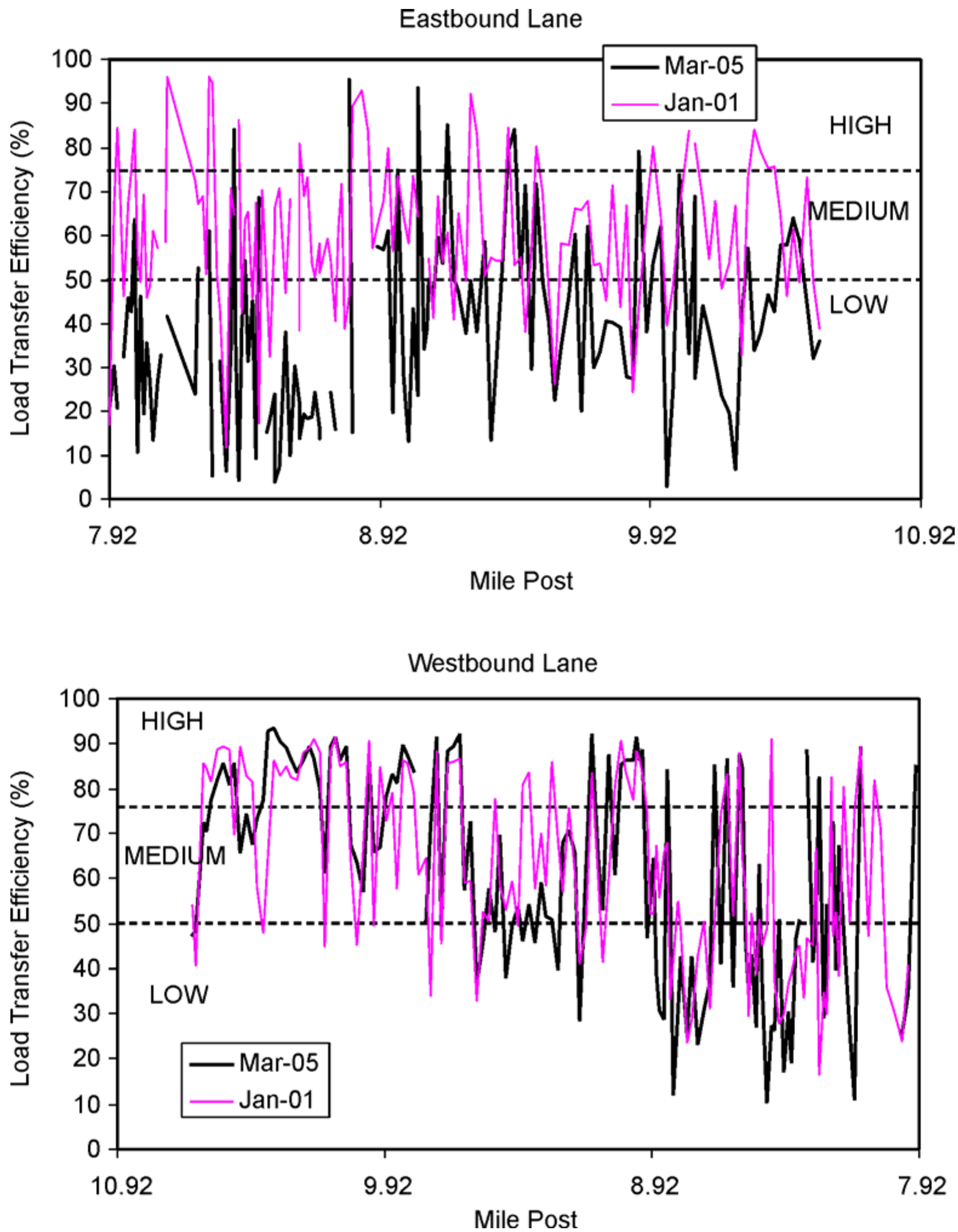

Figure 4. Load Transfer Efficiencies Measured with FWD test for 2001 and 2005

in 1998; it was 9,000 pounds instead of the 16,000 pounds used in all other years. The testing temperature was also varied, as shown in Table 4. The data from July 2003 were discarded because it was suspected that the joints were locked during the testing because of the relatively high temperature. It is obvious from Figure 5 that the eastbound lane is deteriorating at a faster rate than is the westbound lane.

Table 4. Average Load Transfer Efficiencies at Joints Using FWD Testing

Test Date	Load Transfer Efficiency (%)									
	Eastbound Lane					Westbound Lane				
	Distribution (%)	Average	SD	COV	Average Test Temperature ^{0}F	Distribution (%)	Average	SD	COV	Average Test Temperature ^{0}F
March-April 1998	Low: 18 Medium: 47 High: 35	79	61	77	60	Low: 9 Medium: 23 High: 68	68	50	73	32
January 01	Low: 26 Medium: 57 High: 17	60	16	27	47	Low: 32 Medium: 30 High: 38	63	20	32	58
July 03	NA	91	18	20	76	NA	96	4	4	78
January 04	Low: 54 Medium: 37 High: 9	48	19	40	47	Low: 34 Medium: 32 High: 34	61	21	34	52
March 05	Low: 73 Medium: 21 High: 6	38	21	53	51	Low: 36 Medium: 28 High: 36	62	23	37	51

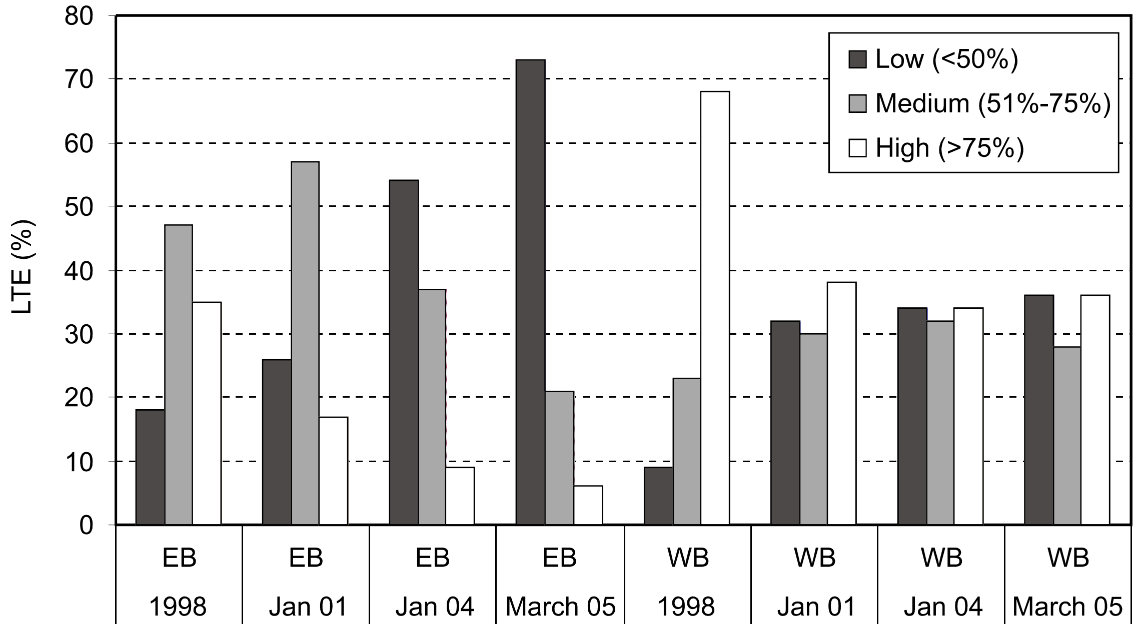

Figure 5. Progression of Load Transfer Efficiencies

Field Testing

The field condition of the pavement structure was evaluated through selective coring and slab replacement. During these processes, samples from different layers of pavement were also collected for laboratory evaluation/testing of the current condition of the respective materials. The investigation through coring and slab replacement was limited to the representative section of the roadway approximately between MP 8.10 and MP 8.35. This section has both damaged and undamaged slabs; therefore, a relative comparison would be possible.

Field Coring

Field cores were collected for the assessment of the in-situ condition of the failed pavement sections. Seven cores were taken from a section of the pavement as mentioned previously. These cores included soil cement as well as surface concrete and drainage layers. Shelby tube soil samples were also collected from the bottom of these cores for a laboratory investigation. Table 5 presents a brief description of the core location with the respective pavement distress condition and field observation.

Core 2 was through a mid-slab crack. The crack propagated through the drainage layer but did not damage the soil cement layer. Core 4 was on a longitudinal joint and through a tie rod. Because of the differential settlement between the travel and acceleration lane, the tie rod was found bent as shown in Figure 6. Core 5 was on a transverse joint through a dowel bar. The dowel bar was found to be wobbly and the hole around the dowel bar was elliptical, as shown in Figure 7, instead of circular, indicating poor load transfer.

A consistent wet condition was observed under the damaged slabs during coring. The water was infiltrating through the drainage layer from the surrounding pavement areas into the

Table 5. Field Observations and Core Locations

Core/ Borehole No.	Mile Post and Station	Location[a]	Pavement Condition	Drainage Layer Condition	Soil Cement Condition	Remarks
7	MP 8.15, Sta. 21+67	Passing lane across from core 1 (8.6 ft left from right slab edge)	No distress	Clean and intact	No visual distress	Cut section (10 ft).
1	MP 8.15, Sta. 21+64	2 ft from joint (7.4 ft right from left slab edge)	Mid-slab crack and 1-in edge settlement	Contaminated with red soil	No visual distress	Cut section (10 ft). Influx of water into bore hole through drainage layer.
2	MP 8.17, Sta. 22+46	Mid-slab crack (6.7 ft right from left slab edge)	Mid-slab crack and minor settlement	Partially disintegrated and contaminated with red soil	No visual distress	Cut section (9 ft). Influx of water into bore hole through drainage layer.
3	MP 8.22, Sta. 25+48	2 ft from joint (5.6 ft right from left slab edge)	Mid-slab crack and 0.5- to 1.5-in edge settlement	Contaminated with red soil	No visual distress	Fill section (8 ft). Wet subsurface condition.
4	MP 8.22, Sta. 25+41	Longitudinal joint (between travel and acceleration lane)	Mid-slab crack and 0.5- to 1.5-in edge settlement	Contaminated with red soil	No visual distress	Fill section (8 ft). Wet subsurface condition.
5	MP 8.28, Sta. 28+36	Transverse joint (10.7 ft right from left slab edge)	Mid-slab crack and 0.5- to 2.0-in edge settlement	Contaminated with red soil	No visual distress	Cut section (4 ft). Wet subsurface condition.
6	MP 8.30, Sta. 29+51	Mid-slab (9.1 ft right from left slab edge)	No distress	Clean and intact	No visual distress	Cut section (7 ft). Dry subsurface condition.

[a]All cores, except for core 7, were in eastbound travel lane; core 7 was in eastbound passing lane just across from core 1.

Figure 6. Core Through Faulted Longitudinal Joint and Bent Tie Rod

Figure 7. Core Through Faulted Transverse Joint and Wobbly Dowel Bar

bore hole during the coring operation, especially for the most seriously damaged slabs such as those with cores 1 and 2. Another consistent observation for damaged slabs was partially clogged drainage layers with red soil as shown in Figure 8. On the other hand, the drainage layers for the undamaged slabs (cores 6 and 7) looked clean and there were no signs of even any soil stain. This is a clear indication of the drainage problem associated with the observed distresses.

Figure 8. Clogged Drainage Layer

Subgrade Evaluation

The Standard Penetration Test (SPT) was conducted in core locations 1, 6, and 7. Table 6 summarizes the SPT data. Overall, the existing subgrade condition was poor, with uncorrected SPT values ranging from 2 to 9. A very low SPT count, such as 1 blow or push of the hammer, was observed between consecutive 12-inch (usual practice is 6-inch increment) increments of an SPT sample. This indicates the sensitive nature of in-situ soil.

Drainage Inspection

The video inspection of the edge drain and outlets by the Lynchburg District's Materials Section revealed no blockage for the edge drain around the selected investigational section. The travel distance for the camera was different for all eight inspected edge drain outlets. The inspection report is presented in Table 7. All eight outlets and pipes were found clear of any debris and blockage. On the contrary, the drainage layers underneath the pavement showed significant blockage near damaged slab areas. This is an indication of discontinuity in the drainage path.

Table 6. Standard Penetration Test Results for Subgrade

Borehole Location	Mile Post and Station (Offset)	Field Description of Soil	Sample Depth (ft)	Uncorrected Blows (N)
Core 1	MP 8.15, Sta. 21+64 @7.4 ft right	Red micaceous clayey silt w/manganese filled joints	1.8-2.3	2
			2.3-2.8	3
			2.8-3.3	4
			3.3-3.8	5
			3.8-4.3	Push
			4.3-4.8	2
			4.8-5.3	3
			5.3-5.8	4
			5.8-6.3	1
			6.3-6.8	2
			6.8-7.3	2
			7.3-7.8	3
Core 6	MP 8.30, Sta. 29+51@ 9.1ft right	Red micaceous clayey silt w/manganese filled joints	4.0-4.5	2
			4.5-5.0	4
			5.0-5.5	5
			5.5-6.0	7
Core 7	MP 8.15, Sta. 21+67 @8.6 ft left	Red micaceous clayey silt w/manganese filled joints	3.7-4.2	1
			4.2-4.7	2
			4.7-5.2	2
			5.2-5.7	2
			5.7-6.2	1
			6.2-6.7	2
			6.7-7.2	2
			7.2-7.7	3

Table 7. Underdrain Inspection Report

Outlet Location			Pipe Diameter (in)	Tested Lengths (ft)	Method	Inspection Satisfied (Y/N)	EW-12 Satisfied (Y/N)	Comments
MP and Station	Lane	Side						
MP 8.11 Sta. 9+69	East	Median	6	58	Camera	Y	Y	OK/Clear
MP 8.13 Sta. 0+69	East	Right	4	10	Camera	Y	Y	OK/Clear
MP 8.15 Sta. 1+48	East	Median	4	80	Camera	Y	Y	OK/Clear
MP 8.23 Sta. 5+77	East	Right	6	67	Camera	Y	Y	OK/Clear
MP 8.28 Sta. 8+26	East	Right	6	65	Camera	Y	Y	OK/Clear
MP 8.32 Sta. 0+72	East	Right	6	17	Camera	Y	Y	OK/Clear
MP 8.37 Sta. 33+15	East	Right	6	17	Camera	Y	Y	OK/Clear

EW-12 = Standard endwall for pipe underdrain (VDOT, 1991).

Groundwater Monitoring

Several observation wells were installed and monitored for a few weeks to establish a perched water table. Table 8 describes the location and water level at those wells. The water table was well below the pavement surface; at least 10 feet. Although there may be some capillary rise of water, it should be minimum. It is important to note that there was rain for several days during this period. Water was found trapped for several days at the ditch line.

Slab Replacement

Full slab replacement on the travel lane was performed during the summer of 2005 as a part of an ongoing rehabilitation effort by the Appomattox Residency. Removal of 137 distressed/damaged slabs was carried out. The slabs were replaced with new cast-in-place concrete slabs. The OGDL was eliminated because of the difficulty in construction and replaced with 21B aggregates in most places except a few where No. 8 aggregate was used. In addition to aggregate, a geosynthetic fabric was used between the aggregate and soil cement to reduce the amount of abrasion. Some critical observations were made during the slab replacement operation.

A very thin layer of red clay (residue) was observed on top of the soil cement with the indentation mark of the aggregate from the drainage layer after the removal of broken slabs. The condition of the soil cement is shown in Figure 9. This softer layer was later confirmed by chemical analysis as the disintegrated/ abraded soil cement. This layer was scraped off before the new base/subbase layer was constructed.

During the replacement operation near the site of this investigation, 4 to 5 inches of standing water was observed, shown in Figure 10, over the soil cement and through the No. 8 aggregate on the day after a heavy rain storm. The site was prepared up to the base layer and was ready for the casting of concrete the day before the storm. This standing water clearly indicated a blockage of the flow of water out of the pavement. Therefore, it was suggested that the shoulder slab be removed to locate the edge drain and that the area between the pavement edge and the edge drain be examined. It was observed that the OGDL was extended for only 8 inches beyond the edge of the auxiliary lane and was blocked by the impervious native soil (16 inches wide); adjacent to that was the edge drain. Figure 11 illustrates the field condition. This was the key finding of the forensic investigation to explain the failure mechanism for the investigated section which started as early as the original construction of the pavement.

In order to confirm the drainage discontinuity in the nearby area, a few more cores at key locations were taken and investigated. Table 9 summarizes the findings. Core AC-4 shows discontinuity in the drainage layer under the shoulder. Although, this core showed the presence of No. 57 aggregate, it was contaminated with native soil. There were no indications of the presence of soil cement and/or continuous OGDL underneath this core. The other cores, AC-7 and AC-8, showed the presence of the OGDL and soil cement under the shoulder. Although the drainage layer thicknesses varied among the sections, all the drainage layers were partially clogged with the red clay. Therefore, it was not possible to ascertain the flow lines only by taking cores. It is important to note that the OGDL near cores

Table 8. Readings from Water Table Observation Wells (installed 04/06/05)

Well No.	Location			Depth of Well (ft)	Approximate Surface Elevation (ft)		In-situ Moisture		Water Level (ft)		
	Mile Post	Distance from Side Shoulder	Adjacent Borehole		Well	Concrete @ CL	Depth (ft)	MC (%)	Initial	24 hours	7 days
MW-1	8.15	5.0 ft right	1	14.9	853.5	855.0	Nearly saturated		11.0	10.4	10.2
MW-2	8.15	9.5 ft right	1	4.3	852.5	855.0	Nearly saturated		Dry	Dry	Dry
MW-3	8.30	6.6 ft right	6	14.3	853.0	853.0	0-5	32.9	Dry	Dry	Dry
							5-10	21.4			
							10-14.5	25.4			
MW-4	8.15	8.7 ft left (Median side)	1	14.8	854.5	855.0	0-5	53.2	13.0	12.4	12.4
							5-10	51.0			
							10-15.1	51.9			
MW-5	8.22	7.5 ft left (Median side)	3	15.0	854.0	853.0	0-5	23.0	Dry	14.0	14.0
							5-10	29.7			
							10-15.4	33.9			
MW-6	8.30	7.2 ft left (Median side)	6	14.6	850.5	853.0	0-5	21.7	Dry	Dry	Dry
							5-10	36.4			
							10-15.0	45.4			

CL = center line; MC = moisture content.


Figure 9. Soft Condition of Soil Cement Underneath Concrete Slab


Figure 10. Trapped Water After Rain storm During Slab Replacement

Figure 11. Discontinuity of Drainage Layer Under Shoulder Slab

AC-6, AC-7, and AC-8 was asphalt stabilized underneath the acceleration lane and shoulder. Table 9 also shows that the depths of blockage were usually higher toward the inside of the pavement rather than under the shoulder. This is an indication of minimum water flow and sediment transportation across the pavement toward the edge.

Laboratory Testing

The materials samples were tested in the laboratory to determine their properties as discussed here.

Soil Classification and Mineralogy

Soil samples were collected from the core locations and tested in the lab for classification. Soils from these locations are classified as silt and elastic silt or A-4 and A-5. The Lynchburg District Materials Laboratory tested soil from core location 1 for CBR values with a soaked CBR of 2.7% along with a swell of 2.7%. Soil samples from borehole locations 1 and 7 were sent to RJ Lee Group for x-ray diffraction testing to verify the presence of Montmorillonite clay minerals. The main clay mineral for both samples was Kaolinite (more than 50%), and there was no trace of Montmorillonite in any of them. The results are presented in Table 10. Some of the samples showed a very high in-situ moisture content as high as their liquid limit, indicating the sensitive nature of the soil in this area.

Table 9. Field Observation of Drainage Through Coring on Shoulder

Additional Cores	Location		Layer Thickness (in)		Condition/Remarks
	Approximate Marker	Core Location	Concrete	OGDL	
AC-1	Near beginning of acceleration lane at MP 8.15. Cores 3 and 4 are adjacent to each other.	Left Wheel Path	8.5	4.5	Mudline in OGDL extends vertically 2.75 in into OGDL. Partially clogged.
AC-2		Right Wheel Path	9	4.25	Mudline in OGDL extends vertically 2.5 in into OGDL. Partially clogged.
AC-3		Right Wheel Path	10	3	Mudline in OGDL extends vertically 2.0 in into OGDL. Partially clogged.
AC-4		Right Shoulder (19 in from longitudinal joint)	10.5	None	OGDL present on acceleration lane side of core hole. Appears to have No. 57 aggregate mixed with soil on other side of core. No soil cement present.
AC-6	Near exit ramp to Route 26. Cores 6 and 7 are adjacent to each other.	Right Wheel Path	9.25	3.75	Mudline in OGDL extends vertically 2.0 in into OGDL. Partially clogged.
AC-7		Right Shoulder (19 in from longitudinal joint)	9.5	4	Mudline in OGDL extends vertically 1.0 in into OGDL. Partially clogged. Soil cement present.
AC-8		Right Shoulder (19 in from longitudinal joint)	12	2.25	Mudline in OGDL extends vertically 2.0 in into OGDL. Partially clogged. Soil cement present.

Table 10. Laboratory Test Results for Soil Samples

Sample Location	Depth (ft)	In-situ Moisture (%)	Soil Classification		Atterburg Limits			Clay (%) (<0.002 mm)	Mineralogy
			USCS	AASHTO	LL	PL	PI		
Core 1	2-4	34.5	MH	A-5(12)	53.0	43.5	9.5		
Core 1	4-6	68.2							
Core 3	2-4	20.0						13	Kaolinite: 55%-60% Quartz and Muscovite: 40%-45%
Core 3	4-5		ML	A-4(3)	39.0	34.2	4.8		
Core 6	2-4	24.0						7	
Core 6	4-6	53.8	ML	A-4(0)	32.8	NP	NP		
Core 7	2-4	48.2						17	Kaolinite: 50%-55% Iron Hydroxide, Quartz, and Hematite: 45%-50%
Core 7	4-6	55.6	MH	A-5(6)	53.3	47.8	5.5		

USCS = Unified soil classification system, LL = liquid limit, PL = plastic limit, PI = plasticity index, MH = elastic silt, ML = silt.

Soil Strength and Permeability Test

Shelby tube samples were collected from core locations 3, 6, and 7. These samples were tested for unit weight, unconfined compression, permeability, and remolded strength. Results are summarized in Table 11. In one sample, the remolded strength was low (about 40% of the original strength), again indicating the sensitive nature of the original soil in this area.

Table 11. Strength and Permeability Results for Shelby Tube Samples

Shelby Tube Sample Location		In-situ Moisture (%)	Unit Weight (lb/ft^3)		Un-drained Shear Strength, psi (Unconfined Compression Test)		Permeability (cm/s)
Core No.	Depth, ft		Wet	Dry	As Received	Remolded	
3	1.7-3.7	20	122.9	102.4	8.4	9.5	4.9E-06
6	2.0-4.0	24	110.8	89.4	8.5	3.1	5.7E-05
7	1.7-3.7	48	99.7	67.2	5.9	4.8	2.6E-05

Soil Cement Evaluation

Soil cement cores were also collected in the same locations as the portland cement concrete cores. All cores, except a few with layering separations, were intact. These soil cement cores were tested at VDOT's Materials Division for compressive strength, and the range for the compressive strength was from 466 psi to 872 psi. The cement content was also determined through chemical analysis and varied from 4% to 13% by weight. Both of these values are reasonable for usual practice of soil cement in Virginia.

Concrete and Drainage Layer Test

From the visual inspection of the concrete cores it was concluded that the concrete itself was uniform and solid without apparent flaws. The unit weights of the concrete from these cores were approximately 155 pcf. Although all drainage layers were intact and attached to the concrete core, those from the cores of the damaged slabs were partially clogged with red soil, as shown in Figure 8. This soil looked similar to the soil from the soil cement, and laboratory testing of this soil found a significant amount of cement (18% to 25%). Again, this gives a clear indication of abrasion loss of soil cement.

DISCUSSION

The premature distress of concrete pavement at the Appomattox Bypass was a result of multiple factors contributing to a progressive failure: topography, soil condition, increased truck traffic, subsurface drainage, load transfer, and the joint seal.

The visual distress survey conducted during this investigation and the previous investigations revealed that the failure is limited to the travel lane. This observation clearly indicates that the truck traffic is an obvious factor in premature distresses. Therefore, these distresses are load related.

The field observations from the coring operation are presented in Table 5. The cores from the damaged slabs have two factors in common compared to the undamaged slabs:

1. a wet condition at the drainage layer
2. contamination and/or blockage of the drainage layer with soil.

Therefore, water is trapped underneath the damaged slab, and at the same time soil is getting pumped into the drainage layer.

Further investigation revealed the sources of the water and soil. The chemical analysis confirmed the clogged soil as part of the soil cement. The source of the water is discussed here. The ground water table is at least 10 feet below the pavement surface. Therefore, ground water or perched water is not likely the source. The only other way water can get into the pavement system is through the joints and/or cracks at the surface from seasonal rain. The deteriorated longitudinal and transverse joint sealants allowed rain water to infiltrate the OGDL, and water was not allowed to drain because of the blockage at the shoulder for the investigated section. The presence of trapped water in the drainage layer during initial coring indicates a lack of positive drainage; in other words, the flow lines must have some sort of blockage or interruption. This lack of drainage was also confirmed during slab replacement as discussed previously. The rain water became trapped, as shown in Figure 10. In order to establish the flow line or find the blockage, the shoulder slab was also removed near core location 1. A physical discontinuity as shown in Figure 11 was found between the drainage layer and edge drain. The speculation is that the edge drain was shifted 2 feet away from the edge of the pavement during construction to allow the concrete paver tracks (24 inches wide) to be on the native soil and not on the edge drain because of a fear that the paver might damage the edge drain.

The OGDL used in the Appomattox Bypass was cement stabilized with the exception of the auxiliary lane in the investigated section, where an asphalt-stabilized OGDL was employed. Therefore, it was rigid, and at the same time the interface at the soil cement was rough because of open-graded aggregate (VDOT No. 57 aggregate). This drainage layer was also found attached to the concrete layer during both coring and slab replacement observations. Both of these layers acted as a monolithic slab under load and applied an abrasive force on the soil cement because of the rough interface between the drainage layer and the soil cement. The presence of water aggravated the situation by producing a higher abrasion loss of soil cement under a wet condition with repeated heavy truck loads. The presence of a rough surface and a very thin softer layer as shown in Figure 9 supports the theory of abrasion loss. The settlement of slabs is a direct result of abrasion loss, although there is no evidence of pumping on the surface. The abraded soil cement did not come out on the surface despite pumping because there was enough pore space in the drainage layer to permit horizontal travel. This abraded soil cement stayed in the drainage layer and contaminated/clogged the drainage layer as shown in Figure 8. The abrasion of the soil cement produced a loss of support underneath the concrete slab and eventually created mid-slab cracks. The erosion of the top 1 in of soil cement allowed the slab to fault easily at both the longitudinal and transverse joints. In this failure mechanism, three elements were present: water entrapment, blockage of the OGDL, and truck loading. A separation layer between the rough surface of the cement-stabilized OGDL and the soil cement would have reduced the amount of abrasion.

CONCLUSIONS

- The pavement failure is the direct result of poor drainage.
- The continuity between the OGDL and edge drain was not established during construction of the investigated section.
- The rain water entered into the drainage layer through the broken seal at both longitudinal and transverse joints. Because of the lack of drainage, water became, trapped underneath the pavement.
- The heavy load of truck traffic caused the abrasion of soil cement by the rough surface of the OGDL and the wet condition accelerated the process.
- Low load transfer at the joint aggravated the situation.
- The mid-slab crack and settlement resulted from the loss of support attributable to the abrasion of the soil cement.

RECOMMENDATIONS

The concrete pavement is failing progressively, and the main contributing factors are drainage and truck traffic. In order to reduce or eliminate the progression of the distresses, both factors must be addressed. The approach to provide the full service life of this pavement without jeopardizing the public safety and comfort is to consider the following four key elements in the rehabilitation effort.

1. Replace the damaged slabs.
2. Stop the intrusion of water from the surface through the joint as much as possible.
3. Eliminate or reduce the abrasion of soil cement to stop OGDL clogging.
4. Provide positive drainage to reduce the entrapment of water.

Toward these goals, VDOT has already started replacing the most seriously damaged slabs. The replacement slabs are of the same thickness as the original slabs, and the strengths are in accordance with current VDOT standards. Because of the difficulty in construction, the ODGL is replaced with a drainable stone base/subbase. The choice aggregate would have been VDOT No. 78, but it may not provide adequate stability for construction of the slab and placement of the dowel bars. Therefore, VDOT standard aggregate base material, No. 21B, was used. The success of this layer will depend on controlling the amount of materials passing the No. 200 sieve. The flexibility of this stone layer should reduce the amount of soil cement abrasion. As an added precaution VDOT has also used a geosynthetic separation layer between the stone subbase and the soil cement. Both longitudinal and transverse joints were sealed with appropriate joint sealers so the intrusion of surface water through the joint will be reduced.

In order to preserve the current investment of replacing the failing slabs, the water blockage needs to be removed. Here are the suggested steps to create a larger and easier water removal system from the adjacent pavement:

1. Saw cut and remove 2 feet of the concrete shoulder immediately adjacent to the edge of the mainline pavement or the auxiliary lanes.

2. Dig out the 16 inches of native soil standing between the drainage layer and the edge drain to a depth at least equal to the depth of the existing edge drain, and replace it with AASHTO No. 57 aggregate to a level meeting the top of the OGDL. This widened trench will provide all the needed elements for proper drainage, i.e., intercept, collect, and discharge.

3. Use capping materials, about 9 inches (replacing the 2 feet of concrete), of either concrete using tie bars or asphalt with its surface enhanced with rumble strips for added safety to reduce the potential of truck traffic wandering on top of the 2-foot repair.

Plans to deepen the ditches are underway by the residency, which will eliminate the chances for water backing into the pavement system in the flat geometric area on this project.

Regarding the remaining section of the project, the presence of faulting in the longitudinal and transverse joints needs to be determined visually. This can be supported by the FWD load transfer data that recently obtained. The presence of faulting and low load transfer indicates the potential for the same failure mechanism. Verification of the presence of blockage in the remaining section can be achieved by spot removal of very few shoulder slabs to ascertain the condition. If the blockage is confirmed, a similar treatment for these sections can be considered to save the rest of the project. Alternatively, if the slab is still undamaged, the following may also be considered if economically feasible:

- Reestablish the drainage as suggested. In this case, edge drains need to be inspected regularly since the soften soil cement which clogged the drainage layer may start to come out through the edge drain.

- Jack up the settled slabs (cracked slabs need to be replaced).

- Retrofit the dowel bars to reestablish the load transfer.

COSTS AND BENEFITS ASSESSMENT

This investigation was very valuable to the Appomattox Residency because it complements their efforts. Identifying the failure mechanism in the field and backing up the finding through the replacement of slabs provided the certainty the residency and the decision makers needed to proceed with a remedial action plan.

The cost of replacing the 137 slabs (2,740 square yards) was $832,894 at the rate of $304/square yard. This makes the findings of this investigation a turning point in the effort to protect the remainder of the project from potential failure. Although it is not expected that all slabs on the project will be replaced in the same manner and at the same cost of the current contract, it is obvious that considerable savings will be realized as a result of this investigation, since the failure mechanism is now identified.

As an example, the estimated cost to install a drainage path for the entire bypass (5.6 lane miles), as a worst case scenario, would be $500,000, where as replacing only 2% of the slabs (80 slabs) would cost the same. Reestablishing the drainage would stabilize the section from further deterioration and slab replacement could be minimized, which would be a great return on the money invested.

ACKNOWLEDGMENTS

The authors thank many VDOT personnel and the contractor for their help during this investigation. They acknowledge the excellent cooperation and professionalism of several VDOT personnel, namely Don French, Don Austin, Andy Babish, and Chris Simpson. They also thank Steve Lane of VTRC, Tanveer Chowdhury of VDOT's Asset Management Division, Bill Wise of VDOT's Lynchburg District, Robert Reid of VDOT's Nondestructive Testing Section, and many other staff of VDOT's Materials Division for their help during this investigation.

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