Final Report VTRC 10-R3

Virginia Transportation Research Council

research report

Evaluation of Jointed Reinforced Concrete Pavement Rehabilitation on I-64 in the Richmond and Hampton Roads Districts of Virginia

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BRIAN K. DIEFENDERFER, Ph.D., P.E.
Research Scientist

DAVID W. MOKAREM, Ph.D.
Research Scientist

Virginia Transportation Research Council, 530 Edgemont Road,
Charlottesville, VA 22903-2454, www.vtrc.net, (434) 293-1900

Standard Title Page - Report on Federally Funded Project

1. Report No.: FHWA/VTRC 10-R3	2. Government Accession No.:	3. Recipient's Catalog No.:
4. Title and Subtitle: Evaluation of Jointed Reinforced Concrete Pavement Rehabilitation on I-64 in the Richmond and Hampton Roads Districts of Virginia		5. Report Date: September 2009 6. Performing Organization Code:
7. Author(s): Brian K. Diefenderfer and David W. Mokarem		8. Performing Organization Report No.: VTRC 10-R3
9. Performing Organization and Address: Virginia Transportation Research Council 530 Edgemont Road Charlottesville, VA 22903		10. Work Unit No. (TRAIS): 11. Contract or Grant No.: 78546
12. Sponsoring Agencies' Name and Address: Virginia Department of Transportation 1401 E. Broad Street Richmond, VA 23219	Federal Highway Administration 400 North 8th Street, Room 750 Richmond, VA 23219-4825	13. Type of Report and Period Covered: Final 14. Sponsoring Agency Code:

15. Supplementary Notes:
This project was financed with Part II State Planning and Research (SPR) funds at an estimated cost of $163,000.

16. Abstract:

Beginning in 2004, the Virginia Department of Transportation (VDOT) undertook a series of pavement rehabilitation projects to address deficiencies in three sections of the I-64 corridor between Richmond and Newport News. I-64 serves as the primary avenue between the Richmond and Hampton Roads metropolitan areas and carries a combined traffic volume ranging from approximately 20,000 to 90,000 vehicles per day. For nearly 100 mi, this roadway is a four-lane divided facility that was originally built between the late 1960s and early 1970s as either a jointed reinforced or continuously reinforced concrete pavement. The existing concrete pavement was rehabilitated using three rehabilitation procedures: two standard approaches and an experimental approach. The standard rehabilitation procedures included the use of full-depth portland cement concrete (PCC) patches overlaid by a hot-mix asphalt (HMA) overlay and full-depth PCC patches followed by grinding of the pavement surface. The experimental rehabilitation procedure consisted of the use of full- and partial-depth HMA patches followed by an HMA overlay. The purpose of this study was to document the initial condition and performance to date of the I-64 project and to summarize similar work performed by state departments of transportation other than VDOT.

The pavement rehabilitation cost per lane-mile was nearly 20% less for the section of I-64 for which full-depth PCC patches followed by grinding of the pavement surface was used than for the other two sections. However, the experimental results do not allow for a comparison to determine any differences in the structural capacity or service life between the sections.

The study recommends that VDOT's Materials Division annually monitor the ride quality of the pavement in the three rehabilitated sections of I-64 so that the end of service life can be defined as the pavement roughness increases because of deterioration. Further, the Virginia Transportation Research Council should collaborate with other research organizations to encourage and pursue full-scale or laboratory-scale accelerated pavement testing to determine the optimum repair materials and methods for pre-overlay repair of existing PCC pavements and to develop models to quantify the deterioration of an asphalt overlay placed over an existing concrete pavement because of reflection cracking.

17 Key Words: Pavement rehabilitation, pavement repair, overlay, diamond grinding, patching	18. Distribution Statement: No restrictions. This document is available to the public through NTIS, Springfield, VA 22161.		
19. Security Classif. (of this report): Unclassified	20. Security Classif. (of this page): Unclassified	21. No. of Pages: 44	22. Price:

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

FINAL REPORT

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Brian K. Diefenderfer, Ph.D., P.E.
Research Scientist

David W. Mokarem, Ph.D.
Research Scientist

Virginia Transportation Research Council
(A partnership of the Virginia Department of Transportation
and the University of Virginia since 1948)

In Cooperation with the U.S. Department of Transportation
Federal Highway Administration

Charlottesville, Virginia

September 2009
VTRC 10-R3

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ABSTRACT

Beginning in 2004, the Virginia Department of Transportation (VDOT) undertook a series of pavement rehabilitation projects to address deficiencies in three sections of the I-64 corridor between Richmond and Newport News. I-64 serves as the primary avenue between the Richmond and Hampton Roads metropolitan areas and carries a combined traffic volume ranging from approximately 20,000 to 90,000 vehicles per day. For nearly 100 mi, this roadway is a four-lane divided facility that was originally built between the late 1960s and early 1970s as either a jointed reinforced or continuously reinforced concrete pavement. The existing concrete pavement was rehabilitated using three rehabilitation procedures: two standard approaches and an experimental approach. The standard rehabilitation procedures included the use of full-depth portland cement concrete (PCC) patches overlaid by a hot-mix asphalt (HMA) overlay and full-depth PCC patches followed by grinding of the pavement surface. The experimental rehabilitation procedure consisted of the use of full- and partial-depth HMA patches followed by an HMA overlay. The purpose of this study was to document the initial condition and performance to date of the I-64 project and to summarize similar work performed by state departments of transportation other than VDOT.

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Brian K. Diefenderfer, Ph.D., P.E.
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David W. Mokarem, Ph.D.
Research Scientist

INTRODUCTION

I-64 serves as the primary avenue between the Richmond and Hampton Roads metropolitan areas in Virginia and carries a combined traffic volume ranging from approximately 20,000 to 90,000 vehicles per day. The roadway is a vital link for vacationers traveling to Virginia's beaches, shipping containers going to and from the Norfolk-area ports, and as a hurricane evacuation route for the Hampton Roads region. For nearly 100 mi, this roadway is a four-lane divided facility that was originally built between the late 1960s and early 1970s as either a jointed reinforced or continuously reinforced concrete pavement.

A study by the Virginia Department of Transportation's (VDOT's) Materials Division (VDOT, 2002) showed portions of the pavement on I-64 between Richmond and Newport News to be in poor condition and requiring maintenance at an ever-increasing rate. Based on information generated from this study, various rehabilitation methods were developed to extend the life of the pavement. After internal discussions with representatives of the asphalt and concrete paving industries in Virginia, three methods for rehabilitating the jointed reinforced concrete pavement (JRCP) (two methods in the Richmond District and one method in the Hampton Roads District) were recommended to provide a pavement service life of 10 years. The rehabilitation methods were performed in three sections as follows:

1. *Section 1 (MP 195.67-200.59):* full- and partial-depth hot-mix asphalt (HMA) patches with a 5-in HMA overlay

2. *Section 2 (MP 200.59-205.40):* full-depth portland cement concrete (PCC) patches with a 3.5-in HMA overlay

3. *Section 3 (MP 237.20-253.60)*: full-depth PCC patches with diamond grinding.

The processes used for Sections 2 and 3 are relatively common and have a history of performing well in Virginia and surrounding states. The materials used for patching proposed for Section 1, however, have not been previously used together for this type of application in Virginia. The approach of using full- and partial-depth HMA patches has worked well for continuously reinforced concrete pavements; however, the performance with JRCP in Virginia is not known. In addition, the performance of these materials cannot be modeled adequately with current pavement design procedures used in Virginia (VDOT, 2000).

The materials recommended for HMA patching in Section 1 are an HMA mixture having a 19 mm or a 25 mm nominal maximum aggregate size (NMAS) and using a performance grade (PG) 70-22 binder. These mixtures are designated by VDOT as IM-19.0D and BM-25.0D, respectively. The HMA overlay consisted of three layers. The first layer, a leveling course, used a 1.5-in-thick surface mixture with an NMAS of 12.5 mm and using a polymer-modified PG 76-22 binder. This mixture is designated by VDOT as SM-12.5(M). The second layer placed was a 2.0-in-thick stone-matrix asphalt (SMA) intermediate mixture with an NMAS of 19.0 mm and having a PG 76-22 binder. This mixture is designated by VDOT as SMA-19.0 (76-22). The third layer, the wearing surface, is a 1.5-in-thick SMA surface mixture with an NMAS of 12.5 mm and having a PG 76-22 binder. This mixture is designated by VDOT as SMA-12.5 (76-22).

The material recommended for patching in Sections 2 and 3 is a PCC requiring a 2,000 lb/in^2 compressive strength prior to the return of traffic. The HMA overlay in Section 2 consisted of two layers. The first layer placed was a 2.0-in-thick layer of SMA-19.0 (76-22); the second layer placed was a 1.5-in-thick SMA-12.5 (76-22) wearing surface. The repair work in Section 3 consisted of removing the existing pavement where the existing joints were in poor condition. These areas were then patched with PCC (having dowel bars inserted prior to placement of the patch material), and then the final surface of the entire pavement was diamond ground.

The pavement in the three test sections was compared to a defined level of acceptable deterioration at 10 years of service. The acceptable level of deterioration for Sections 1 and 2 was defined as follows:

- average International Roughness Index (IRI) less than 110 in/lane-mile
- no 0.1-mi section with an IRI greater than 170 in/mi
- average rut depth of less than 0.5 in per wheel path per mile
- no 0.1-mi section with a rut depth greater than 1 in per wheel path
- no more than 15 Severity Level 3 reflective cracks per lane-mile as defined by VDOT (2007a).

The acceptable level of deterioration for Section 3 was defined as follows:

- IRI of less than 110 in/lane-mile
- no 0.1-mi section with an IRI greater than 170 in/mi
- no more than 15 deteriorated transverse joints or asphalt patches located at a joint per lane-mile requiring Type I or II patches
- no more than 15 deteriorated concrete patches per lane-mile with a condition of Severity Level 3 as defined by VDOT (2007a).

PURPOSE AND SCOPE

The purpose of this study was to summarize previous work performed by departments of transportation (DOTs) other than VDOT and to document the initial condition and performance to date of an existing JRCP in Virginia that was rehabilitated using three rehabilitation procedures: two standard approaches and one experimental approach. The standard rehabilitation procedures included the use of full-depth PCC patches overlaid by an HMA overlay and full-depth PCC patches followed by grinding of the pavement surface. The experimental rehabilitation procedure consisted of full- and partial-depth HMA patches followed by an HMA overlay.

The scope of the project encompassed the three sections of I-64 in VDOT's Richmond and Hampton Roads districts previously enumerated.

METHODS

Three tasks were conducted to achieve the objectives of this study:

1. *A literature review was conducted to identify previous work performed by DOTs other than VDOT.*

2. *The initial condition of the three sections of I-64 was evaluated primarily by ride quality, skid resistance, overlay thickness, time required to perform the rehabilitation, and cost per lane-mile rehabilitated.*

3. *An analysis of the traffic volume and truck loading for each of the three sections was conducted.* Through use of this information, a determination can be made about the traffic loading carried by each section during any future analysis.

Literature Review

The literature review was conducted by searching various transportation engineering-related databases such as: Transportation Research Information Services (TRIS) bibliographic database, the catalog of Transportation Libraries (TLCat), the Catalog of Worldwide Libraries (WorldCat), and the Transportation Research Board Research in Progress (RiP) and Research Needs Statements (RNS) databases.

Evaluation of Initial Condition of the Three Sections of I-64

Pavement Smoothness

Pavement smoothness testing was conducted using an inertial profiler equipped with lasers and accelerometers. The profilers collected longitudinal profile data in accordance with

ASTM E950, Standard Test Method for Measuring the Longitudinal Profile of Traveled Surfaces with an Accelerometer Established Inertial Profiling Reference. The profile data were then used to calculate an IRI value, giving a measure of the roughness of the longitudinal profile. The IRI values were calculated in accordance with AASHTO PP 37-04, Determination of International Roughness Index (IRI) to Quantify Roughness of Pavements.

Skid Resistance

Skid resistance testing was conducted on the pavement sections with a lock-wheeled friction unit (ASTM E-274) using a smooth test tire (ASTM E524). The tests were conducted before and after repairs to determine the effects of repair on the skid resistance of the sections.

Overlay Thickness

Ground-penetrating radar (GPR) was used to assess the layer thickness of the HMA overlay placed on Sections 1 and 2. This technique has been shown to be an effective means for nondestructively determining the pavement layer thickness (Maser, 2002; Maser and Scullion, 1992). As the structural capacity of a pavement section depends on the thickness of the pavement, it could be expected that a thinner pavement would not carry as many vehicles as a thicker pavement, all else being equal. Therefore, it was considered important to determine the overlay thickness in Sections 1 and 2 to determine if the thickness of any areas of pavement was significantly different. If so, such areas would be candidates for early deterioration and should be monitored. GPR testing was not performed in Section 3 as it would be unlikely that the areas patched with PCC could be distinguished from the surrounding original PCC pavement.

The GPR system used in this study consisted of a 2.0 GHz air-launched horn antenna and a SIR-20 controller unit, both manufactured by Geophysical Survey Systems, Inc. The antenna was mounted on a survey vehicle as shown in Figure 1. The pulse rate of the antenna was

Figure 1. VDOT's Air-Launched GPR System

maintained at a constant rate of 2 scans per foot, regardless of the vehicle speed, using an integrated distance measuring instrument. At each test site, the GPR testing was conducted at the prevailing speed of the facility. All data were processed by the software RADAN (version 6.5.3.0) developed by Geophysical Survey Systems, Inc. The software allows the user to view the collected data and identify the layer boundaries. The thickness to each layer boundary is automatically calculated.

Time Required to Perform the Rehabilitation

The construction schedule was used to determine the length of time required to complete the portions of the project that related to the pavement rehabilitation operations.

Cost per Lane-mile Rehabilitated

The average cost per lane-mile to perform the pavement rehabilitation work was calculated based on project payment information provided by VDOT district personnel.

Analysis of Traffic Volume and Truck Loading

The traffic volume and truck loading data were complied from VDOT-published traffic information.

RESULTS AND DISCUSSION

Literature Review

Concrete Pavement Rehabilitation

Determining the appropriate rehabilitation treatment for a jointed concrete pavement is a question that is not unique to Virginia. The literature contains many studies of field trials performed by state and provincial DOTs to determine the service life of various concrete pavement rehabilitation options (Hall et al., 1991; Kazmierowski and Sturm, 1991; McGhee, 1979; Pierce, 1994; Von Holdt and Scullion, 2005; Wen et al., 2006). These rehabilitation options range from minor patching to patching plus overlay to complete reconstruction. The rehabilitation intensity level, overlay thickness, overlay material type (whether asphalt or concrete), and type of patching material (asphalt or concrete) are all potential variables.

Hall et al. (1991) investigated the survival of asphalt overlays on interstate pavements in Illinois. They investigated 213 sections where asphalt overlays (ranging from 1.5 to 6.0 in thick) were placed on JRCP. The study reported that asphalt overlay of existing PCC pavements is the most common rehabilitation method used in the United States. Further, the predominant causes of overlay failure are reflection cracking at the transverse joints, localized distress caused by D-cracking (an issue where water freezes in certain types of porous aggregates, not considered to be a problem in Virginia) in the underlying concrete pavement, rutting in the overlay, and deterioration of patches and expansion joints. The authors stated that the use of asphalt patching as a means of pre-overlay repair "often" results in early deterioration of the asphalt overlay. The study discusses reflection cracking as a phenomenon caused by a concentration of strain energy in the asphalt overlay from vehicular load- or temperature-induced movement at cracks or slabs in the concrete below. Therefore, a reduction in the movement across a crack or a joint is an effective means for reducing the development of strain in the asphalt overlay. The study further stated that full-depth dowelled PCC repairs at joints and working cracks is an effective means for reducing the occurrence of reflection cracking. Further, improving subsurface drainage, repairing voids to improve slab support, restoring load transfer, and increasing the overlay thickness are also effective in reducing or delaying the occurrence of reflection cracks.

More contemporary studies have involved other techniques to minimize the occurrence of reflection cracking, including the use of reinforcement (Brown et al., 2001; Kim and Buttlar, 2002), geosynthetics (Buttlar et al., 2000; Button and Lytton, 2007; Elseifi and Al-Qadi, 2005), granular interlayers (Kim and Buttlar, 2002; Titi et al., 2003), and slab-fracturing techniques (Freeman, 2002; Sebesta and Scullion, 2007). These treatments vary in their effectiveness at delaying or reducing reflective cracking, but inevitably, reflection cracking still occurs.

Hall et al. (1991) also stated that reflection cracking can have a "considerable (often controlling) influence" on the life of an asphalt overlay on top of a JRCP. Reflection cracking is detrimental as it negatively impacts the pavement smoothness and provides an avenue for moisture to enter the pavement system. The study recommended sealing as an effective procedure for reducing moisture intrusion and as a means for retarding their progression from low to higher severity levels.

Wen et al. (2006) investigated the development of transverse reflection cracking in asphalt overlays placed on existing concrete pavements by evaluating three pre-overlay methods of repair: doweled full-depth concrete patches, non-doweled full-depth concrete patches, and full-depth asphalt patches. The results of this study showed that the rate of transverse crack development was lowest for the doweled concrete patches and highest for the asphalt patches. The study also developed a regression equation describing the relationship between transverse crack development rate and overlay thickness. The regression equation suggested that overlays greater than approximately 3 in offer only a marginal improvement in reducing or delaying the transverse reflection cracking development rate with increasing overlay thickness.

Gharaibeh and Darter (2003) conducted a probabilistic study of the service life of more than 4,000 directional miles of original and rehabilitated pavements in Illinois. Their study seems to support the marginal improvement in additional thickness reported by Wen et al. (2006). Gharaibeh and Darter concluded that concrete pavements should be programmed for overlay before their condition requires that a thick overlay be placed. The study concluded "a thick overlay placed over a severely deteriorated pavement has a load-carrying capacity similar to or less than that of a thin-overlay placed over a pavement in better condition."

Kazmierowski and Sturm (1991) reviewed a concrete pavement rehabilitation project in Ontario, Canada. The project consisted of full- and partial-depth concrete patching followed by diamond grinding in one direction and an unbonded concrete overlay in the other direction. The full-depth repaired areas included dowel bars to transfer loading from the concrete patches to the existing concrete pavement. The authors noted that previous repair techniques included full- and partial-depth asphalt patching followed by an asphalt overlay of varying thickness. They stated that this repair technique was effective in the short term but that over time, the asphalt patches began to distort causing a reduction in the pavement smoothness. The authors concluded that concrete patching and concrete overlay repair techniques were effective in increasing the structural capacity of the pavement but that long-term monitoring was needed for confirmation.

The performance of diamond-ground pavements was studied by Rao et al. (1999). Diamond (or surface) grinding is a process where the surface of a concrete pavement is ground smooth to reduce any roughness that occurs at faulted joint locations. As it does not increase the load-carrying capacity of a concrete pavement or restore sub-slab support, it is often used in conjunction with other repair techniques such as dowelled full-depth repairs or slab under-sealing (a process where grout is injected underneath the slab to fill any voids and restore support) to provide a pavement surface that has an IRI nearly equal to that of an HMA overlay. The authors suggested that a rehabilitation treatment more substantial than diamond grinding (such as overlay or reconstruction) should be considered if a pavement is structurally deficient. When comparing the surface texture between diamond-ground and tined surfaces, the authors stated that the ground surface texture was superior to tined surfaces in that accident rates were shown to be significantly lower in both wet and dry conditions for one subset of the data. The authors also conducted a survival analysis that showed a high probability that a diamond-ground pavement would last at least 10 years before another diamond grinding cycle was required. The analysis showed that the probability of failure before 8 and 10 years was less than approximately 2% and 12%, respectively. A subsequent diamond grinding might be needed because of either the return of faulting or a reduction in surface texture over time. The authors concluded that diamond grinding is effective at extending the service life of concrete pavements and that so long as the pavement structural capacity is sufficient, its life may be extended even further by multiple cycles (up to three or four) of diamond grinding.

Hall et al. (2002) studied the effectiveness of rehabilitation treatments on flexible and rigid pavements using data from the long-term pavement program (LTPP). The study offered several conclusions regarding rehabilitation of pavements using various treatment types and intensity levels. The study concluded that concrete pavements that received surface grinding and non-overlay repair tended to be smoother over the long term than pavements that were not ground but also received non-overlay repair. The average initial post-treatment IRI of pavements that received overlay repair was approximately 70 in/mi versus an average initial post-treatment IRI of pavements that received surface grinding and non-overlay repair of 66 in/mi. The study compared the development of roughness over time for pavement sections that received grinding and non-overlay repair with sections that received overlay repair versus control sections that received no repair at all. The study showed that those sections that received grinding and non-overlay repair approached the roughness of the control sections faster than those sections that received overlay repair.

Pavement Smoothness

Pavement smoothness was defined by McGhee and Gillespie (2007) as "the absence of bumps and dips in the riding surface of a pavement." To the traveling public, a lack of pavement smoothness may result in a decreased ride quality, minor annoyance, or reduction in fuel economy (Gillespie and McGhee, 2007). Smith et al. (1997) stated that rough pavements can pose safety concerns, disrupt the flow of traffic, reduce optimum travel speeds, and increase vehicle wear and may increase fuel consumption.

The benefits of smoother pavements have been documented by many studies. Smith et al. (1997) studied more than 200 pavement sections and reported that initial pavement smoothness had a significant effect in extending pavement life for 80% of new construction (both flexible and rigid pavements) and for 70% of asphalt overlay projects. In addition, pavements that were initially smoother were more likely to remain smoother throughout their service life. An analysis of fuel economy was provided by Sime and Ashmore (2000) during testing at the WesTrack accelerated pavement test facility in Nevada. The authors showed that when comparing the effects before and after a pavement rehabilitation on the test track, a 10% reduction in IRI reduced truck fuel consumption by 4.5%. McGhee and Gillespie (2006) studied the effects of VDOT's use of paving contracts with a smoothness specification. Their analysis showed that projects including a smoothness specification were on average 8.8 in/mi smoother than projects that did not include the specification. The authors suggested that this increase in smoothness should allow VDOT to delay rehabilitation to a later date, offering potentially significant cost savings to VDOT.

Initial Condition of I-64 Pavement Sections

Pavement Smoothness

As part of the analysis, pavement smoothness testing was conducted on all of the sections before and after repairs. Each lane was tested individually, and IRI values were calculated for each 0.01 mi. The IRI data for each lane were then averaged for the entire section and for each 0.1 mi. The 0.1-mi average was calculated to determine the number of 0.1-mi segments with an average IRI greater than 170 in/mi. The associated standard deviations were also calculated for each lane to determine variability.

A frequency distribution analysis of the IRI data was performed for each lane of each section. The results for each lane are presented in Appendix A. The frequency percentage was calculated for each lane according to IRI values ranging from 0 to more than 200. The IRI for each section was compared to the following limit of acceptable deterioration after 10 years:

- average IRI less than 110 in/lane-mile
- no 0.1-mi segment with an IRI greater than 170 in/mi.

An analysis was performed using these limits to determine the condition of each section before and after repairs.

Section 1

Section 1 was a portion of I-64 in Henrico County. The section consists of three lanes in each the eastbound and westbound directions with the right-hand lane designated as Lane 0; the milepost locations for each lane are shown in Table 1. Profile testing was conducted on July 15, 2004, and June 17, 2008. Table 2 presents the data for the eastbound lanes, and Table 3 presents the data for the westbound lanes.

After repairs, the average IRI improved for all lanes in both directions. For the eastbound lanes, the average IRI improved 61%, 63%, and 51% for lanes 0, 1, and 2, respectively. For the westbound lanes, the average IRI values improved 59%, 58%, and 46% for lanes 0, 1, and 2, respectively.

Based on the 10-year limits in the original proposal, the IRIs for all lanes in Section 1 were below the 110 in/lane-mile average IRI. Lane 2 in the eastbound direction had one 0.1-mi segment with an IRI greater than 170 in/mi.

Table 1. Milepost Locations for Section 1

Eastbound		**Westbound**	
Lane	**Mileposts**	**Lane**	**Mileposts**
Lane 0	195.67 to 197.43	Lane 0	197.72 to 195.67
Lane 1	195.67 to 200.59	Lane 1	200.59 to 195.67
Lane 2	195.67 to 200.59	Lane 2	200.59 to 195.67

Table 2. Results of Pavement Smoothness Testing: Eastbound, Section 1

Lane		**07/15/2004**	**06/17/2008**
Lane 0	Average IRI Value	175	68
	Standard Deviation	67.3	54.1
	0.1-Mi Segments > 170	11	0
Lane 1	Average IRI Value	157	58
	Standard Deviation	60.2	32.9
	0.1-Mile Segments > 170	15	1
Lane 2	Average IRI Value	124	61
	Standard Deviation	47.4	28.5
	0.1-Mile Segments > 170	1	0

Table 3. Results of Pavement Smoothness Testing: Westbound, Section 1

Lane		**07/15/2004**	**06/17/2008**
Lane 0	Average IRI Value	176	73
	Standard Deviation	70.6	20.8
	0.1-Mile Segments > 170	11	0
Lane 1	Average IRI Value	147	62
	Standard Deviation	52.7	30.9
	0.1-Mile Segments > 170	12	0
Lane 2	Average IRI Value	121	65
	Standard Deviation	42.6	34.9
	0.1-Mile Segments > 170	2	0

Section 2

Section 2 was a portion of I-64 in Henrico and New Kent counties. The section consisted of two lanes each in the eastbound and westbound directions with the right-hand lane designated as lane 1; the milepost locations for each lane are presented in Table 4. Profile testing was conducted on July 15, 2004, and June 17, 2008. Table 5 presents the data for the eastbound lanes, and Table 6 presents the data for the westbound lanes.

After repairs, the average IRI improved for all lanes. For the eastbound lanes, the average IRI improved 66% and 50% for lanes 1 and 2, respectively. For the westbound lanes, the average IRI improved 54% and 31% for lanes 1 and 2, respectively.

Based on the 10-year limits in the original proposal, the IRIs for all lanes in Section 1 were below the 110 in/lane-mile average IRI. However, lane 2 in the westbound direction had one 0.1-mi segment for which the IRI was greater than 170 in/mi.

Table 4. Milepost Locations for Section 2

Eastbound		**Westbound**	
Lane	**Mileposts**	**Lane**	**Mileposts**
Lane 1	200.59 to 205.40	Lane 1	205.40 to 200.59
Lane 2	200.59 to 205.40	Lane 2	205.40 to 200.59

Table 5. Results of Pavement Smoothness Testing: Eastbound, Section 2

Lane		**07/15/2004**	**06/17/2008**
Lane 1	Average IRI Value	172	58
	Standard Deviation	63.1	32.9
	0.1-Mile Segments > 170	23	0
Lane 2	Average IRI Value	117	59
	Standard Deviation	42.1	26.6
	0.1-Mile Segments > 170	1	0

Table 6. Results of Pavement Smoothness Testing: Westbound, Section 2

Lane		**07/15/2004**	**06/17/2008**
Lane 1	Average IRI Value	164	76
	Standard Deviation	57.4	32.4
	0.1-Mile Segments > 170	19	0
Lane 2	Average IRI Value	111	77
	Standard Deviation	46.5	35.5
	0.1-Mile Segments > 170	2	1

Section 3

Section 3 was a portion of I-64 in York County. The section consists of two lanes in each the eastbound and westbound directions with the right-hand lane designated as Lane 1; the milepost locations for each lane are presented in Table 7. Profile testing was conducted on April 13, 2004, and July 1, 2008. Table 8 presents the data for the eastbound lanes, and Table 9 presents the data for the westbound lanes.

Table 7. Milepost Locations for Section 3

Eastbound		Westbound	
Lane	**Mileposts**	**Lane**	**Mileposts**
Lane 1	237.20 to 253.60	Lane 1	253.60 to 237.20
Lane 2	237.20 to 253.60	Lane 2	253.60 to 237.20

Table 8. Results of Pavement Smoothness Testing: Eastbound, Section 3

Lane		04/13/2004	07/01/2008
Lane 1	Average IRI Value	123	84
	Standard Deviation	44.3	26.1
	0.1-Mile Segments > 170	32	0
Lane 2	Average IRI Value	108	81
	Standard Deviation	41.7	26.9
	0.1-Mile Segments > 170	6	0

Table 9. Results of Pavement Smoothness Testing: Westbound, Section 3

Lane		04/13/2004	07/01/2008
Lane 1	Average IRI Value	149	88
	Standard Deviation	81.1	24.8
	0.1-Mile Segments > 170	29	0
Lane 2	Average IRI Value	116	85
	Standard Deviation	60.1	27.5
	0.1-Mile Segments > 170	13	0

After repairs, the average IRI improved for all lanes. For the eastbound lanes, the average IRI values improved 32% and 25% for lanes 1 and 2, respectively. For the westbound lanes, the average IRI values improved 41% and 27% for lanes 1 and 2, respectively.

Based on the 10-year limits in the original proposal, the IRIs of all lanes in Section 1 were below the 110 in/lane-mile average IRI. None of the lanes in either direction had a 0.1-mi segment with an IRI greater than 170 in/mi.

Summary of IRI Results

Sections 1 and 2. Repairs for Sections 1 and 2 involved patching and asphalt overlays. Section 1 had a 5-in asphalt overlay, and Section 2 had a 3.5-in asphalt overlay. From the IRI data obtained before and after repairs were performed on each section, the data show that the before-repair average IRI values were highest for lane 0, followed by lane 1 and then lane 2. Following repairs, the average IRI values were still highest in lane 0 followed by lane 1 and then lane 2. Table 10 presents the average IRI data for each lane before and after repairs.

The data show that there are larger differences in average IRI values according to lane type before the repairs were performed on Sections 1 and 2 as compared to after-repair average IRI values. Before repairs, lane 0 and lane 1 had significantly higher average IRI values than lane 2. After repairs, the average IRI values for all lanes were within 10% of one another.

Section 3. Repairs for Section 3 involved patching and then diamond grinding. Table 11 presents the average IRI data for each lane before and after repairs.

Table 10. Average Pavement Smoothness: Sections 1 and 2

Lane/Section	Avg. IRI (Before)	Avg. IRI (After)
Lane 0		
Eastbound Section 1	175	68
Westbound Section 1	176	73
Average	176	71
Lane 1		
Eastbound Section 1	157	58
Westbound Section 1	147	62
Eastbound Section 2	172	58
Westbound Section 2	164	76
Average	160	64
Lane 2		
Eastbound Section 1	124	61
Westbound Section 1	121	65
Eastbound Section 2	117	59
Westbound Section 2	111	77
Average	118	66

Table 11. Average Pavement Smoothness: Section 3

Lane/Section	Avg. IRI (Before)	Avg. IRI (After)
Lane 1		
Eastbound Section 3, Lane 1	123	84
Westbound Section 3, Lane 1	149	88
Average	136	86
Lane 2		
Eastbound Section 3, Lane 2	108	81
Westbound Section 3, Lane 2	116	85
Average	112	83

The data show that there are larger differences in average IRI values according to lane type before the repairs were performed on Section 3 as compared to after-repair average IRI values. Before repairs, the average IRI value for the travel lane was about 18% higher than the average IRI value for the passing lane. After repairs, the average IRI values for the travel and passing lanes were within 3% of each other.

Skid Resistance

Table 12 presents the average skid resistance for the repair sections before and after repairs were conducted. These results are the average skid resistance of the entire section tested. Sections 1 and 2 were tested together because they abut each other and an average was calculated for the entire section.

Table 12. Average Skid Resistance

Section	Before	After
Sections 1 & 2 Eastbound	45.3	42.3
Sections 1 & 2 Westbound	42.9	46.6
Section 3 Eastbound	40.8	39.3
Section 3 Westbound	47.2	41.9

The results from Table 12 show that the average skid resistance values for all sections changed very little after repairs were conducted. The lowest average skid resistance value was 39.3, which was well above the recommended trigger value of 20 (Mahone and Sherwood, 1996).

Overlay Thickness

GPR was used to survey Sections 1 and 2 to determine the HMA overlay thickness. Figure 2 shows an example of the results for a portion of the eastbound direction of lane 1 from Laburnum Avenue to Airport Road (approximately Milepost 195.6 to 197.8). Figure 2 shows three bridges that are encountered over this segment: Laburnum Avenue, Oakleys Lane, and Airport Road.

Figures B.1 through B.10 in Appendix B show the results of the GPR survey for the eastbound and westbound directions of Sections 1 and 2. The data are shown for lanes 0, 1, and 2. By visual inspection of Figures B.1 through B.10, it can be seen that although the majority of the overlay is equal to or greater than the as-designed thickness, there are some segments that show an overlay that was thinner than expected (coring was not performed to confirm the GPR survey results).

Figures B.3, B.5, B.7, and B.9 in Appendix B show a deep repair at approximately Milepost 199.2 in both directions of lanes 1 and 2 in Section 1. It is unclear if this action was the result of a repair from a previous project or the current rehabilitation effort. In addition, certain figures in Appendix B show that the HMA overlay became much thinner before and after certain bridges. This is not an unexpected occurrence; at several bridges within the project limits the overlay is tapered to where there is no overlay directly underneath the bridge because of concerns of vertical clearance.

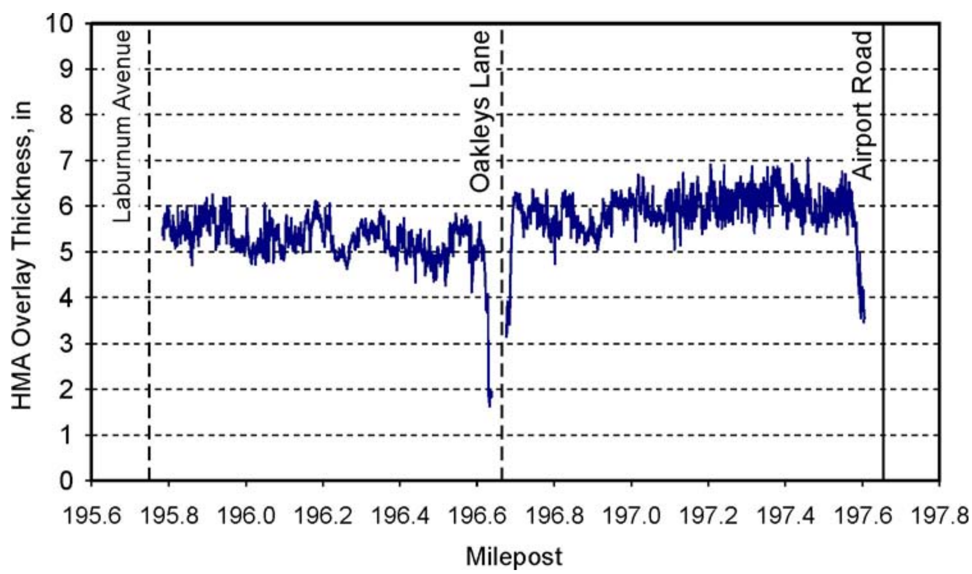

Figure 2. GPR Results From Laburnum Avenue to Airport Road (Eastbound, Lane 1). The approximate locations of these structures are indicated by vertical lines; a solid vertical line indicates I-64 is carried over another roadway; a dashed vertical line indicates that I-64 passes beneath another roadway.

Time Required to Perform the Rehabilitation

The construction schedules for Sections 1 and 2 (combined) and Section 3 were assessed to determine the time required to perform the pavement rehabilitation. Sections 1 and 2 were considered together as the contract documents grouped these two sections into one construction project. Section 1 is in a section of I-64 that consists of 6 lanes (3 in each direction). By multiplying the number of lanes by the project length, the number of lane-miles rehabilitated was calculated to be 29.52. The number of lane-miles for Section 2 (having 2 lanes per direction) was similarly calculated to be 19.24. The sum of the lane-miles for Sections 1 and 2 was calculated as 48.76. The number of lane-miles for Section 3 (also having 2 lanes per direction) was calculated to be 65.60.

Figure 3 shows the construction schedule for pavement-related items occurring in Sections 1 and 2. The figure shows that this construction started during the last week of April 2005 and was completed during the last week of November 2007; the resulting duration was 31 months. Figure 4 shows the construction schedule for pavement-related items occurring in Section 3. The figure shows that this construction started during the first week of May 2005 and was completed during the last week of November 2007; the resulting duration was slightly less than 31 months. By comparing Figures 3 and 4 it can be seen that the project durations were nearly identical.

Cost per Lane-mile Rehabilitated

Since the rehabilitation work completed in Sections 1 and 2 and in Section 3 was of differing lengths, a direct comparison of project costs was not valid. Therefore, a comparison of project costs based on the number of lane-miles was required. The project costs (including any adjustments for asphalt materials) for Sections 1 and 2 were $25,813,909 and Section 3 was $27,622,656. Given a lane mileage of 48.76 for Sections 1 and 2 and a lane mileage of 65.60 for Section 3, the cost per lane-mile to rehabilitate the pavement in Sections 1 and 2 and Section 3 was calculated to be $529,407 and $421,077, respectively, as shown in Table 13.

From Table 13 it can be seen that the cost to rehabilitate Sections 1 and 2 was approximately $108,000 per lane-mile more than the cost to rehabilitate Section 3. Table 13, however, does not indicate if the treatments performed in Sections 1 and 2 versus those performed in Section 3 resulted in pavements with a similar structural capacity or service life.

Traffic Volume and Truck Loading

The traffic volume, expressed as annual average daily traffic (AADT), is routinely collected by VDOT on segments of the interstate. Tables 14 and 15 show the 2007 AADT for Sections 1, 2, and 3 in the eastbound and westbound directions, respectively (VDOT, 2007b). As shown, the single-direction AADT ranged from 17,000 to 48,000 vehicles per day and the percent trucks varied between 5% and 14%.

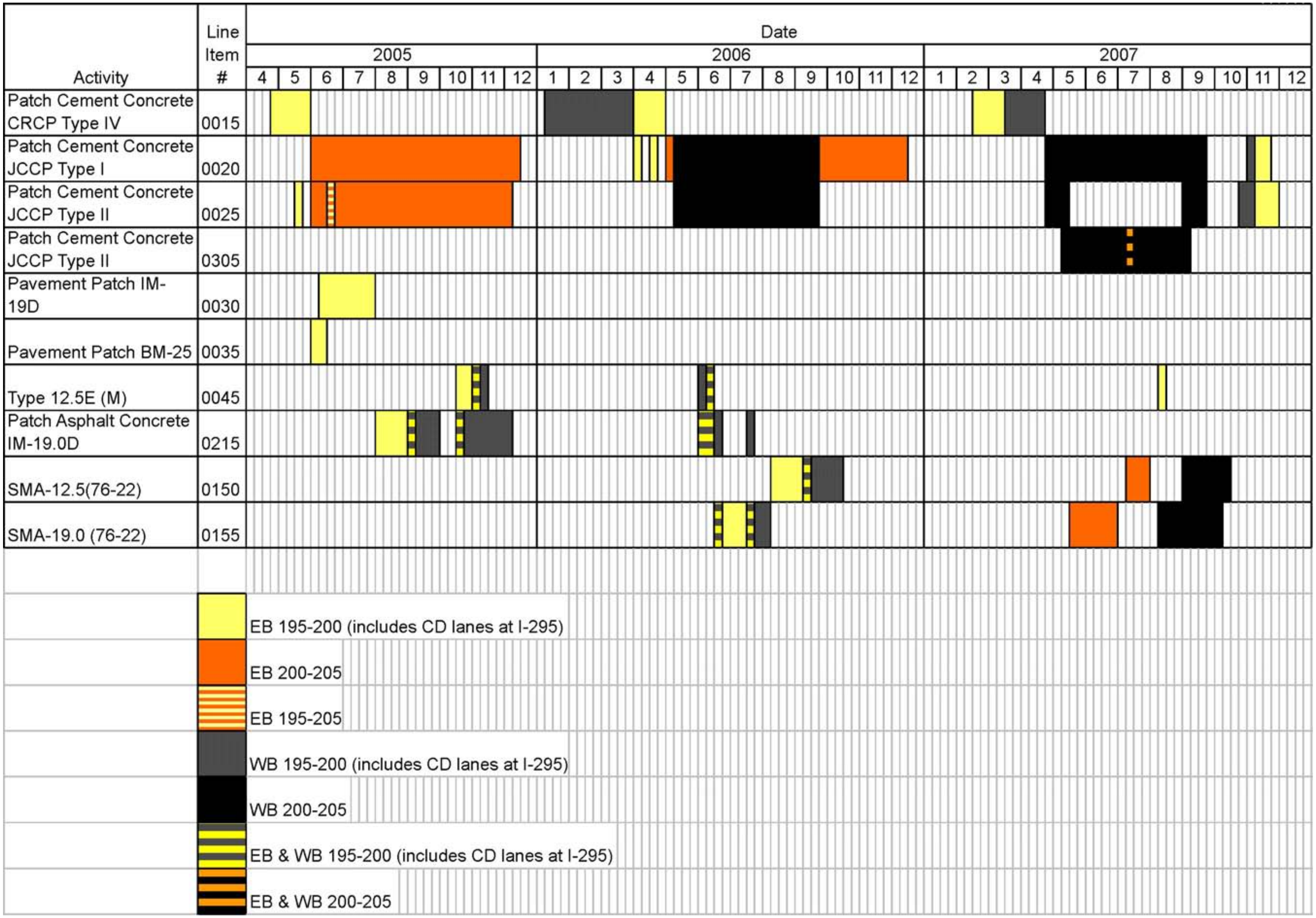

Figure 3. Construction Schedule for Sections 1 and 2 (Mileposts 195-205)

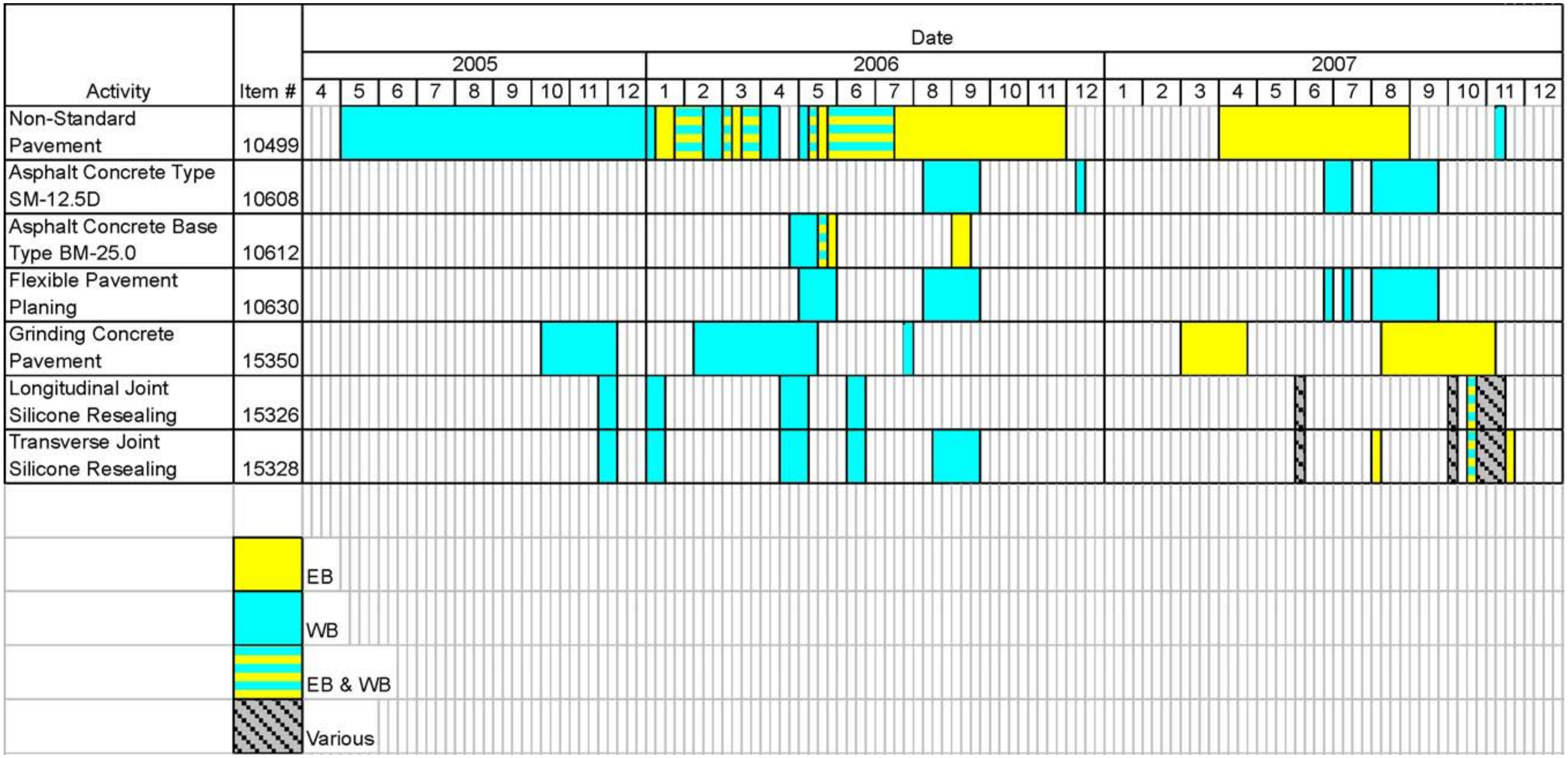

Figure 4. Construction Schedule for Section 3 (Mileposts 239-255)

Table 13. Cost per Lane-mile Rehabilitated

Section	Distance, mi	Lane Mileage	Project Cost (including asphalt adjustment), $	Cost per Lane-mile, $
1 and 2	9.73	48.76	25,813,909	529,407
3	16.4	65.60	27,622,656	421,077

Table 14. 2007 Traffic Volume and Percent Trucks for Eastbound I-64

Section No.	Location		AADT[a]	Overall % Trucks	% Single-Unit Trucks	% Tractor-Trailer Trucks
	From	To				
1	Laburnum Avenue	SR 156, Airport Road	24,000	6	2	4
	SR 156, Airport Road	I-295	17,000	10	3	7
2	I-295	SR 33, Bottoms Bridge	34,000	10	2	8
3	SR 143, Camp Perry Road	SR 199 East, Humelsine Parkway	30,000	10	3	7
	SR 199 East, Humelsine Parkway	US 60, Pocahontas Trail / SR 143, Merrimac Trail	40,000	10	2	8
	US 60, Pocahontas Trail / SR 143, Merrimac Trail	SR 143 Merrimac Trail	38,000	5	2	3
	SR 143 Merrimac Trail	SR 238, Yorktown Road	38,000	5	2	3
	SR 238, Yorktown Road	SR 105, Ft Eustis Boulevard	42,000	5	2	3
	SR 105, Ft Eustis Boulevard	SR 143, Jefferson Avenue	46,000	5	2	3

[a] AADT = annual average daily traffic (vehicles/day).

Table 15. 2007 Traffic volume and percent trucks for Westbound I-64

Section No.	Location		AADT[a]	Overall % Trucks	% Single-Unit Trucks	% Tractor-Trailer Trucks
	From	To				
1	Laburnum Avenue	SR 156, Airport Road	25000	5	2	3
	SR 156, Airport Road	I-295	17000	9	3	6
2	I-295	SR 33, Bottoms Bridge	36000	10	2	8
3	SR 143, Camp Perry Road	SR 199 East, Humelsine Parkway	30000	14	2	12
	SR 199 East, Humelsine Parkway	US 60, Pocahontas Trail / SR 143, Merrimac Trail	38000	5	1	4
	US 60, Pocahontas Trail / SR 143, Merrimac Trail	SR 143 Merrimac Trail	40000	5	1	4
	SR 143 Merrimac Trail	SR 238, Yorktown Road	40000	5	1	4
	SR 238, Yorktown Road	SR 105, Ft Eustis Boulevard	43000	5	2	3
	SR 105, Ft Eustis Boulevard	SR 143, Jefferson Avenue	48000	5	2	3

[a] AADT = annual average daily traffic (vehicles/day).

From the AADT and percent truck data shown in Tables 14 and 15, the number of trucks per day can be determined. VDOT's current design procedure uses the equivalent single-axle load (ESAL) quantity to determine the amount of pavement damage produced by various vehicles (VDOT, 2000). Given that the ESAL factor assigned to trucks is several thousand times greater than that for passenger vehicles, it is not uncommon to consider only the number of trucks and not the entire AADT when assessing the number of loads carried by a particular pavement.

The number of trucks for each section, shown in Tables 16 and 17 for the eastbound and westbound directions, respectively, was determined using the AADT and percent truck data shown in Tables 14 and 15. The average number of trucks in each section was weighted by

Table 16. Weighted Average Number of Trucks for Eastbound I-64 (Based on 2007 AADT)

Section No.	Location		Link Length, mi	Weighted Average No. of Single-Unit Trucks	Weighted Average No. of Tractor-Trailer Trucks
	From	To			
1	Laburnum Avenue	SR 156, Airport Road	1.88	516	1,163
	SR 156, Airport Road	I-295	4.07		
2	I-295	SR 33, Bottoms Bridge	4.14	680	2,720
3	SR 143, Camp Perry Road	SR 199 East, Humelsine Parkway	3.44	878	1,720
	SR 199 East, Humelsine Parkway	US 60, Pocahontas Trail / SR 143, Merrimac Trail	1.62		
	US 60, Pocahontas Trail / SR 143, Merrimac Trail	SR 143 Merrimac Trail	2.63		
	SR 143 Merrimac Trail	SR 238, Yorktown Road	1.62		
	SR 238, Yorktown Road	SR 105, Ft Eustis Boulevard	2.04		
	SR 105, Ft Eustis Boulevard	SR 143, Jefferson Avenue	5.03		

Table 17. Weighted Average Number of Trucks for Westbound I-64 (based on 2007 AADT)

Section No.	Location		Link Length, mi	Weighted Average No. of Single-Unit Trucks	Weighted Average No. of Tractor-Trailer Trucks
	From	To			
1	Laburnum Avenue	SR 156, Airport Road	1.99	525	979
	SR 156, Airport Road	I-295	3.62		
2	I-295	SR 33, Bottoms Bridge	4.07	720	2,880
3	SR 143, Camp Perry Road	SR 199 East, Humelsine Parkway	3.31	712	2,071
	SR 199 East, Humelsine Parkway	US 60, Pocahontas Trail / SR 143, Merrimac Trail	1.41		
	US 60, Pocahontas Trail / SR 143, Merrimac Trail	SR 143 Merrimac Trail	2.72		
	SR 143 Merrimac Trail	SR 238, Yorktown Road	1.34		
	SR 238, Yorktown Road	SR 105, Ft Eustis Boulevard	2.32		
	SR 105, Ft Eustis Boulevard	SR 143, Jefferson Avenue	5.22		

multiplying the link length by the AADT by the percent trucks for each link, summing the values in each section, and then dividing by the total length of each section. As Tables 16 and 17 show, the weighted average number of trucks varied significantly among the three sections, and any future analysis of the service life of these three sections should be based on the number of trucks carried rather than the length of time in-service.

Summary of Findings

Literature Review

- Pre-overlay repair of existing PCC pavements by using PCC patching is a preferable method to using HMA patching, and PCC grinding is a viable rehabilitation alternative for jointed concrete pavements.

- Reflection cracking is the predominant mode of failure for an asphalt overlay placed on top of a jointed concrete pavement.

- Overlays thicker than 3 in offer only a marginal improvement in reducing the transverse reflection cracking development rate with increasing overlay thickness.

- For jointed concrete pavements receiving grinding and non-overlay repair as a method of rehabilitation, the pavement smoothness decreased faster than for jointed concrete pavements that received overlay repair as a method of rehabilitation.

- Smoother pavements can potentially have an increased service life. In addition, pavements that are initially smoother are more likely to remain smoother throughout their service life.

Rehabilitated Sections of I-64

- Analysis of the IRI data before and after repairs showed that the ride quality of all pavement sections improved after repairs. The IRI values were currently within the limits set forth at the initiation of this study. The average IRI values for Sections 1 and 2 were currently less than those for Section 3.

- The average skid resistance values for each section showed that the repairs did not negatively affect the average skid resistance of the pavement.

- An examination of the construction schedule showed that the time to complete the pavement rehabilitation was nearly identical when comparing Section 1 and 2 versus Section 3.

- The average cost per lane-mile rehabilitated was approximately $515,200 for Sections 1 and 2 combined and approximately $417,000 for Section 3. Although this cost difference may seem great, the results of this study do not allow for a determination of any difference in structural capacity or service life among sections.

- GPR testing showed that the thickness of the HMA overlay in the majority of the pavement in Sections 1 and 2 was equal to or slightly greater than the as-designed thickness. However, areas exist where the thickness of the HMA overlay appears to be less than the as-designed thickness.

CONCLUSION

- *To date, the pavement in all three sections of I-64 examined in this study is performing satisfactorily.* The data provided in this study will be important in future studies to assess the costs and benefits of the rehabilitation treatments used.

RECOMMENDATIONS

1. *VDOT's Materials Division should annually monitor the ride quality of the pavement in Sections 1, 2, and 3 of this study and monitor the rate of transverse crack development in Sections 1 and 2.* This information can be obtained from annual distress data currently collected by VDOT's Maintenance Division. Doing so would allow for an estimation of the service life for the various repair options. Given the difference in the number of trucks carried per section, the service life should be calculated in terms of number of trucks carried rather than years of service, making the result more applicable to other locations.

2. *The Virginia Transportation Research Council, in collaboration with other research organizations, should encourage and pursue full-scale and/or laboratory-scale accelerated pavement testing to determine optimum repair materials and methods for pre-overlay repair of existing PCC pavements and to develop models to quantify the deterioration of an asphalt overlay placed over an existing concrete pavement because of reflection cracking.*

COSTS AND BENEFITS ASSESSMENT

The experimental results and the literature review did not provide enough information to allow an estimation or comparison of the benefits of the three rehabilitation methods used. This is in part attributable to the lack of a scientific model to quantify the deterioration of an asphalt overlay placed over an existing concrete pavement by reflection cracking. This lack of an available model does not allow for a comparison to be made in order to estimate or project the anticipated service life of Section 1 versus Section 2 or to compare the loss of smoothness in Sections 1 and 2 versus Section 3.

If additional or more detailed construction schedule data were available, the various repair methods could be compared based on lane closure time as a cost to the traveling public. However, using the information available for this study, the construction times were nearly identical when comparing Sections 1 and 2 versus Section 3 despite the fact that Section 3 encompassed nearly 35% more lane mileage that Sections 1 and 2.

The pavement rehabilitation cost per lane-mile was nearly 20% less for Section 3 as compared to Sections 1 and 2. However, the experimental results do not allow for a comparison to determine any differences in structural capacity or service life among the sections.

ACKNOWLEDGMENTS

The authors acknowledge the assistance of Trenton Clark, Affan Habib, and Sean Nelson, VDOT Materials Division; Fred Durr and Tom Tate, VDOT's Hampton Roads District; Frank Wiles and Mike Wells, VDOT's Richmond District; Stacey Diefenderfer, Kevin McGhee, and Celik Ozyildirim, VTRC; Bob Long, American Concrete Pavement Association; and Richard Schreck, Virginia Asphalt Association. Thanks also go to Jim Gillespie, VTRC, for his assistance with the cost and benefits analysis. The authors thank Randy Combs, Ed Deasy, and Linda Evans of VTRC for their assistance with the graphics and editorial process.

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APPENDIX A

PAVEMENT SMOOTHNESS TESTING RESULTS

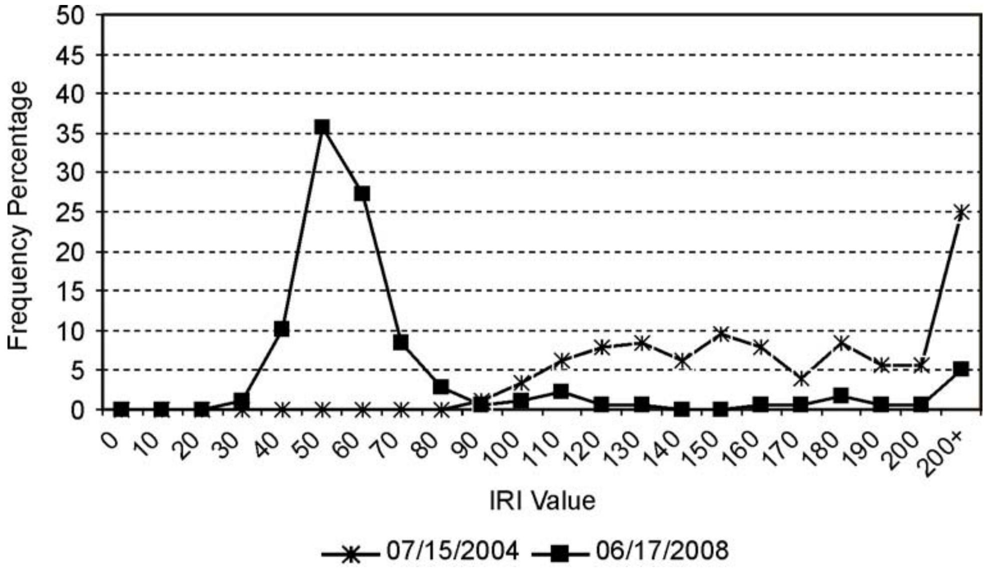

Figure A.1. Distribution of International Roughness Index Before (7/15/2004) and After (6/17/2008) Construction (Section 1, Eastbound Lane 1)

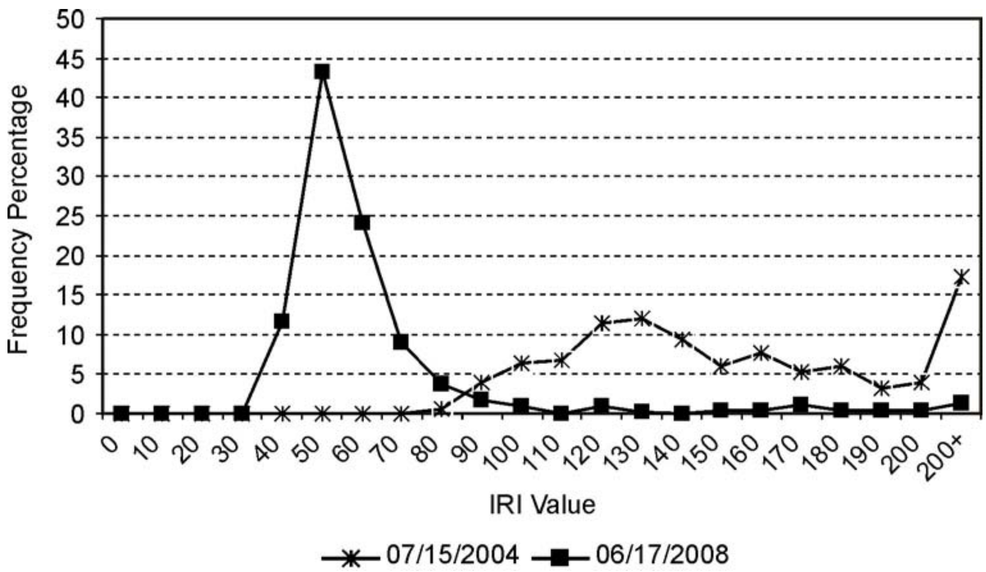

Figure A.2. Distribution of International Roughness Index Before (7/15/2004) and After (6/17/2008) Construction (Section 1, Eastbound Lane 2)

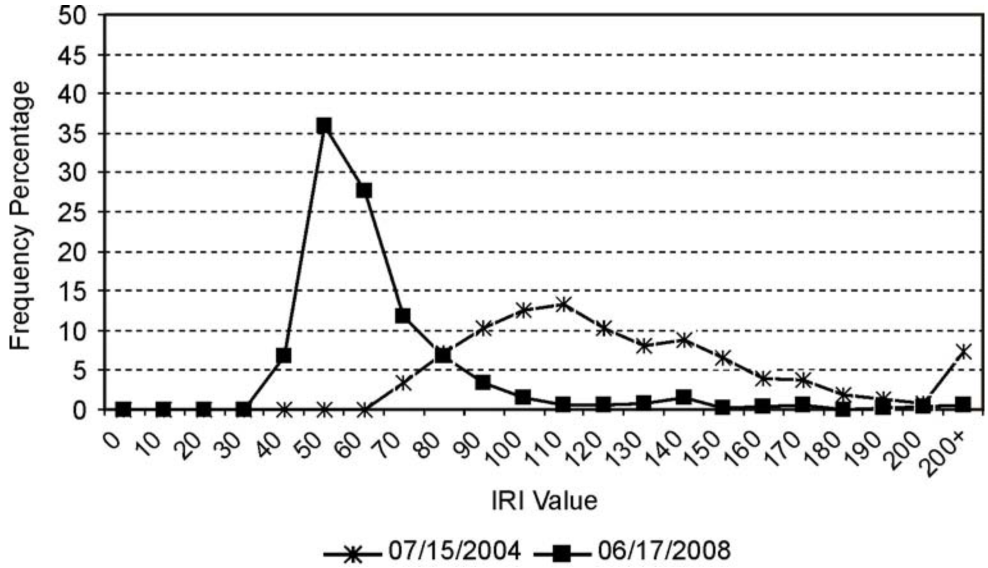

Figure A.3. Distribution of International Roughness Index Before (7/15/2004) and After (6/17/2008) Construction (Section 1, Eastbound Lane 3)

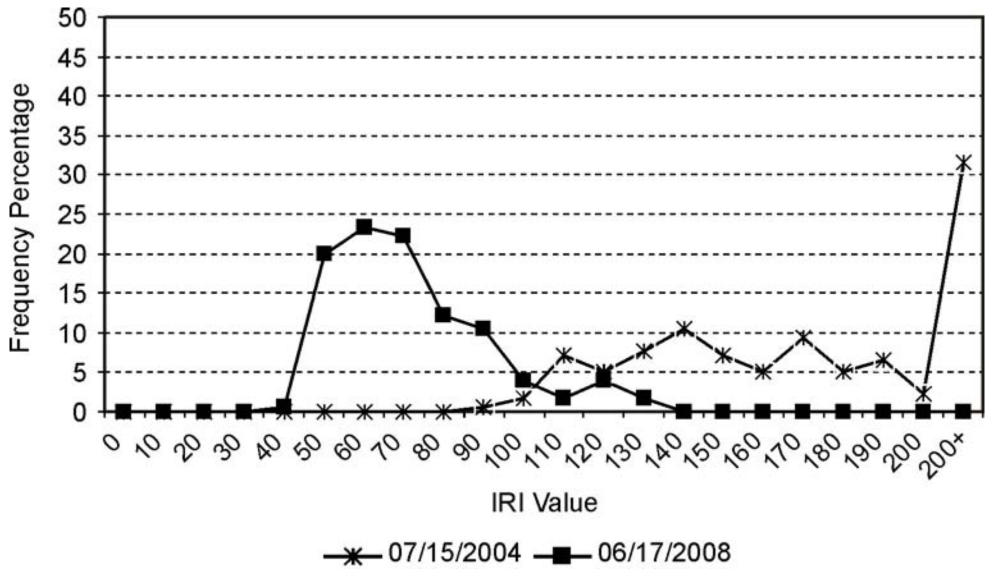

Figure A.4. Distribution of International Roughness Index Before (7/15/2004) and After (6/17/2008) Construction (Section 1, Westbound Lane 1)

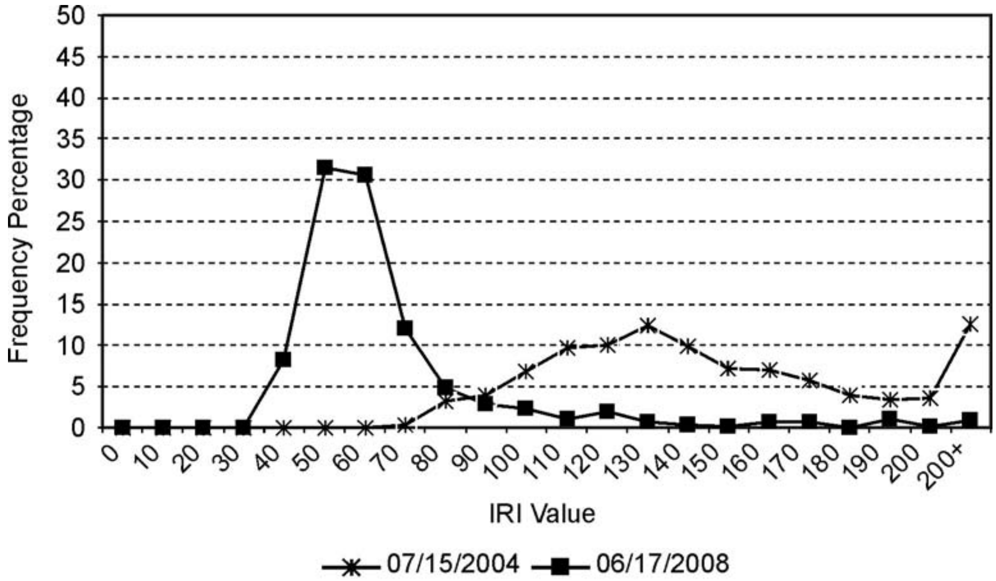

Figure A.5. Distribution of International Roughness Index Before (7/15/2004) and After (6/17/2008) Construction (Section 1, Westbound Lane 2)

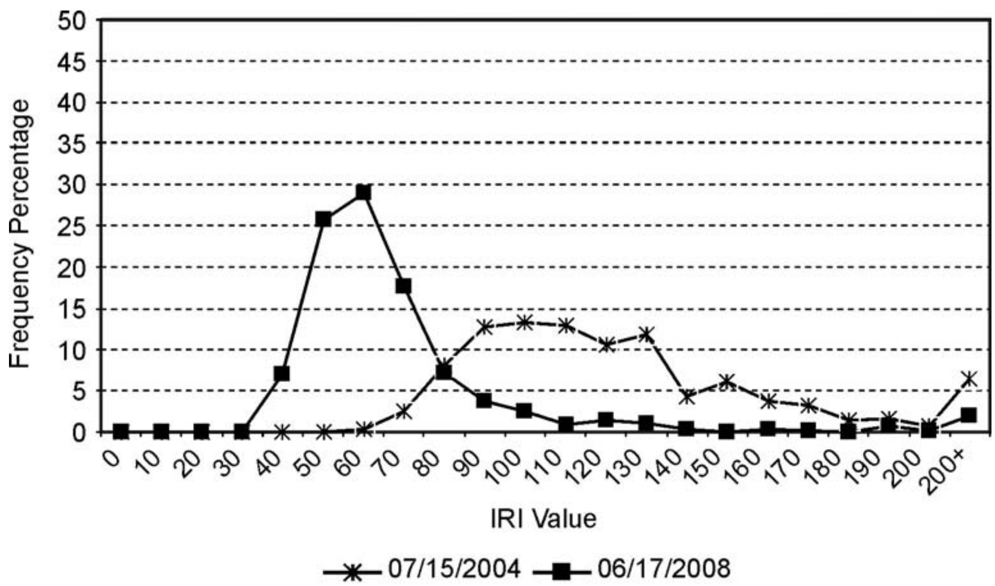

Figure A.6. Distribution of International Roughness Index Before (7/15/2004) and After (6/17/2008) Construction (Section 1, Westbound Lane 3)

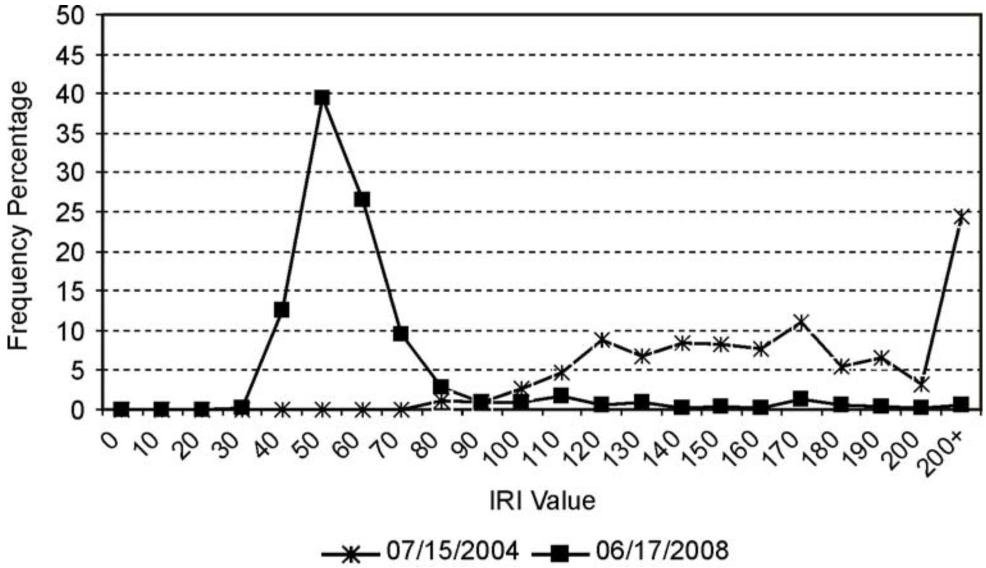

Figure A.7. Distribution of International Roughness Index Before (7/15/2004) and After (6/17/2008) Construction (Section 2, Eastbound Lane 1)

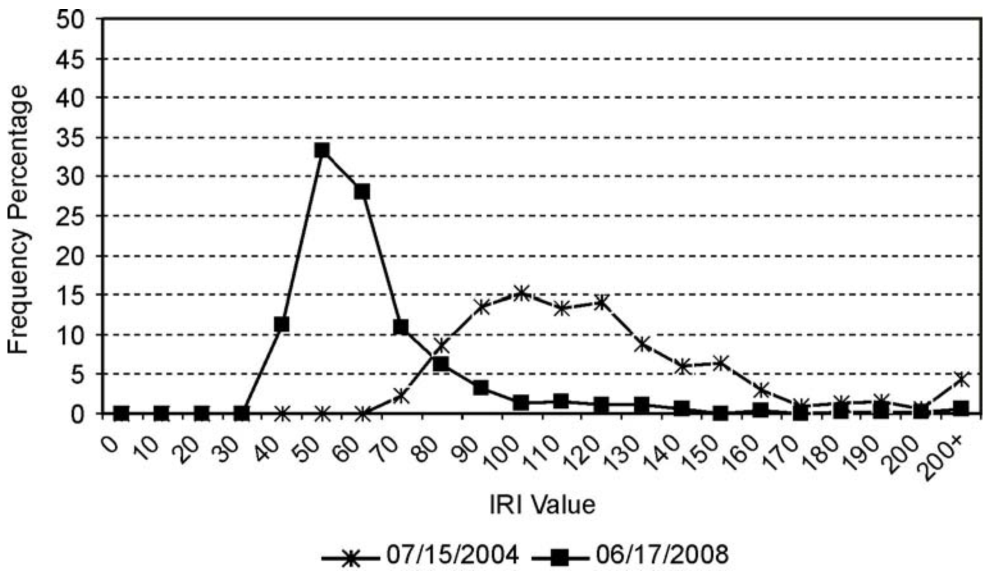

Figure A.8. Distribution of International Roughness Index Before (7/15/2004) and After (6/17/2008) Construction (Section 2, Eastbound Lane 2)

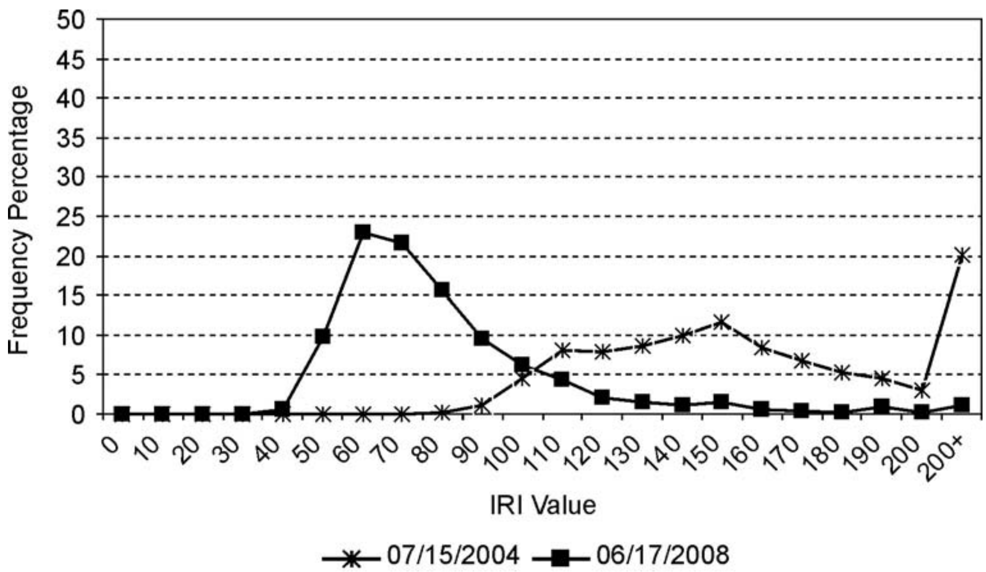

Figure A.9. Distribution of International Roughness Index Before (7/15/2004) and After (6/17/2008) Construction (Section 2, Westbound Lane 1)

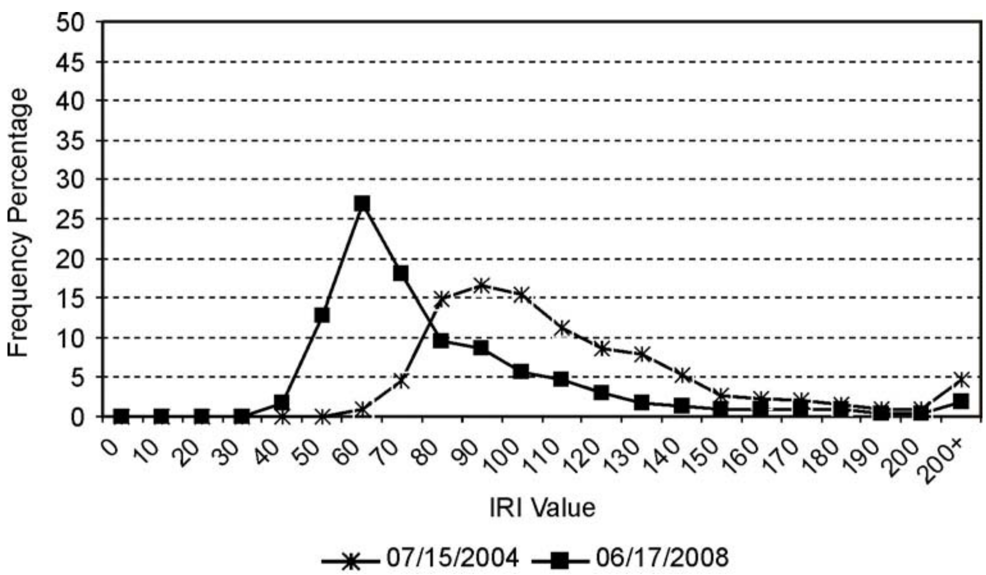

Figure A.10. Distribution of International Roughness Index Before (7/15/2004) and After (6/17/2008) Construction (Section 2, Westbound Lane 2)

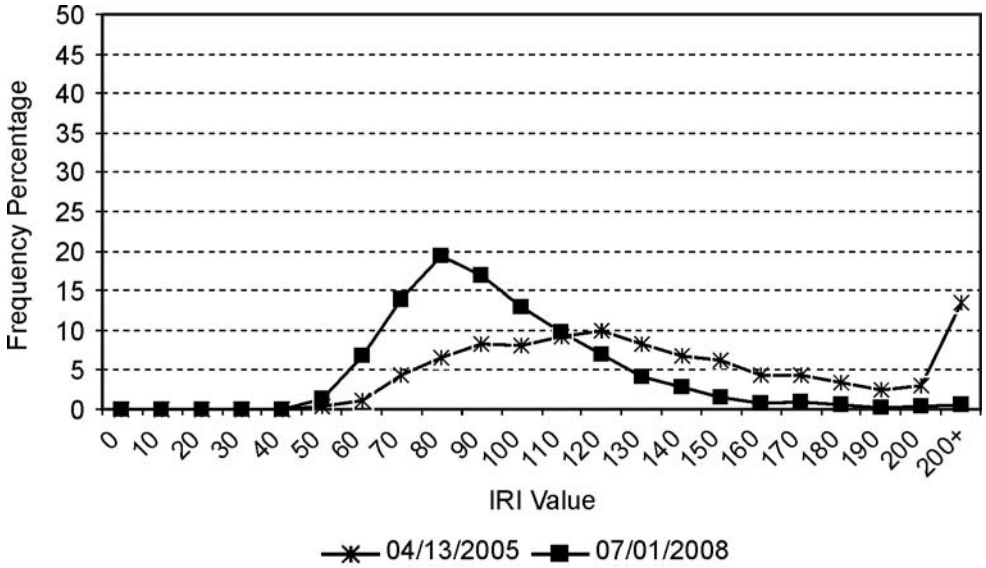

Figure A.11. Distribution of International Roughness Index Before (4/13/2004) and After (7/1/2008) Construction (Section 3, Eastbound Lane 1)

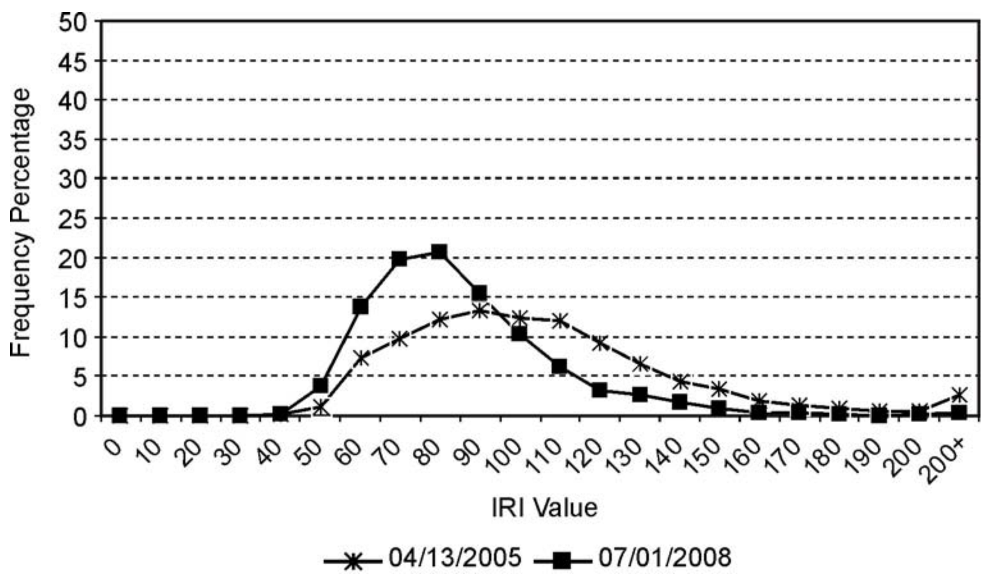

Figure A.12. Distribution of International Roughness Index Before (4/13/2004) and After (7/1/2008) Construction (Section 3, Eastbound Lane 2)

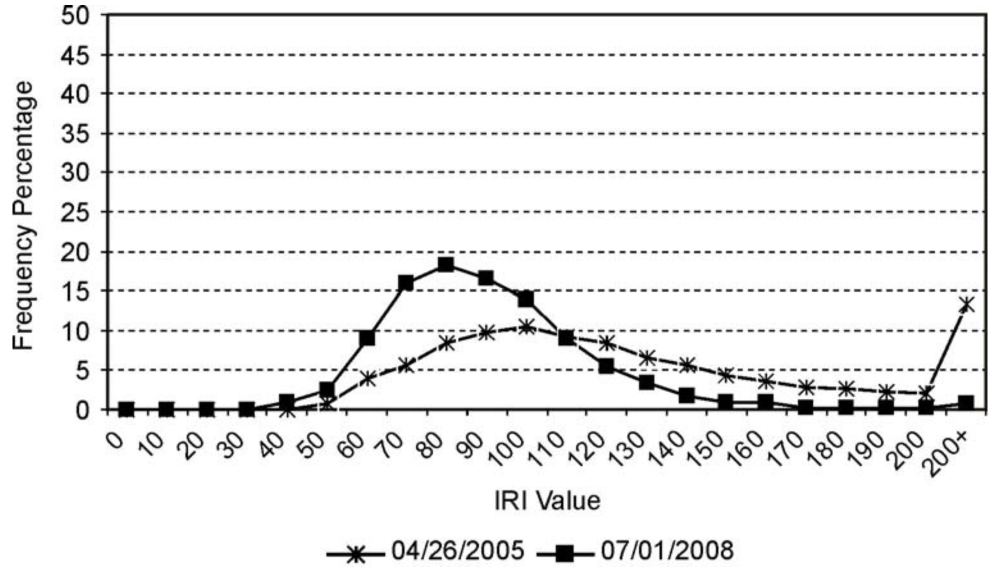

Figure A.13. Distribution of International Roughness Index Before (4/26/2005) and After (7/1/2008) Construction (Section 3, Westbound Lane 1)

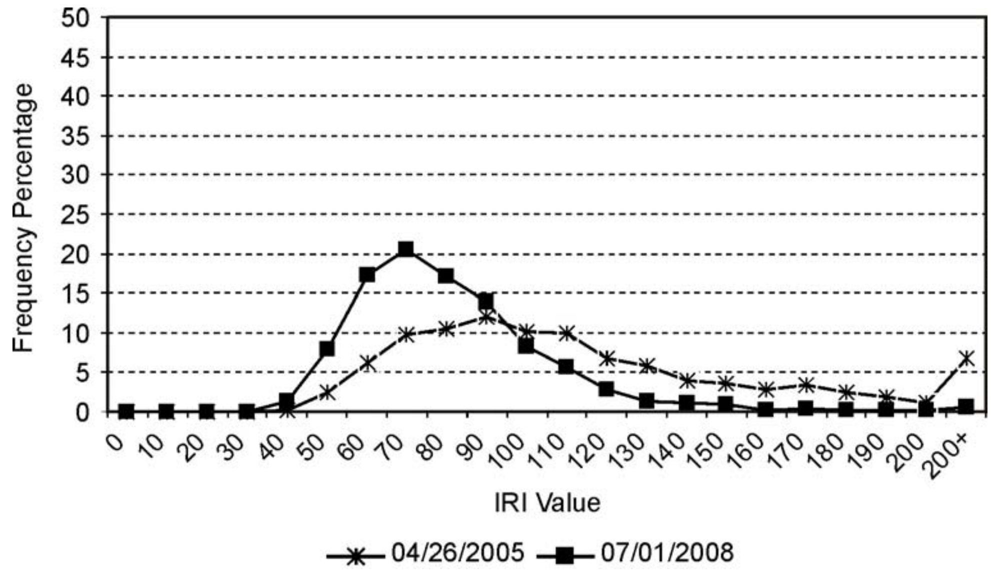

Figure A.14. Distribution of International Roughness Index Before (4/26/2005) and After (7/1/2008) Construction (Section 3, Westbound Lane 2)

APPENDIX B

GROUND-PENETRATING RADAR TESTING RESULTS

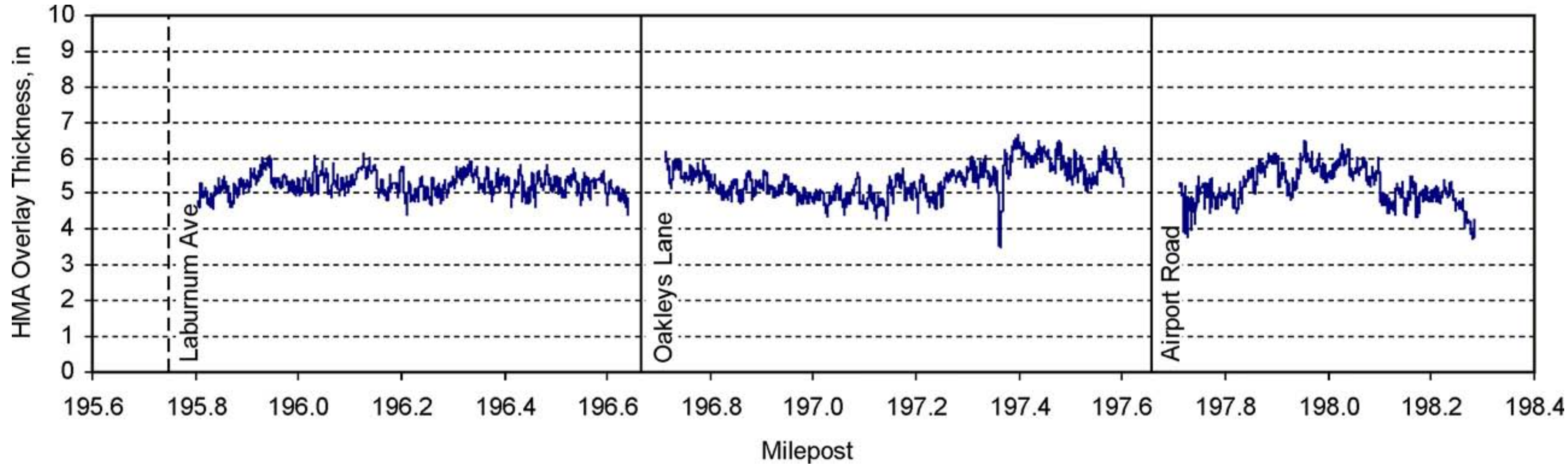

Figure B.1. HMA Overlay Thickness from Ground-Penetrating Radar Testing (Eastbound, Section 1, Lane 0)

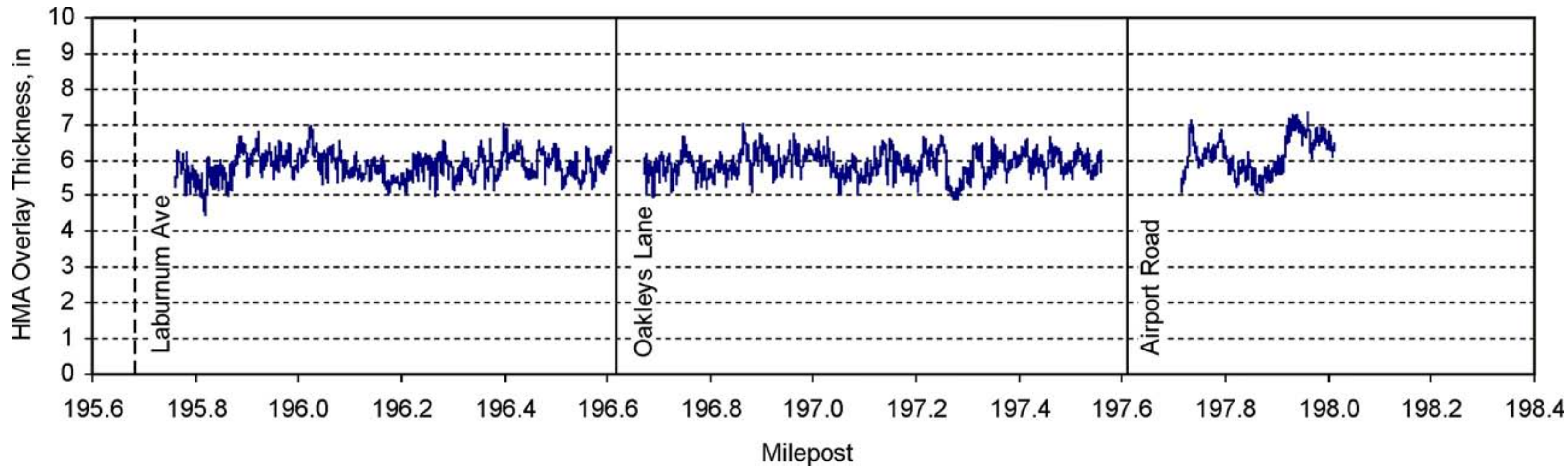

Figure B.2. HMA Overlay Thickness from Ground-Penetrating Radar Testing (Westbound, Section 1, Lane 0)

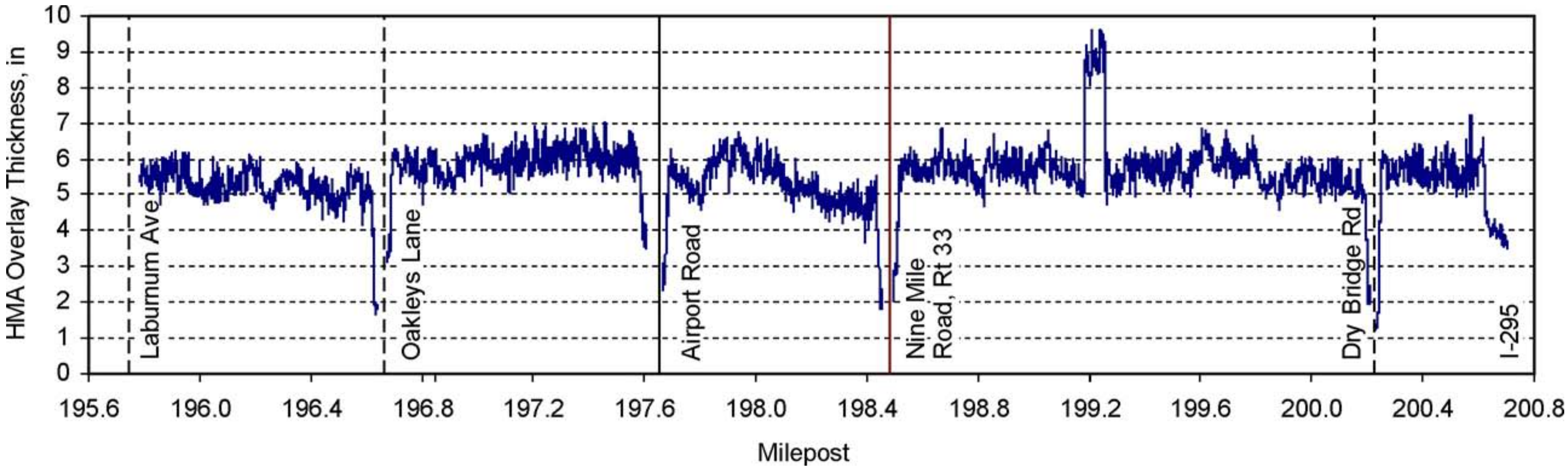

Figure B.3. HMA Overlay Thickness from Ground-Penetrating Radar Testing (Eastbound, Section 1, Lane 1)

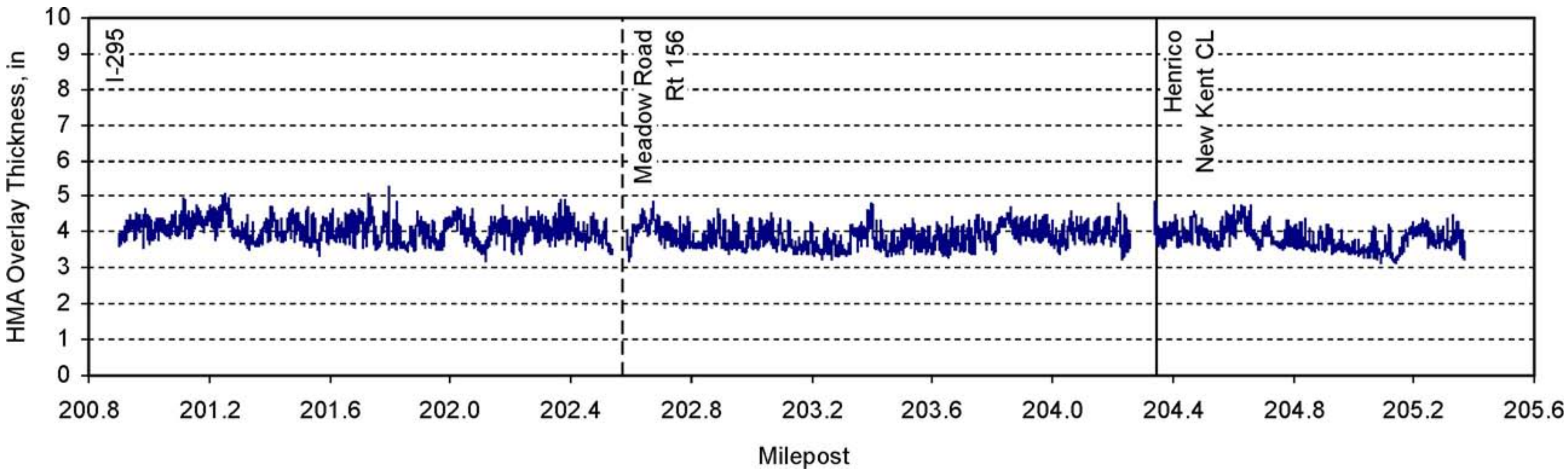

Figure B.4. HMA Overlay Thickness from Ground-Penetrating Radar Testing (Eastbound, Section 2, Lane 1)

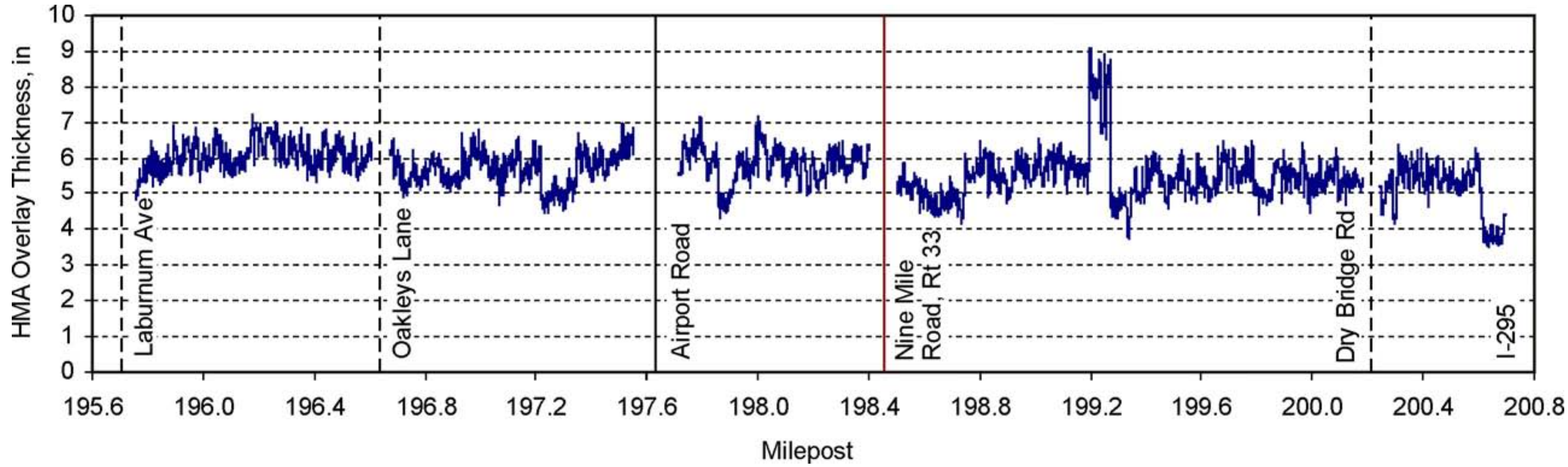

Figure B.5. HMA Overlay Thickness from Ground-Penetrating Radar Testing (Westbound, Section 1, Lane 1)

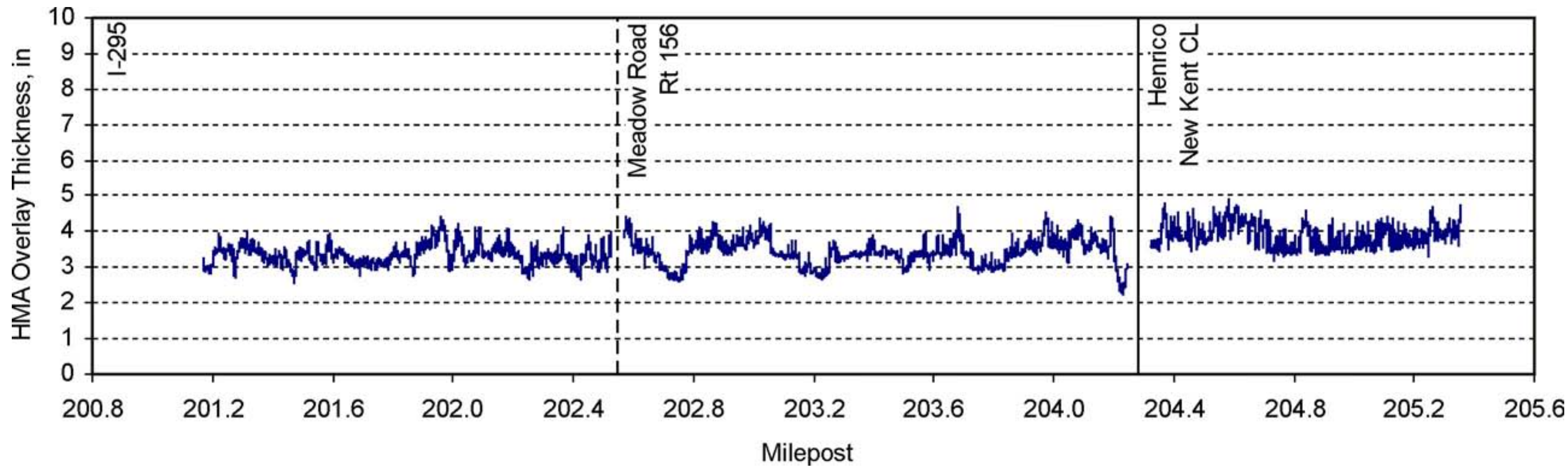

Figure B.6. HMA Overlay Thickness from Ground-Penetrating Radar Testing (Westbound, Section 2, Lane 1)

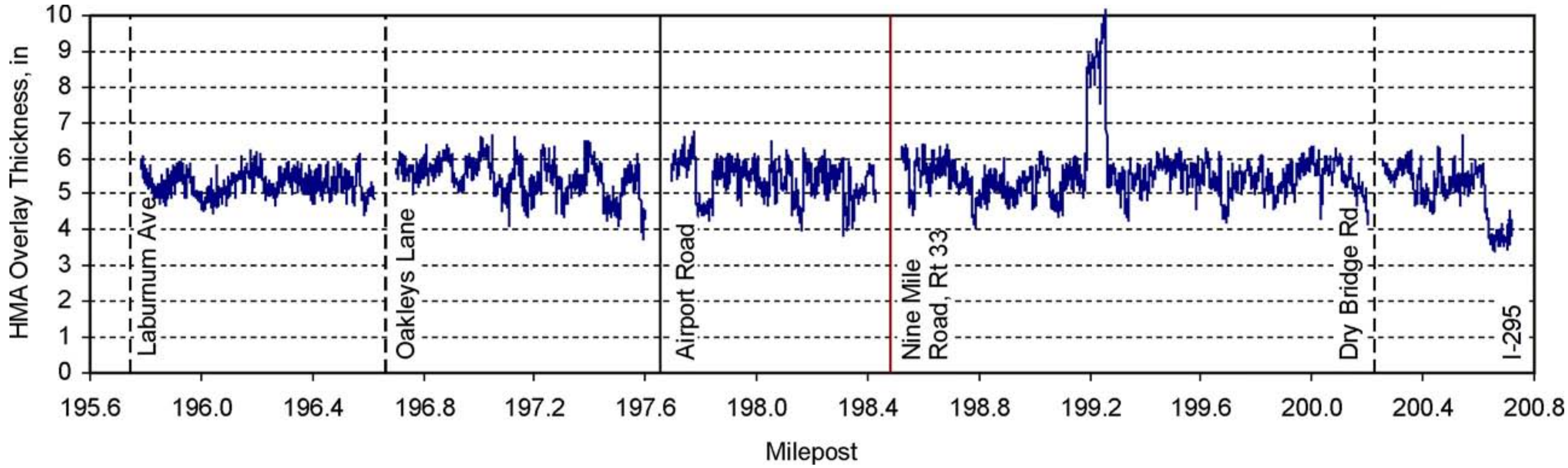

Figure B.7. HMA Overlay Thickness from Ground-Penetrating Radar Testing (Eastbound, Section 1, Lane 2)

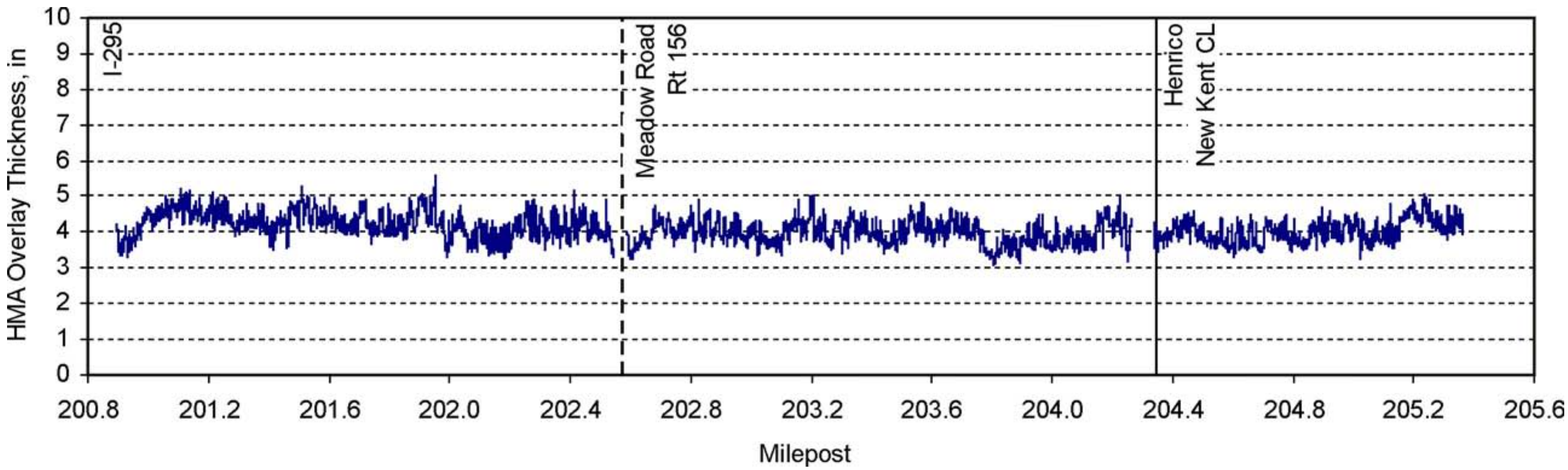

Figure B.8. HMA Overlay Thickness from Ground-Penetrating Radar Testing (Eastbound, Section 2, Lane 2)

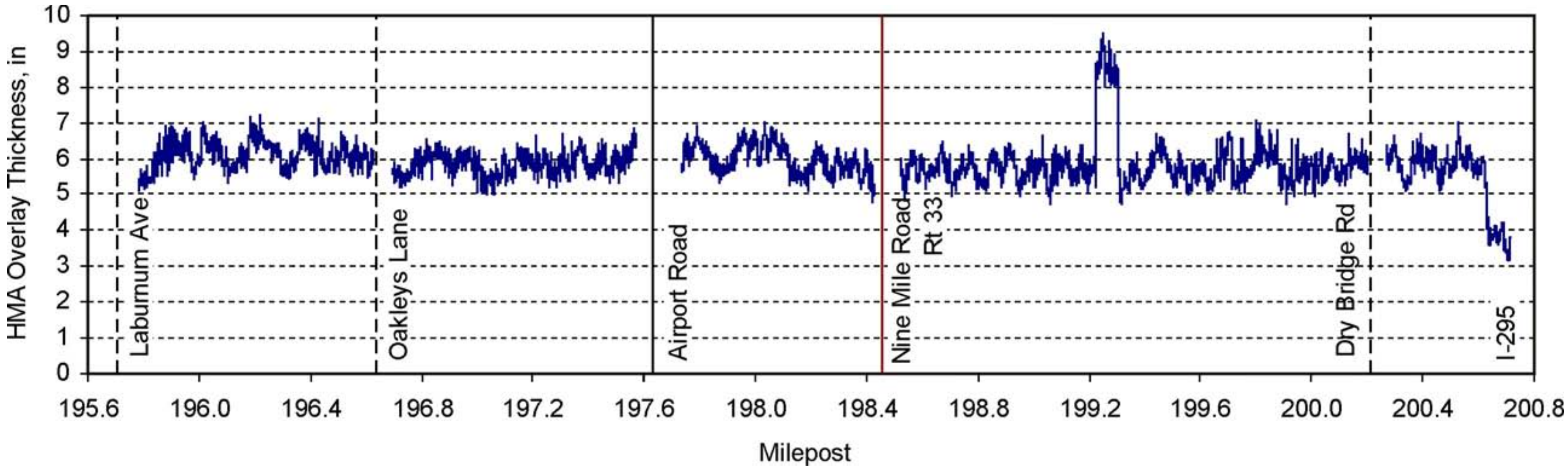

Figure B.9. HMA Overlay Thickness from Ground-Penetrating Radar Testing (Westbound, Section 1, Lane 2)

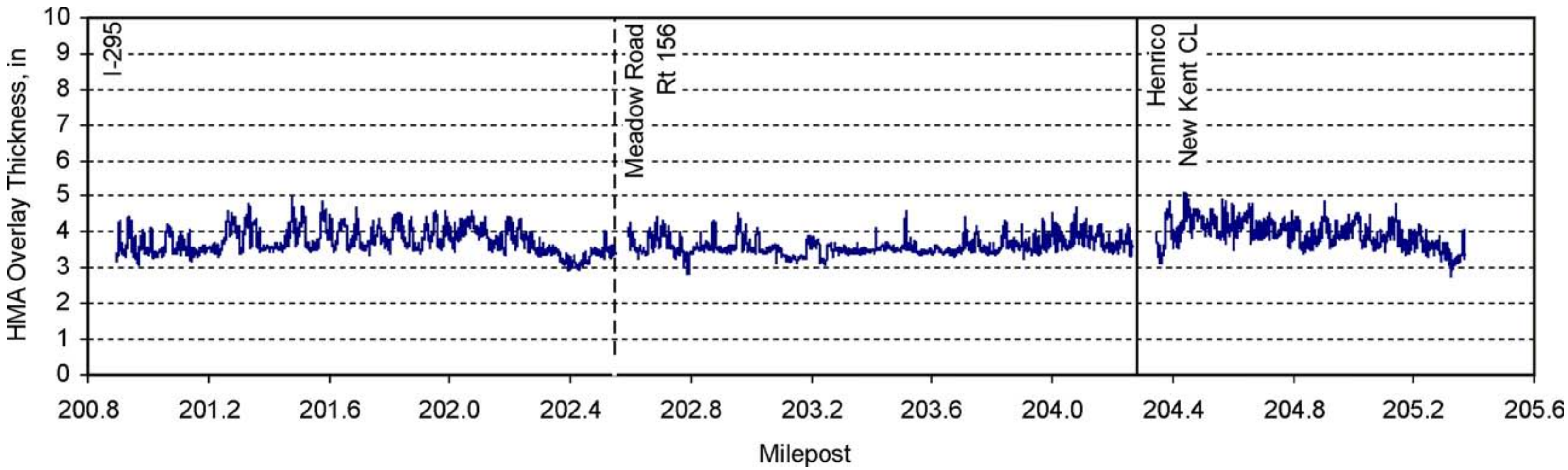

Figure B.10. HMA Overlay Thickness from Ground-Penetrating Radar Testing (Westbound, Section 2, Lane 2)