Improved Performance of JPCP Overlays

Final Report

Submitted to
Michigan Department of Transportation
Bureau of Field Services
Research Administration
8885 Ricks Rd. P.O. Box 30049
Lansing MI 48909

by

Will Hansen (Principal Investigator)
Zhichao Liu (Graduate Research Assistant)

Department of Civil & Environmental Engineering
University of Michigan
2350 Hayward
Ann Arbor, MI 48109-2125

July 2013

1. Report No.
RC-1574

2. Government Accession No.
N/A

3. MDOT Project Manager
Ben Krom

4. Title and Subtitle
Improved Performance of JPCP Overlays

5. Report Date
July 2013

6. Performing Organization Code
N/A

7. Author(s)
Will Hansen and Zhichao Liu
Department of Civil & Environmental Engineering
The University of Michigan

8. Performing Org. Report No.
N/A

9. Performing Organization Name and Address
The Regents of the University of Michigan

10. Work Unit No. (TRAIS)
N/A

11. Contract No.
2009-0747

11(a). Authorization No.
Z 4

12. Sponsoring Agency Name and Address
Michigan Department of Transportation
Bureau of Field Services
Research Administration
8885 Ricks Rd. P.O. Box 30049
Lansing MI 48909

13. Type of Report & Period Covered
Final Report
10/20/2009 to 5/15/2012

14. Sponsoring Agency Code
N/A

15. Supplementary Notes

16. Abstract The overall performance of Michigan concrete overlays has been good. However, some recent JPCP overlay projects have developed premature distress with signs of pumping. It is suspected that lack of drainage was the main cause for the distresses ranging from corner breaks to longitudinal slab cracking originating at the joint. A joint UM-MDOT forensic investigation was initiated in December 2009 to determine the causes for these distresses.

The forensic investigation confirmed that pumping, which is the rapid movement of trapped water from a moving truck axle across a joint, is the major cause of distress, and a result of inadequate drainage. Inadequate drainage was associated with construction related issues ranging from blocking water to reach the drainage trench to omitting the drainage system. Improved drainage solutions were developed jointly.

Finite element analysis (EverFE) predicts that concrete overlays are more sensitive to developing top-down longitudinal slab cracking from loss of joint support, due to the stiffer slab support condition that exists as compared to JPCP on aggregate bases.

Pavement roughness as measured by IRI (International Roughness Index) complemented forensic investigation results that support the pumping mechanism. Two distinctly different performance stages were identified from IRI results. Initially the percentage of good IRI (IRI < 95) for a project length is constant and nearly 100%. This is the period where pumping erosion has little if any structural level effect, but is developing at the sub-structural level. This period is much shorter if drainage is ineffective.

Implementing the findings from this study will enable MDOT to consistently extend the anticipated service life of concrete overlays.

17. Key Words
Concrete overlay pavements, drainage solutions, pumping distress, improved performance

18. Distribution Statement
No restrictions. This document is available to the public through the Michigan Department of Transportation.

19. Security Classification - report	20. Security Classification - page	21. No. of Pages	22. Price
Unclassified	Unclassified	44	N/A

DISCLAIMER

The contents of this report reflect the views of the authors/principal investigator who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the views or policies of the Michigan Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

ACKNOWLEDGEMENTS

The authors would like to thank Michigan Department of Transportation (MDOT) for sponsoring this project and Research Administration for its assistance with the project administration. They wish to thank especially Ben Krom, MDOT (Project Manager) and RAP members (Andy Bennett, Mike J. Eacker, and Dave Phillips) for their guidance and substantial contributions to the report. Field FWD testing carried out by Steve Purdy, MDOT, (RAP member) is greatly appreciated. The team wishes to thank Dave Smiley (retired from MDOT) for his guidance in the early stages of this project. The authors wish to thank Jan Pantolin, UM, for his assistance with field and laboratory testing.

EXECUTIVE SUMMARY

Problem Statement

In 1984 MDOT resumed using concrete overlays with an asphalt interlayer as a rehabilitation treatment for severely distressed concrete pavement. The first overlays were jointed reinforced concrete pavement (JRCP). Since the mid 1990s, overlays have been jointed plain concrete pavement (JPCP), which is a majority of the projects. There have been twenty-four overlay projects since 1984. Table 2.1 contains summary information on the 24 concrete overlay projects that MDOT has constructed since 1984. The summary contains project ID, location, year of construction, interlayer type including open graded and dense graded, drainage system and comments.

Background

The overall performance of concrete overlays in Michigan has been good. However, some recent JPCP projects have developed premature distress with signs of pumping. It was suspected that lack of drainage leading to pumping erosion was involved. If this could be confirmed, then the outcome of this study could help MDOT engineers focus on developing improved drainage solutions that would greatly reduce the likelihood of premature distress, and enable MDOT to extend the average service life of concrete overlays.

A joint UM-MDOT forensic investigation was initiated in 2009 with the major objective to determine the causes of poor performance of some of Michigan's concrete overlays. The major areas of concern included longitudinal cracking starting year one (I-75, West Branch), in the undoweled test sections, and rapid distress development in some recent projects. To accomplish the objectives from a forensic investigation, eight projects were selected based on age and varying performance levels (good to poor). Finite element modeling was used to understand the longitudinal cracking phenomenon found in the I-75 NB, West Branch project, and hot weather construction related early cracking that developed in another project (US-131, Kalamazoo). Laboratory testing was undertaken to assess the frost durability resistance in one project (US-131, Rockford), which had developed joint spalling similar to that found in JPCP projects with poor salt-frost scaling resistance.

Forensic testing consisted of selecting a 500 ft to 1000 ft test section within each project. Measurements included conducting a distress survey, joint faulting in the outer wheel-path, surface profiling, falling weight deflectometer (FWD) testing at outer joint corners and outer wheel-path (OWP), before and after the joint. FWD D0 (at the load-plate) deflections were used to assess pumping erosion. Coring was done to evaluate pumping erosion effects and effectiveness of the drainage system, and whether cracking is bottom up or top down. Cores were brought to the laboratory for air-void analysis and salt frost testing if low air was found and if the project had developed joint spalling consistent with salt-frost deterioration. Drainage pipe outlets were checked for running water.

Major Findings

Pumping related distress was typical for the majority of the field projects. And some projects did not have any drainage system (I-75 NB, West Branch, US-131, Plainwell, US-131, Kalamazoo). In two projects (I-69, NB, Charlotte, and US-23, NB, north of Faussett Rd.) the trench drains were capped with a dense graded interlayer during overlay construction.

Pumping consists of rapid movement of water trapped at joints under the high pressure being generated by the rapid movement of truck axles, resulting in erosion and permanent settlement (faulting) of the downstream (after the joint) slab edge and slab cracking (corner breaks, transverse and longitudinal). Pumping is a result of inadequate drainage.

For I-75 NB West Branch, several construction-related factors were found to be the main cause for pumping: 1) The leaching trench drain was omitted at the outside shoulder during construction as soil boring tests showed sandy soil condition, and the open-graded interlayer associated with overlay construction was found to extend into the shoulder, but the old dense-graded hot mix asphalt (HMA) shoulder was not removed prior to construction of the overlay, thus preventing trapped water from draining vertically. 2)

The concrete overlay in the shoulder was about ¼ inch higher than the outside lane. 3) The outside lane-shoulder joint was left unsealed. Thus, the open-graded interlayer has served as a reservoir for trapped water.

Forensic investigations also show that doweled joints (Section 5, I-75 NB, Westbranch) are more effective in reducing and delaying pumping erosion effects (i.e. joint faulting, slab cracking) consistent with state of the art knowledge (NCHRP 1-37 Design Guide), as dowels provide a condition of uniform slab deflection when subjected to moving axle loads.

Pumping distresses affect ride quality. Increasing distress levels will result in an increase in International Roughness Index (IRI). IRI measurements in the MDOT PMS (Pavement Management System) database complemented forensic investigation results that support the pumping mechanism. Due to a strong correlation between increasing roughness and pumping distress such as joint/crack faulting, the increase in IRI is a good parameter for determining the need for repair. The better the drainage, the longer it takes for IRI to increase to a level where pavement distress develops. This factor can also be used to locate "hot spots" within a project, where pumping is more pronounced, thus requiring attention since IRI is reported every 0.1 mile in the outer wheel-path (OWP) and inner wheel-path (IWP) on a yearly or bi-yearly basis.

Finite element analysis (EverFE) predicts that concrete overlays are more sensitive to slab cracking from loss of joint support due to the stiffer slab support condition that exists as compared to JPCP on aggregate bases. Loss of joint support in concrete overlays is the permanent gap that develops between the slab and the interlayer as a result of pumping erosion due to trapped water and truck wheel loading. This condition is simulated in EverFE by imposing a negative temperature differential between slab top and bottom, which causes a gap from corner uplift. These simulations can explain the top-down cracking associated with joint truck axle loading. These longitudinal cracks originated at the transverse joint in the un-doweled sections of the I-75 NB, West Branch.

A construction issue such as initiating joint sawing as early as possible is especially important for concrete overlays. EverFE finite element analysis predicts that thermal stresses are significant as a result of friction between a continuous concrete slab and the interlayer during slab cooling for a continuous slab condition (i.e. prior to saw cutting). This analysis can explain early-age cracking which occurred in the US-131, Kalamazoo project in 2004.

Laboratory test results for air content and salt frost scaling freeze-thaw resistance are consistent with field results for one project (US-131, Rockford) that show rapid development of joint concrete spalling. For more detail on this type of distress see MDOT research report RC-1534.

Implementation of study findings

This research study achieved its objective of determining the potential causes for the premature deterioration that has developed in some concrete overlay projects. The findings from this study confirmed that pumping erosion was a major factor. Further, this study has provided the understanding and necessary evidence to modify drainage specifications and construction practices to prevent future occurrences of similar premature deterioration caused by pumping erosion within the interlayer.

Pavement monitoring for drainage performance

As a result of this study IRI was found to be an excellent measure for evaluating the effectiveness of a drainage system due to the strong correlation between pumping related distress and increase in pavement roughness over time. It is recommended that MDOT monitor IRI over time for in-service concrete overlays, as a pre-cursor to determining the need for preventive maintenance.

Benefits

Results of the study indicate that if proper consideration is given to ensuring that drainage systems are properly constructed and functioning as intended, the likelihood of premature distress from pumping will be greatly reduced. Implementing the findings from this study will enable MDOT to consistently extend the anticipated service life of unbonded concrete overlays well beyond the design life (20 years).

TABLE OF CONTENTS

List of Figures

List of Tables

1.0 INTRODUCTION

1.1 Problem Statement

Unbonded concrete overlays have become a popular rehabilitation option for restoring a high serviceability rating to a severely distressed concrete pavement. A concrete overlay enhances structural capacity and greatly improves ride quality. A thin HMA separator layer is used between the old and new pavements to allow independent movement of each pavement layer. The overlay thickness is usually 6-8" with a corresponding transverse joint spacing of about 12 ft. The underlying separator and old pavement act as a very stiff base. Thus, the overlay slab is susceptible to high curling and warping stresses with the likely development of mid-slab transverse cracking under commercial truck loading.

In 1984 MDOT resumed using concrete (unbonded) overlays as a rehabilitation treatment for severely distressed concrete pavement. Concrete overlays had been used prior to 1954 to modernize old concrete pavements that were generally geometrically outdated, as well as distressed. The first overlays were jointed reinforced concrete pavement (JRCP). Since the mid 1990s, overlays have been jointed plain concrete pavement (JPCP), which is a majority of the projects. There have been twenty-four overlay projects by contract since 1984. Table 2.2 contains summary information on the 24 unbonded overlay projects that MDOT has constructed since 1984. The summary contains project ID, location, year of construction, interlayer type including open graded and dense graded HMASL, drainage system and comments.

The overall performance of overlays has been good. However, some recent JPCP projects have developed premature distress with signs of pumping. It is suspected that lack of drainage leading to pumping erosion is the major cause. If this could be confirmed, then the outcome of this study could help MDOT engineers focus on developing improved drainage solutions that would greatly reduce the likelihood of premature distress, and enable MDOT to extend the average service life of unbonded concrete overlays.

1.2 Research Objectives

The major objectives are: (1) Determine from a forensic investigation the causes of poor performance of some of Michigan's unbonded concrete overlays. (2) Establish the causes for the longitudinal cracking starting at the transverse joints that is pronounced in two of the five test sections within the NB I-75, West Branch, Demonstration Project. (3) Also, determine from this project the benefits of doweled joints.

1.3 Research Approach

To achieve the major project objectives a number of tasks were undertaken. Eight projects of varying performance levels (good to poor) and age were selected by the project team and RAP for forensic investigation, laboratory testing and data analysis.

1.4 Report Organization

In chapter 2 a summary table lists some major characteristics of Michigan's unbonded concrete overlays. Study findings are presented in Chapter 3. Conclusions and are presented in Chapter 4, and study implementation plan is presented in Chapter 5, followed by references. Appendix A contains a separate report on the major findings from a forensic evaluation of the NB I-75, West Branch overlay. Appendix B contains summary results from air-void analysis.

2.0 METHODOLOGY

2.1 Michigan Overlay Projects

Between 1984 and 2011 a total of twenty-four Michigan concrete overlays have been constructed. Project summary information is found in Table 2.2 at the end of this section. They are chronological, consecutive starting with the oldest. Two projects, one from 1984 (CS 58033 & 58034) and one from 1995 (CS 38103), have since been reconstructed. The last two columns list the drainage feature and comments regarding drainage. It is noteworthy, that several concrete overlay projects either had no drainage system installed, or the paths to drains were blocked in spots as the drainage trench was capped with HMA during construction.

2.2 Field Projects for Detailed Investigation

At the start of this project in 2009, eight overlays were selected by the project team (PI and MDOT RAP) for field study covering a wide range in performance level (good to poor) and age (5 to 24 years). The selected eight projects listed in Table 2.1 are in order of their age, and initial performance rating (good to poor):

Table 2.1 Field project locations and performance

Route	Location	Year built	Performance	Comments
I-96	Portland	1984	Good	Oldest in-service overlay (41 ft JRCP)
WB US-10	Between Bay City and Midland	1990	Poor	sections with drains/no drains & bond breaker vs. rubblized Two undoweled sections replaced
US-131	Plainwell	1998	Fair	No drains & reflected distress from underlying pavement
NB US-23	North Livingst. Co.	1999	Good	Drain outlets cleaned periodically
NB I-69	Charlotte	2000	Fair	Lots of edge faulting, indicating loss of support
US-131	Rockford	2000	Poor	Some materials related distress of joint concrete
NB I-75	West Branch	2003	Fair	Longitudinal cracking in Section 3 starting year one at joint
US-131	Kalamazoo	2004	Good	No drains. Open graded separator layer NB lanes and shoulder. Dense-graded separator layer SB

2.3 Field and Laboratory Testing

Field testing consisted of selecting a 500 ft. to 1000 ft. test section from each of the projects listed above. Measurements included conducting a distress survey, joint faulting in outer wheel-path, surface profiling, falling weight deflectometer (FWD) testing at outer joint corners and outer wheel-path (OWP) before and after the joint. Coring was done to evaluate pumping erosion effects and effectiveness of the drainage system, and whether cracking is bottom up or top down. Drainage pipe outlets (where installed) were checked for running water.

Cores were obtained for air-void analysis based on ASTM C457 method for hardened concrete. Laboratory salt-frost scaling resistance was evaluated for the US-131, Rockford project since joint staining and spalling deterioration was found, which was typical for concrete with poor scaling resistance (inadequate air-void system) and poor joint drainage condition. A detailed description of these laboratory test methods can be found in a previous report (MDOT Research Report RC-1534).

2.4 Data Analysis

Data obtained from falling weight deflectometer (FWD) testing before and after the joint along with surface profile and Georgia Faultmeter results provided information on the extent of pumping (loss of joint support, slab rotation, moisture warping and joint faulting).

MDOT's Pavement Management System (PMS) database contains International Roughness Index (IRI) measurements on MDOT's network. IRI was used to assess increasing severities of distresses over time as the increase in IRI has been found to empirically correlate well to faulting, cracking, spalling and slab support changes (NCHRP Report 1-37a).

Finite element analysis using EverFE (www.civil.umaine.edu/everfe/) was applied in order to help explain certain project specific distresses such as longitudinal slab cracking originating at the transverse joint within a year after construction (I-75 NB, West Branch) and early cracking related to hot weather construction and nighttime cooling (US-131,Kalamazoo).

Table 2.2. Michigan DOT Concrete Overlay Information (B. Krom, MDOT)

ID*	Route	CS	Year Built	Section Details	HMASL Type	Drainage System	Comments
1	I-96 (Portland)	34044	1984	7” JRCP 41’ joints	Bit Mix No 1100T, 35A (3/4”-1.5” thick for crown correction)	12”-24” wide 2NS trench at EOP, w/ 4” wrapped pipe	Old HMA shoulder cut to place drains
3 & 5	US-23	58033 & 58034	1984	7” JRCP 41’ joints	Bit Mix No 1100T, 35A (3/4”-1.5” thick for crown correction)	2NS trench at EOP, w/ 4” wrapped pipe	Old HMA shoulder cut to place drains
7 & 8	US-10/ M-25	09101 & 09042	1990	7” JPCP Variable joint spacing (12’-17’)	Bit Mix No 1100T, 35A (1”-1.75” thick for crown correction) None over rubblized	Varies: none, 18” PDS, 4” subbase underdrain; all at EOP	Includes many test sections w/ different combinations
9	I-96 (Wacousta)	19022	1991	7.5” JRCP 27’ joints	Bit Mix No 1100L, 20AA (1.25”-1.75” thick for crown correction)	PDS at EOP	Water’s path to drains is blocked in spots
11	I-94 (Jackson)	38103	1995	8” JRCP 27’ joints	Bit Mix No 13 (1”-2.5” thick for crown correction)	None	Sandy HMASL
13	US-131 (Plainwell)	03111	1998	7.1” JPCP 13’ joints	Bit Mix No 13A (±1” thick)	None	Crown correction w/ concrete
15	US-23	47014	1999	7.9” JPCP 13’ joints	Bit Mix No 13A (±1” thick)	18” PDS at EOP; 3”-6” wide trench, backfilled w/ 34R	Path to drains capped w/ HMA during const.
17	I-69 (North of I-94)	13074	1999	7.1” JPCP 13’ joints	Bit Mix No 36A (±1” thick)	None	
19 & 21	I-69 (Charlotte)	13074 & 23061	2000	7.1” JPCP 13’ joints	Bit Mix No 13A (±1” thick)	PDS at EOP (as constructed)	NB PDS capped off in spots

23 & 25	US-131 (Rockford)	41132 & 41133	2000	6.3” JPCP 13’ joints	Bit Mix No 13A (±1” thick)	18” PDS at EOP; 2”-10” wide trench	Plan detail shows paving over drainage trench
27 & 28	US-23	47014	2001	7.1” JPCP 13’ joints	Bit Mix No 36A (±1” thick)	18” PDS at EOP; 3”-6” wide trench	“34-R shall be constructed to top of HMASL”
29	I-75 NB (West Branch)	65041	2003	6” JPCP 10’/12’ joints	Open-Graded Mix (±1” thick)	Not installed	Drains installed on inside shldr
33 & 35	US-131 (Kalamazoo)	39014 & 03111	2004	6.5” JPCP 12’ joints	Open-Graded Mix on all shoulders and NB mainline & 1 mile of SB; Dense HMA Mix under rest of SB (±1” thick)	None	Some crown correction with HMASL
37	I-96 (Coopersville)	70063	2004	6.5” JPCP 14’ joints	Open-Graded Mix (±1” thick) Also under shoulders	18” PDS at EOP	An old HMA under HMASL
39 & 41	I-75	25032 & 73171	2004 & 2005	7” JPCP 14’ joints	Milled old HMA to obtain cross-slope	None	Median lane was new JPCP
45	I-94	77111	2006	7” JPCP 14’ joints	Open-Graded Mix (±1” thick)	6” Open-Graded Underdrain; 5.5’-12.5’ from EOP	
47 & 49	I-96 (Walker)	70063 & 41026	2006 & 2007	6.5” JPCP 14’ joints	Open-Graded Mix (±1” thick) Also under shoulders	18” PDS at EOP	An old HMA under HMASL
51	US-131	41132	2007 & 2008	6.5” JPCP 12’ joints	Open-Graded Mix (±1” thick) Also under shoulders	18” PDS at EOP	
53	I-75 SB	16091	2008	6” JPCP 12’ joints	Open-Graded Mix (±1” thick) Also under shoulders	Varies: 6” leaching trench or 18” PDS at EOP	

55	I-196	41029	2008 & 2009	6” JPCP 12’ joints	Open-Graded Mix (±1” thick) Also under shoulders	18” PDS at EOP	Some old HMA under HMASL. Areas of CRC & rubblized conc
57	I-75 NB	16091	2008	6” JPCP 12’ joints	Open-Graded Mix (±1” thick) Also under shoulders	Varies: 6” leaching trench or 18” PDS at EOP	
59	I-196	03033	2009	6” JPCP 12’ joints	Open-Graded Mix on outside shoulders (±2” thick); Modified HMASL elsewhere (2”-4.5” thick)	New (NB) & Ex (SB) 6” subbase underdrains; 3’ from EOP	Shoulders reconstructed on NB, w/ OGDC & HMASL
61	US-10 EB (Midland)	56044	2010	6” JPCP 12’ joints	Open-Graded Mix (±1” thick) Also under inside shoulder	18” PDS at EOP; 3”-6” wide trench	Outside shoulder reconstructed; OGDC at EOP
63	I-75 (Linwood)	09035	2011	6” JPCP 12’ joints	HMA Mix 4E3 (±2” thick) Under mainline only	6” Open-Graded underdrains; 2’ from EOP	Shoulders reconstructed with OGDC

*ID numbers are for MDOT Pavement Management tracking purposes only.

3.0 MAJOR FINDINGS

3.1 Field Test Results

3.1.1 Drainage

An ineffective drainage system was common. In several of the eight projects investigated, drain trenches were partially capped with dense-graded HMA from construction (e.g. I-69 NB, Charlotte; US-131 Rockford, WB I-96, Portland and NB US-23) (Figure 3.1 and Figure 3.2), or no drainage system was installed during construction. For those cases the added problem is that dense-graded HMA in the shoulder blocked water from draining vertically (e.g. I-75 NB, West Branch; US-131 SB, Kalamazoo). In one project, WB US-10, sections with and without drainage system were constructed in order to study the effect of drainage. The results for WB US-10 are presented below in section 3.1.3.

(a) (b)

Figure 3.1 (a-b). US-23 NB, with capped drainage trench (a). Once opened water drained quickly (b).

(a) (b)

Figure 3.2 (a-b). Edge-drain location (a) and close-up of HMA cap over the pea-stone drain trench (b) (I-69 NB, Charlotte).

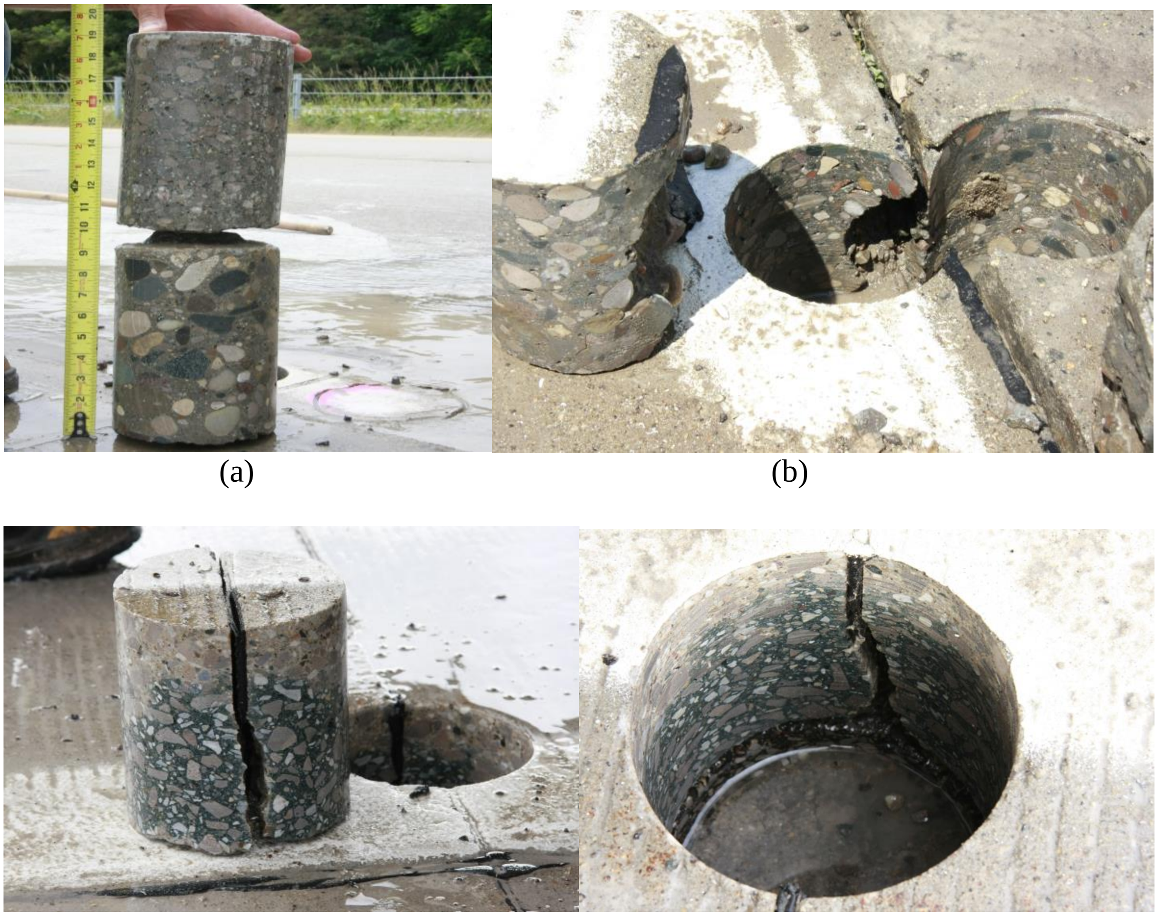
3.1.2 Joint Pumping Related Pavement Distresses

As a consequence of an ineffective drainage system or no drain at all, rain-water entering the joints can lead to pumping-related distresses. This was found to be a common factor in this study. Pumping consists of rapid movement of water trapped at joints under the high pressure being generated by the rapid movement of truck axles. Coring showed that erosion was typical within the separator layer along the slab-shoulder edge and in some cases within the concrete. Loss of material is pronounced on the downstream (i.e. after the joint) side of the transverse joint causing joint faulting and loss of joint support. Eventually, loss of support can lead to top-down slab cracking such as corner breaks, transverse cracking and longitudinal cracking. Joint pumping was visible in several projects (e.g. Figure 3.3 (a-d)). Cored samples at joints show extensive erosion within the HMA and bottom portion of the concrete (e.g. Figure 3.4 (a-d)). Loss of joint support due to pumping erosion can lead to a variety of slab cracking ranging from corner breaks to transverse cracks and even longitudinal cracks as seen in Figure 3.5 (a-e).

(a) (b)

(c) (d)

Figure 3.3 (a-d). Typical pumping observations following a heavy rain-fall.



(a) (b)

(c) (d)

Figure 3.4 (a-d). Pumping erosion at joints.

(a) (b)

(c) (d)

(e)

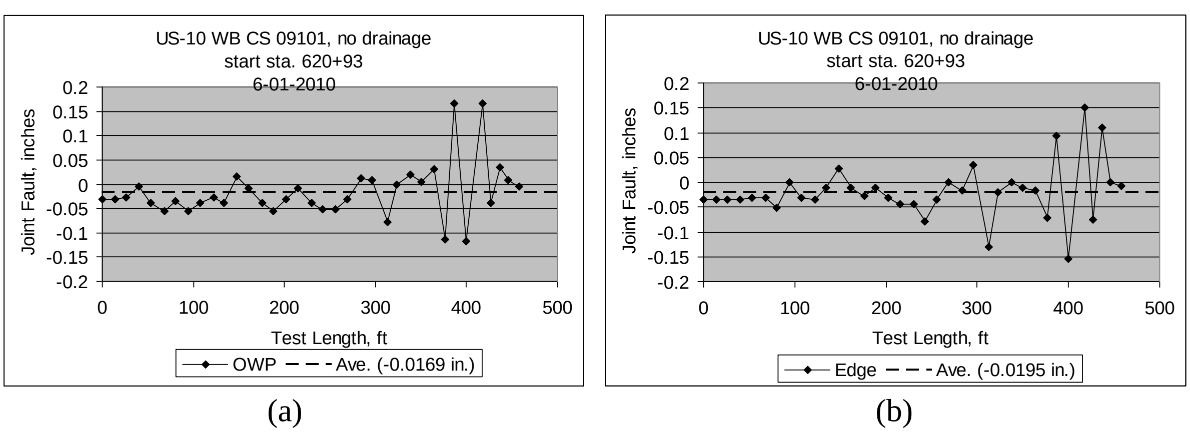
Figure 3.5 (a-e). Examples of pumping related slab cracking.

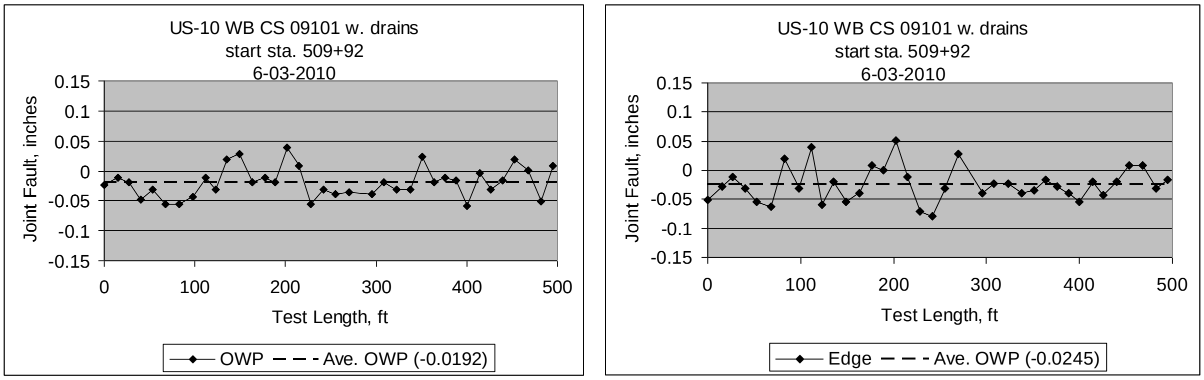
3.1.3 Transverse Joint Measurements

Joint fault measurements established the occurrence of pumping which is caused by repeated truck loading during periods where water is trapped in the joint (i.e. during and right after a rainfall). This causes joint lifting here defined when the load is before the joint (BJT), and a permanent joint depression after the joint (AJT).

Dowels are not only important shear-load transfer devices, but they also reduce and delay pumping effects by facilitating a uniform joint deflection as a wheel is moving across a joint. The I-75 NB, West Branch project clearly demonstrates this (Appendix A). The doweled section within this project has developed no slab cracking in year 6 at the time of field testing, while the undoweled sections have developed slab cracking. Another doweled, but much older project, built in 1990, where pumping is evident, has developed joint faulting (WB US-10) and slab cracking. Thus dowels can delay onset of faulting and cracking but not prevent them if pumping continues to be a factor.

The main purpose of field work on WB US-10 was to evaluate whether any differences in distress (joint faulting and slab cracking) had developed between test sections with and without a drainage system. No major differences could be determined since both test sections have developed extensive pumping related cracking and similar joint faulting values (Figure 3.6). These results are consistent with field coring showing erosion of HMA separator layer, and the examination of drain outlets, which showed that they were clogged and ineffective.

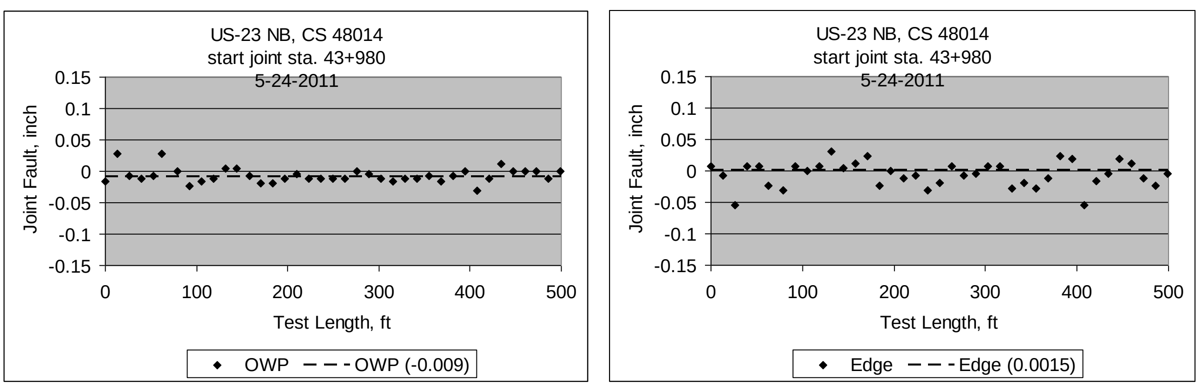




(c) (d)

Figure 3.6 (a-d). Joint fault values for US-10 WB test sections with and without drainage systems.

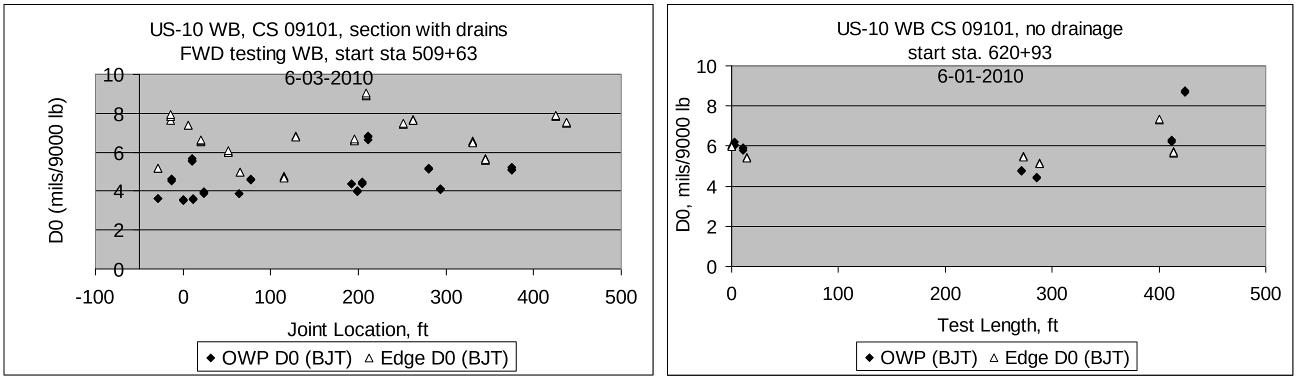
It is significant that one of the older projects from 1999 (US-23 NB, Northern Livingston Co.) has developed no slab cracking at the time of the field investigation (May, 2011) in the entire NB direction. Average joint faulting for the 500 ft test section was found to be insignificant (Figure 3.7) in both the outer wheel-path (OWP) and transverse joint edge. Thus, it is concluded that pumping is insignificant even though the drainage system was found to be capped (Figure 3.1).



(a) (b)

Figure 3.7 (a-b). US-23 NB, Northern Livingston Co., joint fault results.

Due to pumping effects in the WB US-10 joint edge deflections from FWD testing (Table 3.1) are higher as compared to a project where pumping is not a factor (e.g. NB US-23 project). (Figures 3.8 and 3.9, and Table 3.1).



(a) (b)

Figure 3.8 (a-b). Falling Weight Deflectometer (FWD) joint deflection values for the WB US-10 overlay.

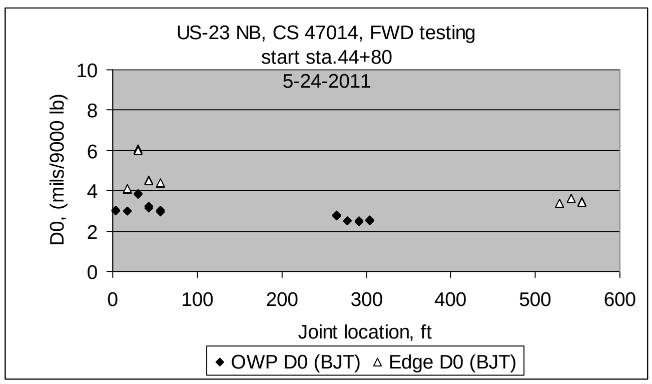


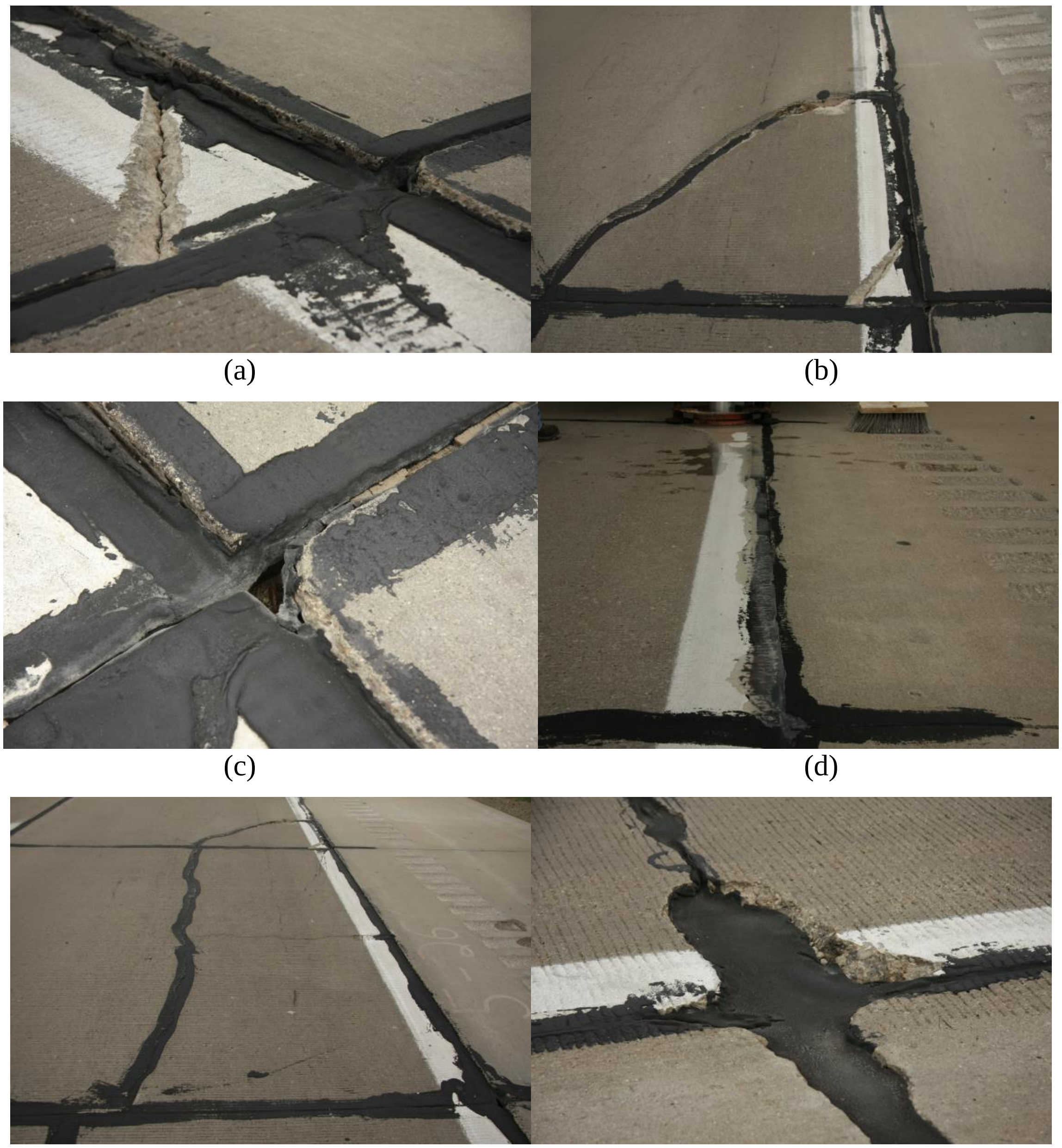
Figure 3.9. FWD joint deflection for the US-23 NB overlay.

Average joint deflections values in Table 3.1 are small as compared to conventional JPCP on granular bases due to the stiffer system of an overlay (overlay slab on 1-2 inch HMA and 9-10 inch old concrete).

Table 3.1 Normalized FWD deflection results (mils/9000 lb).

Route	Year Built	Month & Year Tested	Test Section	Midslab D0	OWP				Edge			
					D0 (BJT)	D1 (AJT)	D0(AJT)	D4 (BJT)	D0(BJT)	D1 (AJT)	D0 (AJT)	D4 (BJT)
I-96, Portland	1984	July, 2010	EB	2.34	4.97	3.7	5.42	4.21	6.34	4.99	6.57	4.87
			WB	2.2	5.58	3.68	4.62	3.57	7.52	5.88	6.53	5.23
WB US-10, Between Bay City and Midland	1990	June, 2010	No drain	N/A	6.02	4.26	5.86	4.07	5.85	4.63	5.7	4.52
			With drain	3.09	4.48	3.23	4.46	3.2	6.74	5.25	6.46	5.21
US-131, Plainwell	1998	August, 2010	NB	2.15	2.43	2.23	2.45	2.19	3.32	3.07	3.2	2.92
US-23,North Livingst. Co.	1999	May, 2011	NB	3.6	2.93	2.7	3.19	2.36	3.68	2.82	4.82	2.85
I-69, Charlotte	2000	June, 2010	NB	5.26	3.36	2.78	3.43	2.67	6.28	4.21	6.79	4.1
US-131, Rockford	2000	June, 2010	NB	4.01	2.9	2.71	2.88	2.63	3.74	3.48	3.78	3.44
			SB	7.64	5.43	5.14	5.31	4.95	3.96	3.74	3.94	3.67
NB I-75, West Branch	2003	Nov., 2009	1 (undoweled)	1.43	1.97	1.62	2.14	1.62	2.77	2	3.01	2.07
			2 (undoweled)	1.78	2.29	1.63	2.15	1.69	3.17	1.95	2.79	2.24
			3 (undoweled)	3.84	3.25	2.3	3.07	2.22	3.88	2.53	4.51	2.88
			4 (undoweled)	N/A	3.38	2.48	3.58	2.4	4.49	2.95	7.01	3.36
			5 (doweled)	2.35	3.17	2.67	2.71	2.06	5.92	4.38	6.47	4.27
US-131, Kalamazoo	2004	July, 2011	NB	2.08	2.62	2.29	2.58	2.26	4.8	4.11	4.88	4.18

The I-69 NB Charlotte overlay has developed major HMA erosion along the outer edge due to capped drains (Figure 3.2) consistent with loss of joint support and high edge deflection values (Table 3.1). Pumping has lifted the shoulder edge about 0.5 inches (Figure 3.10a). Despite the slab distresses (Figure 3.10 a-f) from pumping, joint faulting is insignificant (Figure 3.11b). It is likely that sealing the joints and cracks in years 8 and 9 has had a beneficial effect with respect to joint faulting. These distresses are similar to those found in the I-75 NB West Branch overlay sections 3 and 4, consistent with loss of slab support (Appendix A).

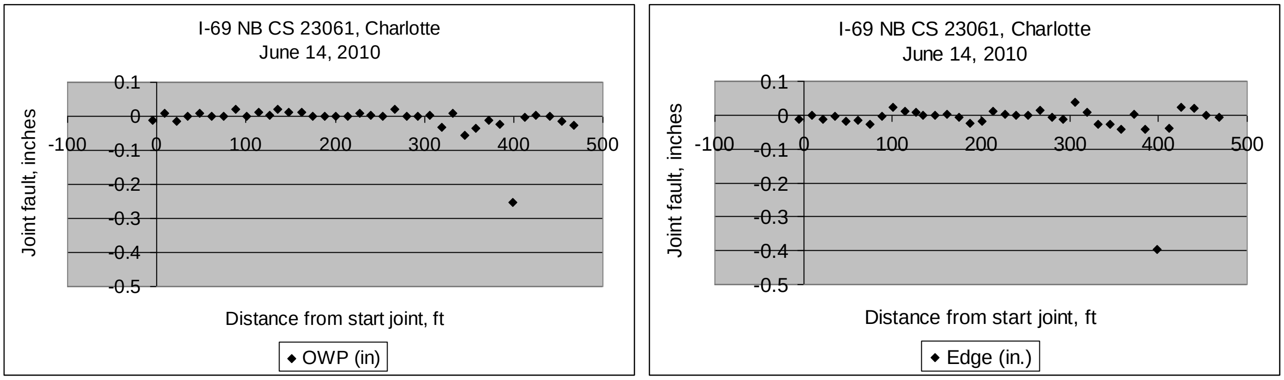


(a) (b)

(c) (d)

(e) (f)

Figure 3.10 (a-f). Slab distress found in the NB I-69, Charlotte overlay.



(a) (b)

Figure 3.11 (a-b). Joint fault in I-69 NB, Charlotte overlay

3.1.4 US-131 Rockford Overlay

Durability related joint distress has developed (Figure 3.12 a) in the NB direction, consistent with spalling due to poor salt scaling resistance. This condition points towards poor joint drainage and low air content in the concrete. Low air content was confirmed from laboratory test results (Appendix B) which show that total air in the field concrete is about 3% or less. Laboratory freeze-thaw tests confirmed poor salt-scaling resistance. The test results and mitigation by surface sealant coating are discussed further in section 3.3.

During field investigation, several drain outlets were found to be blocked and full of sand (e.g. Figure 3.12 b). A non-working drainage system for this project is consistent with plan details showing that the drainage trench was paved over (Table 2.2).

(a) (b)

Figure 3.12 (a-b). Joint staining and spalling in NB US-131, Rockford and drain outlets were clogged with sand.

The SB overlay section had developed straight-line longitudinal cracking in the vicinity of the wheel-paths. Coring showed that cracking had developed bottom-up. This type of cracking is consistent with vibrator trails due to over-vibration during concrete placement (Figure 3.13). Over vibration will increase the mortar content where the vibrator is placed.

(a) (b)

Figure 3.13 (a-b) Longitudinal cracking in the SB direction, US-131, Rockford.

3.1.5 Limitations of Field Results.

The results presented from the field reviews above are one "snapshot" in time. As such they are not sufficient to make absolute pavement performance predictions over time. The results do however show that good performance is possible as long as the drainage system is functioning as intended. Pumping is the major distress initiator. It causes faulting and slab cracking which will increase a road's roughness over time. The pumping distress mechanism has in previous studies been empirically linked to road roughness (NCHRP 1-37a). This is further discussed in section 3.2.

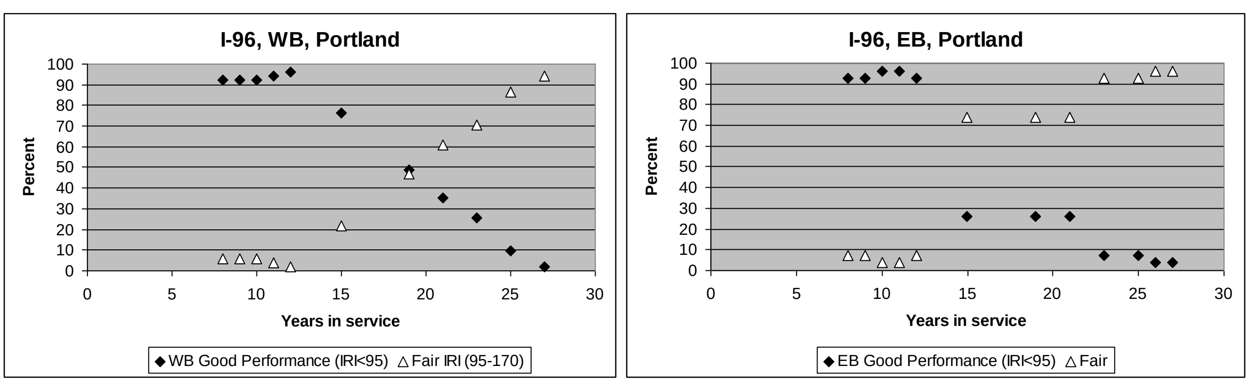
3.2 Pavement Roughness Development

The International Roughness Index (IRI) is a single parameter measure of a pavement's roughness.IRI values are calculated based on profile measurements every 1/10th mile, then stored in the MDOT Pavement Management System (PMS). IRI has units of inches/mile. FHWA defines IRI below 95 as "good" while IRI values above 95 and below 170 are in the "fair" category. A newly constructed pavement in Michigan must be less than 75 for acceptance.

Increases in IRI have been correlated empirically to joint faulting, slab cracking and site factor (i.e. erodibility) (NCHRP 1-37a). Each of these are affected by pumping. Thus, based on the field test results in this study the IRI increase over time can be used to assess effectiveness of the drainage condition for Michigan concrete overlays.

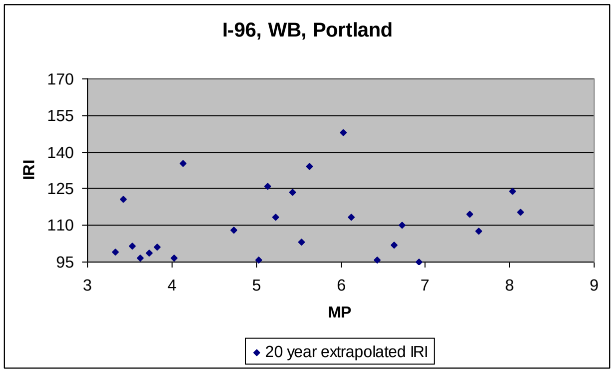
The percentage of IRI values for each traffic direction rated good is decreasing over time once structural level distresses develop, while the percentage of IRI values rated fair is increasing. This is illustrated for the oldest in-service overlay project (I-96, Portland) in Figure 3.14. Initially, pumping is localized and no structural level distress is developing. At some point, in this case after about 12 years, structural level distress is developing which can be seen from a decrease in percent of good IRI 0.1 mile segments at the expense of a rise in percent fair IRI values. For this project, preventive maintenance, PM, occurred in 2005 (i.e. year 21) and again in 2012 (year 28).

Individual IRI results (/0.1 mile) versus milepoint (MP) further show that distress development is location specific, likely the result of varying localized drainage quality. Figure 3.15 illustrates only IRI values in the fair category for WB and EB directions. These results identify “hot spots” (i.e. locations by MP where PM is most needed) with localized high IRI values. After 20-years (i.e. design life), about 50% of IRI values are now in the fair category. Again, in year 28 (2012) joint repairs were undertaken (Figure 3.16).

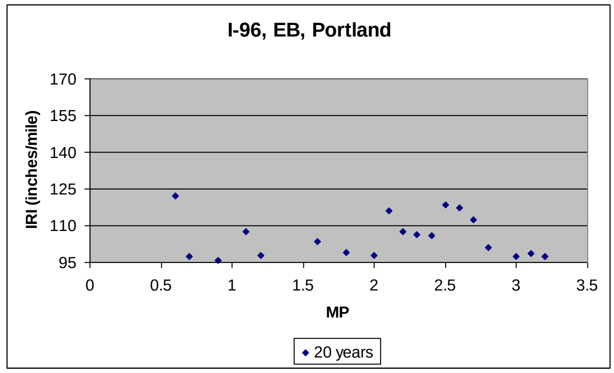


(a) (b)

Figure 3.14 (a-b). Development of percent IRI of total project IRI in the Good and Fair ranges over time for the I-96, Portland JRCP overlay.



(a)



(b)

Figure 3.15 (a-b). Localized distress "hot spots" (IRI>110) in WB (a) and EB (b) directions.

Figure 3.16. Joint repairs in year 28 (2012) for the I-96, Portland overlay.

The US-23, NB, project has not developed any pavement distress by year 12 (2011) consistent with IRI plot in Figure 3.17.

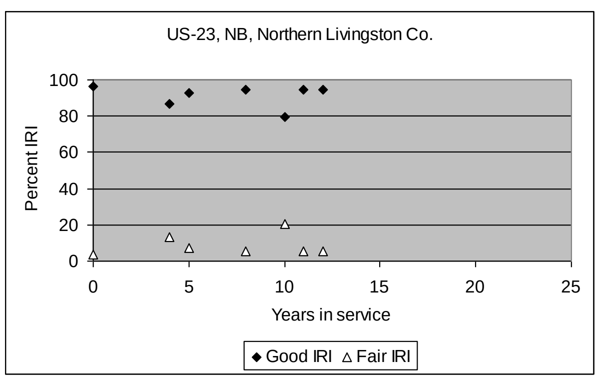


Figure 3.17. Good performance is found for US-23, NB, Northern Livingston Co. overlay project in year 12 (2011).

3.3 Laboratory Test Results for the US-131 Rockford Concrete

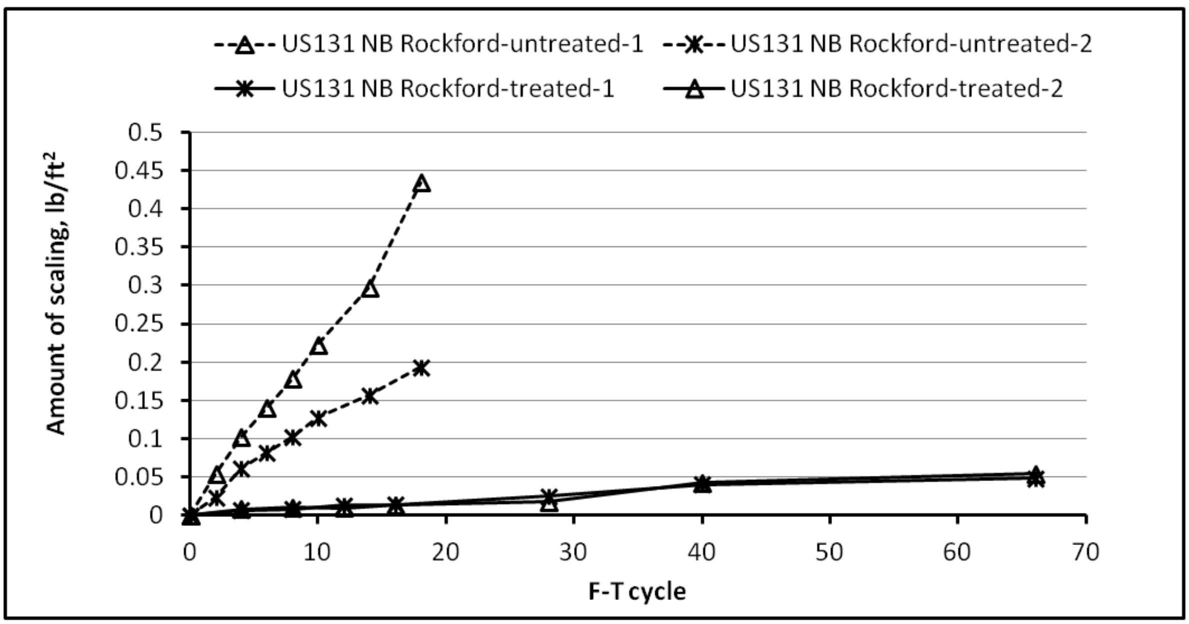
3.3.1 Air-Void Analysis based on ASTM C-457

Laboratory test results for air content and air-void system based on ASTM C457 showed that the field concrete (NB and SB) had insufficient air content (about 3% total air), which according to practical experience renders poorly drained joint concrete susceptible to rapid deterioration from repeated freeze-thaw exposure in the presence of deicer salts (PCA, EB001, 14th edition). The air content and air-void parameters were found to be adequate for the other field concretes (Appendix B) except for the NB I-69 Charlotte concrete samples. However, the Rockford overlay had developed joint spalling, typical of poor salt scaling resistance. Thus, it was decided to subject concrete samples to laboratory salt-frost scaling tests.

3.3.2 Salt Frost Scaling

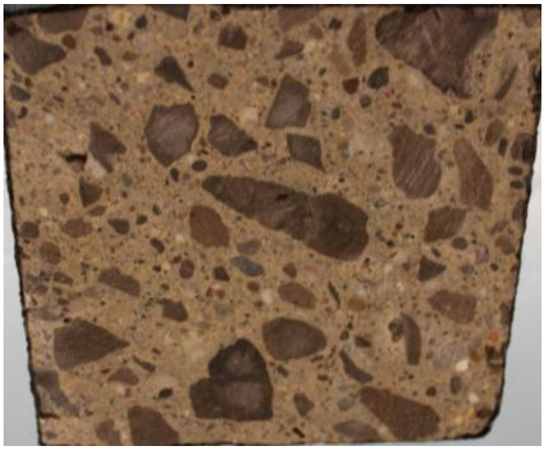
Laboratory salt scaling test results verified poor resistance to a 3% NaCl solution (Figure 3.18 a and b**)** for the US-131, Rockford, concrete. Mitigation by means of a surface treatment with a silane liquid was investigated for this concrete. The surface of two cut field samples were pre-dried followed by re-saturation to a saturated surface dry condition and air-dried at room temperature for about 3 hours before surface treatment. The silane treated specimens were cured for one week at room temperature and then subjected to freeze-thaw testing using German test procedure (RILEM TC-117). This test consisted of exposing the silane treated surface to a 3% salt solution during a freeze-thaw cycle. The surface scaling was reduced substantially (Figure 3.18 b). A comparison of the untreated surface before and after 22 freeze-thaw cycles illustrates that salt scaling deterioration is occurring within the paste (Figure 3.19 a and b), and that a silane surface treatment is effective (Figure 3.19 c) in reducing salt scaling of concrete with low air content for at least 66 freeze-thaw cycles in the laboratory.

(a)

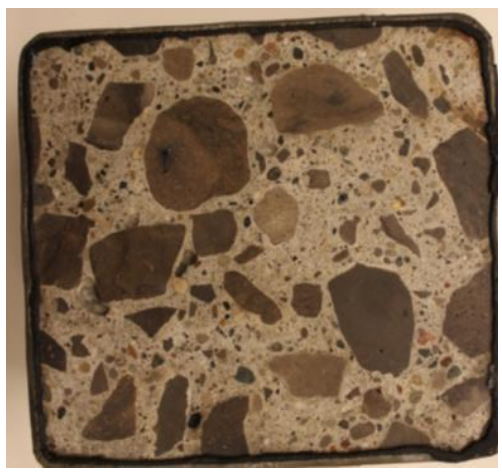


(b)

Figure 3.18 (a- b). Salt scaling test equipment (a) and results (b).




(a) (b)



(c)

Figure 3.19 (a-c). Specimen surface before test (a) following 22 freeze-thaw cycles of untreated surface (b) and after 28 freeze-thaw cycles of treated surface (c).

3.4 Early-Age Thermal Cracking: US-131 Kalamazoo

Random slab cracking developed in one project during hot weather paving conditions (July, 2004) where ambient temperatures ranged between 75 ^{0}F and 95 ^{0}F.

Slab-HMA base analysis was conducted in EverFE for a 6.5 inch thick slab and elastic modulus of 2 x 10^{6} psi (lower modulus during early age).The base consisted of a 1 inch rough HMA with a friction based shear stress-slip relation developed by Rasmussen and Rozycki, (2001) assuming a 0.999 ksi/inch slip for the elastic range. The critical slip value of 0.0098 inches marks the transition from elastic behavior to a fully plastic behavior (i.e. sliding friction). During cooling the slab-base stress was analyzed for a temperature drop of 15 ^{0}F and 25 ^{0}F for increasing continuous slab length in order to simulate a pre-saw cut condition. A rough HMA interface produces significant tensile stress at the slab bottom as seen from the simulations in Figure 3.20.

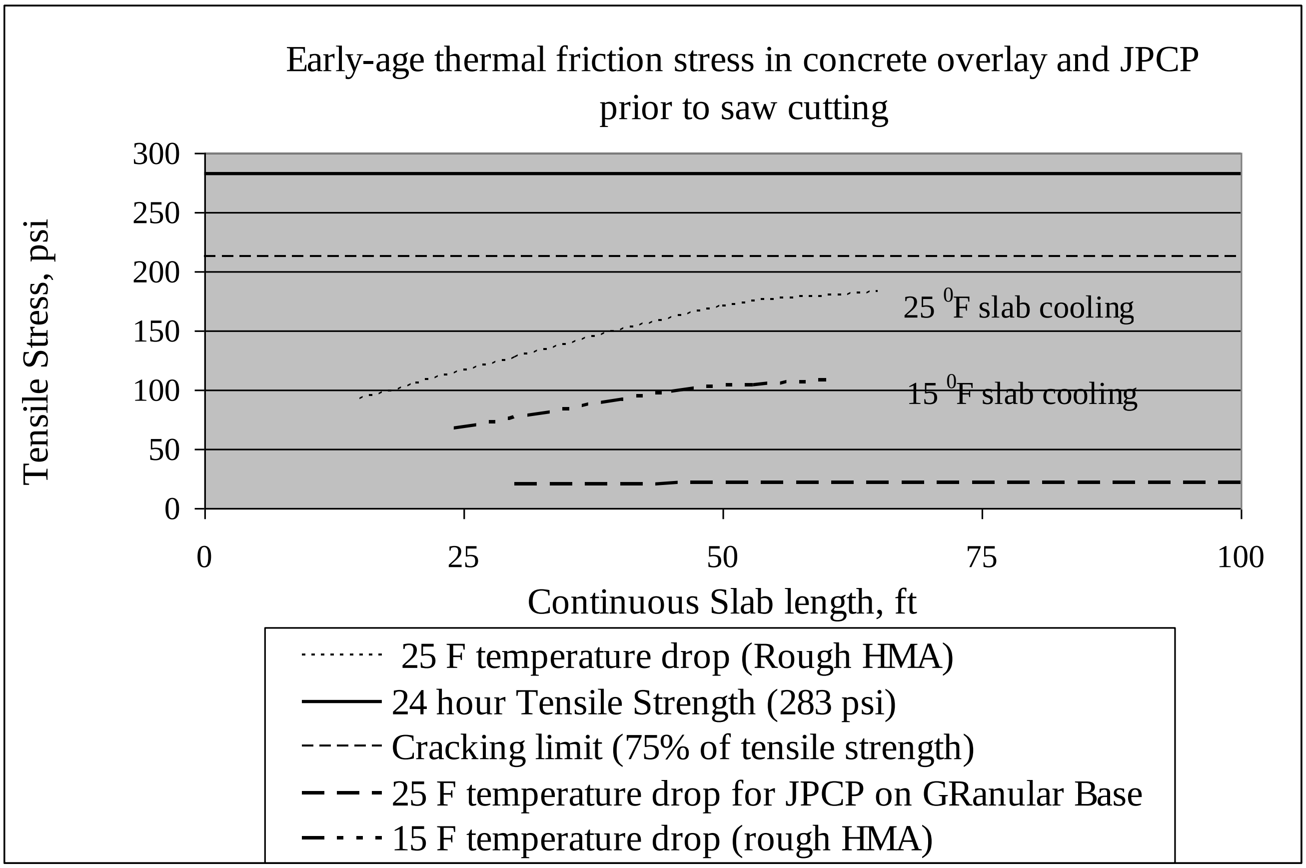


Figure 3.20. EverFE tensile stress predictions at bottom of overlay concrete for increasing continuous slab length due to friction forces between the concrete and the HMA interlayer.

A 24-hour tensile strength of 283 psi was calculated from compression tests on laboratory concrete containing same mix proportion as the US-131, Kalamazoo, concrete. These simulations support practical observations that joint sawing in unbonded concrete overlays should be undertaken as soon as possible due to the stiffer base condition (Harrington et al., 2008). The same simulations were run for a JPCP over a granular base with recommended shear-slip constants (k=0.099ksi/inch; slip limit=0.02 inches). Friction-based slab stresses were small for this case.

3.5 Importance of Drainage on Future Maintenance Activities (B. Krom, MDOT)

An in-depth cost analysis of actual maintenance costs was not performed as part of this study, but some general observations can be made based on field evaluations and project maintenance histories. Only three of Michigan's concrete overlays have reached their 20 year design life, and are all still in-service, with the following maintenance history to date:

• I-96, Portland (1984): drainage system installed; two cycles of maintenance, performed at ages 21 & 28

• US-10 WB, Bay City (1990): drains versus non-drains test sections; four cycles of maintenance at ages 6, 11, 16 & 17

The Portland project on I-96 is JRCP, which may not be representative of JPCP overlay performance. The US-10 WB project is JPCP, but contains multiple test section configurations (drains vs. no-drains, dowels vs. no-dowels), which is not standard practice for current overlays.

Two newer concrete overlays have notable maintenance histories as well:

• I-94, Jackson (1995, JRCP): no drainage system; required 8 maintenance cycles; achieved 15 years of life prior to reconstruction.

• US-23, Northern Livingston County (1999, JPCP): drainage system installed (field testing indicated potential blockage); no maintenance after 14 years.

The maintenance histories of the other concrete overlays built to date fall in-between the two extremes above, or are too young to exhibit any trends.

The common thread that runs between the oldest and newer concrete overlays appears to be a functioning drainage system. Planning for drainage in the project design phase, and building it correctly in the field, appears to be the keys to a well-performing, and long lasting, concrete overlay

4.0 CONCLUSIONS

The overall performance of Michigan concrete overlays has been good. However, some recent JPCP overlay projects have developed premature distress with signs of pumping. To determine the causes for the distress a joint UM-MDOT forensic investigation was initiated in December 2009.

4.1 Conclusions

- Forensic investigations into eight concrete overlay projects show that pumping, which is the rapid movement of trapped water from a moving truck axle across a joint, was found to be the major cause of distress. This condition is the result of inadequate drainage. Pumping erodes primarily the hot-mix asphalt separator layer causing loss of slab support, which is most severe along the outer longitudinal edge.

- Forensic investigations also show that doweled joints are effective in reducing and delaying pumping erosion effects (i.e. joint faulting, slab cracking) consistent with state of the art knowledge (NCHRP 1-37 Design Guide), as dowels provide a condition of uniform slab deflection when subjected to moving axle loads.

- Finite element analysis (EverFE) predicts that concrete overlays are more sensitive to slab cracking from loss of joint support due to the stiffer slab support condition that exists as compared to JPCP on aggregate bases. These simulations can explain the top-down cracking associated with joint truck axle loading. These longitudinal cracks originated at the transverse joint in the un-doweled sections of the I-75 NB, West Branch project due to loss of support along the outer longitudinal edge.

- A construction issue such as initiating joint sawing as early as possible is especially important for concrete overlays. EverFE finite element analysis predicts that thermal stresses are significant as a result of friction between a concrete slab and the hot-mix asphalt separator layer during slab cooling for a continuous slab condition (i.e. prior to saw cutting).

- Laboratory test results for air content and salt scaling freeze-thaw resistance are consistent with field results for one project (US-131, Rockford) that show rapid development of joint concrete spalling. The cause is low air (<3%) consistent with poor salt scaling resistance from laboratory testing of field concrete.

- IRI measurements in the MDOT PMS (Pavement Management System) database complemented forensic investigation results that support the pumping mechanism. Two distinctly different performance stages were identified from IRI results. Initially the percentage of good IRI (IRI < 95) for a project length is constant and nearly 100%. This is the period where pumping erosion has little if any structural level effect, but is developing at the sub-structural level. This period is much shorter if drainage is poor. Once structural level pumping distress is developing, commensurate with joint faulting, slab cracking and spalling, IRI values above 95 are found, and percentage of IRI in the good range (i.e. <95) is decreasing rapidly. Analysis of percentage of IRI values above 95 versus milepoint is a good measure for establishing rehab strategies. Preventive maintenance and rehabilitation include joint sealing, drainage retrofit, diamond grinding, and full depth repairs.

5.0 IMPLEMENTATION OF STUDY FINDINGS

This research study achieved its objective of determining the potential causes for the premature deterioration that has developed in some concrete overlay projects. The findings from this study confirmed that pumping erosion was a major factor. Further, this study has provided the understanding and necessary evidence to modify drainage specifications and construction practices to prevent future occurrences of similar premature deterioration caused by pumping erosion within the interlayer.

The steps involved to implement the study's findings, after review and acceptance by the department, include:

5.1 Education and communication within and between the Department and Industry are essential in meeting & improving service life of concrete overlays

The following points should be emphasized:

- The department will need to explain the need for improved drainage systems to the construction industry through their in-place committee contacts.
- Improve existing construction practices and quality control procedures to eliminate avoidable problems. Identify new QC/QA procedures that may be needed.
- Educate MDOT personnel about importance of periodically cleaning drain outlets.

5.2 Preparation/Development - The department will need to revise current drainage designs

The following drainage options shown below are proposed to serve as a possible solution for improved performance. Figure 5.1 shows a combination of open-graded HMA extended and thickened (e.g. NB US-131, Kalamazoo) under the shoulder, combined with a conventional vertical drainage system, depending on the soil condition. Another option, shown in Figure 5.2, depicts the case where an open-graded interlayer is not placed under the shoulder, but uses an unbound open-graded drainage course and a vertical drainage system to remove water from the pavement structure (e.g. EB US-10, Midland).

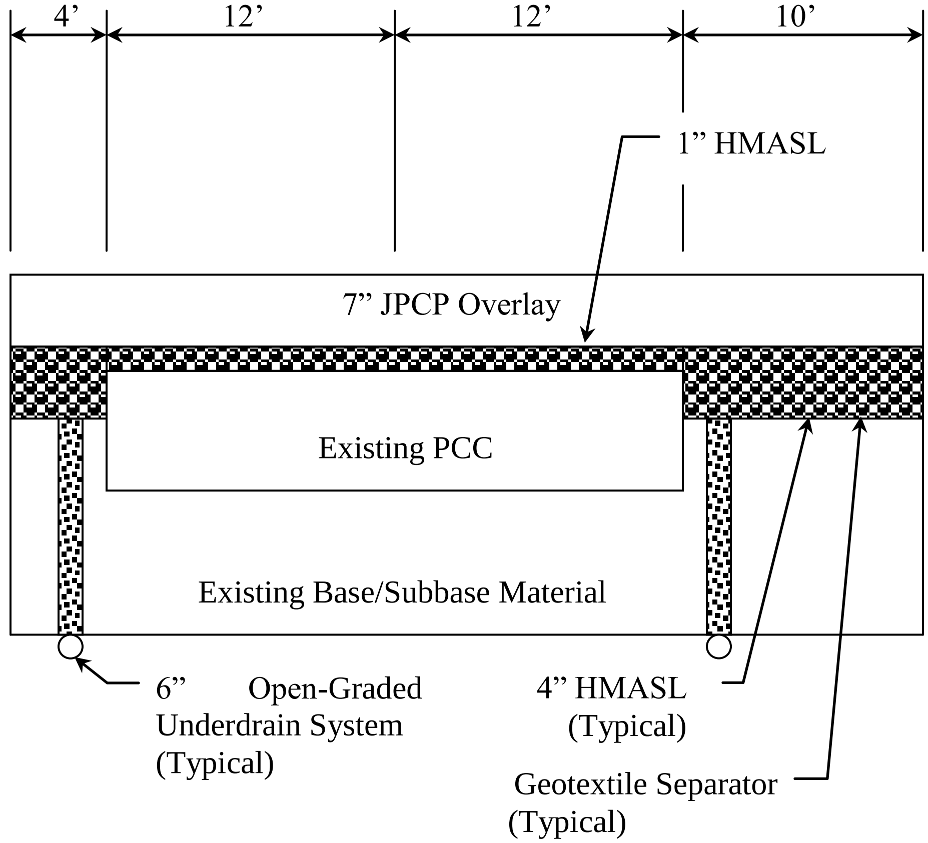


Figure 5.1 New Overlay Drainage System: Thickened Shoulder Design with Open-Graded Underdrains (depending on soil conditions).

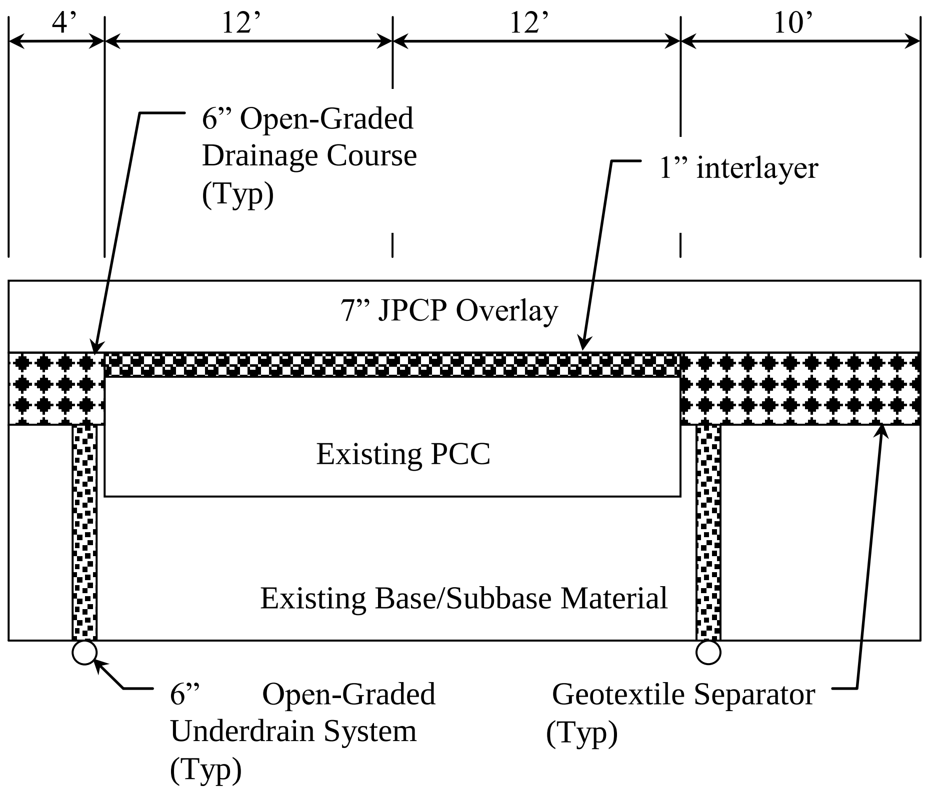


Figure 5.2 New Overlay Drainage System: OGDC Shoulder Design

5.3 Pavement Monitoring for Drainage Performance

As a result of this study IRI was found to be an excellent measure for evaluating the effectiveness of a drainage system due to the strong correlation between pumping related distress and increase in pavement roughness over time. It is recommended that MDOT monitor IRI over time for in-service concrete overlays, as a pre-cursor to determining the need for CPM. This requires fine-tuning the IRI threshold when a forensic investigation should be initiated.

REFERENCES

M. J. Eacker and A. Bennett, "Unbonded Concrete Overlay Demonstration Project on I-75 in Ogemaw County – Construction Report", MDOT Research Report R-1465, pp.1-22, June 2005.

"Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures", NCHRP Report 1-37a, Part 3, Chapter 7 on PCC Rehabilitation Design of Existing Pavements", ARA Consultants, March 2004.

www.civil.umaine.edu/EverFE/Theory_Manual.pdf by William Davids, University of Maine, 18 pp. 2003.

W. Hansen and Y. Kang, "Durability Study of the US-23 Aggregate Test Road and Recent JPCP Projects with Premature Joint Deterioration", MDOT Research Report RC-1534, 38 pp., 2010.

"Evaluation of Unbonded Portland Cement Concrete Overlay," NCHRP Report 415, TRB, National Research Council, Washington, D.C., 1999.

"Standard Practice for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete", ASTM C457, American Society for Testing and Materials (ASTM International), pp.1-14, Vol. 04-02.

"Design and Control of Concrete Mixtures", Portland Cement Association, Bulletin EB001.14, 2003.

Harrington, D. et al., "Guide to Concrete Overlays: Sustainable Solutions for Resurfacing and Rehabilitating Existing Pavements," National Concrete Pavement Technology Center, Iowa State University, Ames, IA (Second Edition 2008).

RILEM TC 117-FDC, "TC 117-FDC Recommendation - CDF test - Test method for the freeze- thaw and deicing resistance of concrete - Tests with sodium chloride (CDF)", Journal, Materials and Structures, Vol. 29, pp.523-528, (1996).

Rasmussen, R.O. and Rozycki, D.K. "Characterization and Modeling of Axial Slab-Support Restraint," Transportation Research Record 1778, TRB, National Research Council, Washington, D.C., pp. 26 – 32. (2001).

APPENDIX A I-75, NB, WEST BRANCH

1.0 Introduction

To assess specific design features of unbonded overlays and their cost relationships, MDOT constructed a demonstration project on I-75 NB near West Branch in Ogemaw County in the Fall of 2003. The project limits are from Ski Park Rd. to the Ogemaw/Roscommon county line as illustrated in the map (Figure A1). It has a total length of 20,314 ft.

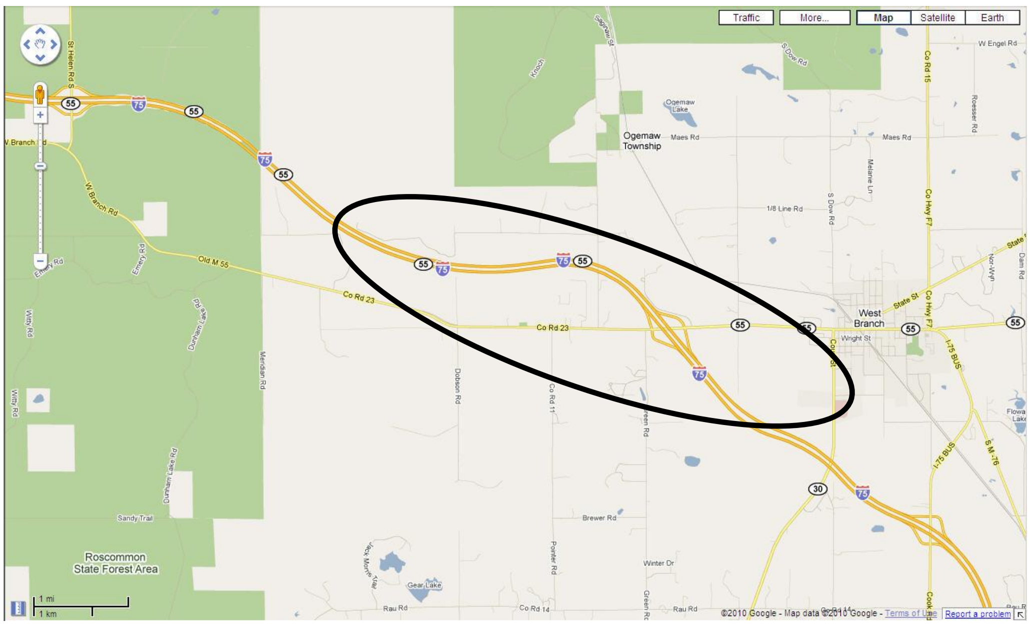


Figure A1. Map showing location of the I-75 NB West Branch Demo Project.

MDOT Research Report R-1465 describes in detail the experimental features and construction of this Demonstration Project. In summary, the project consists of 5 test sections:

- #1 (1372 ft): 10' joint spacing, undoweled, unsealed
- #2 (1336 ft): 10' joint spacing, undoweled, sealed
- #3 (7,600 ft): 12' joint spacing, undoweled, unsealed
- #4 (7,500 ft): 12' joint spacing, undoweled, joint seal (hot pour rubber) in transverse and one longitudinal joint (inner joint)
- #5 (2,506 ft): 12' joint spacing, doweled, sealed only in transverse joints and longitudinal joints between lanes.

The design is intended for pavements with low truck traffic (< 5,000 trucks a day). For all sections the outer joint between lane and shoulder was left unsealed. All sections consist of a thin (6 in.) concrete overlay, a one inch thick permeable HMA separator layer. The original 1973, I-75 pavement is a nine inch thick JRCP with a 71 ft-2 inch joint spacing, and the shoulder consisted of a dense-graded HMA, Full-depth concrete patches were constructed where deteriorated joints, cracks and distressed repair patches existed.

2.0 Major Findings

2.1 Drainage

Although the Demonstration Project included a PDS or leaching trench at the outside shoulder (Figure A2) it was decided during the construction phase to omit the PDS based on the subgrade soil condition (sandy soils). However, the old dense-graded HMA shoulder was not removed prior to construction of the overlay. Thus, the open-graded was found to butt up against the old HMA, preventing trapped water from draining (Figure A3). Based on coring in the shoulder near the outer pavement edge it appears that the old concrete extended into the shoulder. Finally, the concrete overlay in the shoulder was about ¼ inch higher than the outside lane. These factors combined have created a condition of trapped water in the unsealed joint areas leading to pumping distress.

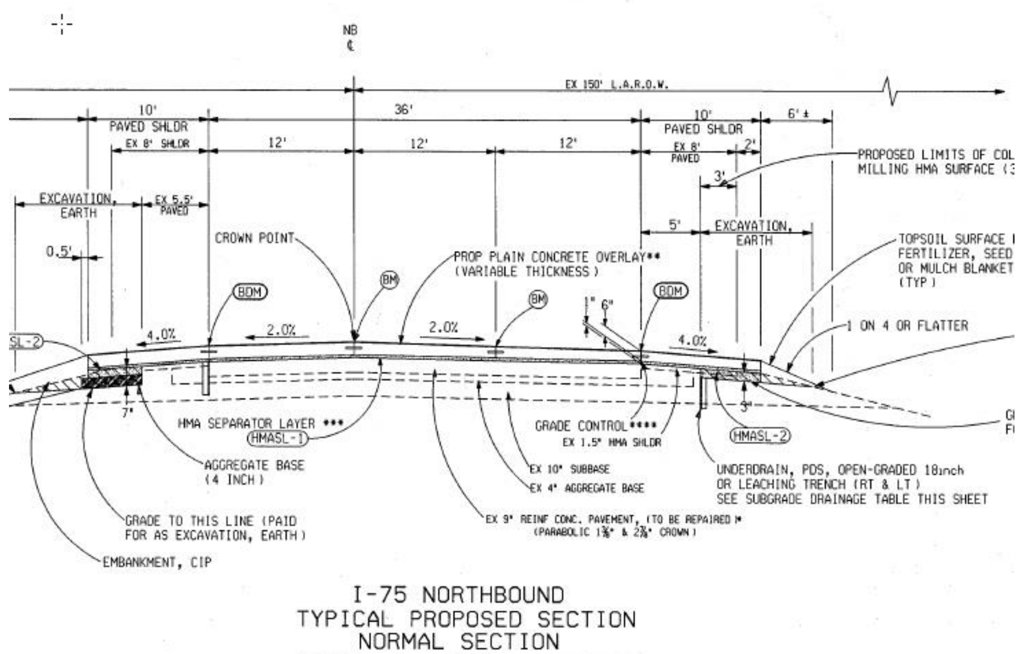


Figure A2. Proposed typical section and drainage design showing open-graded HMA throughout the shoulder and a vertical under-drain.

Figure A3. Dense-graded HMA not milled off during construction.

2.2 Pumping and Associated Distress

Sections 1 and 2 are mostly on a super elevation and hill section sloping away from the outer lane towards the two inner lanes. Therefore, signs of pumping within the HMA are more evident in these lanes (Figure A4) and help explain why no slab distress was found in the outside lane of sections 1 and 2. These findings suggest that dowels, which were a major study factor for a thin overlay, may not be needed for this traffic level as long as pumping is not a factor. However, when pumping erosion is a factor, dowels are important. This is concluded by comparing slab distresses (joint faulting and slab cracking) of the undoweled sections (3 & 4) with section 5 (doweled). Due to dowels, pumping distress is reduced in section 5, while sections 3 and 4 have developed significant pumping related distresses (Figures A5 to A7). The joint fault results are shown in Figure A7 (a-e). The convention is to present a drop-off across a joint in the traffic direction as a negative joint fault value, while a positive value means a step-up condition. Section 3, with no joint sealant, has developed largest negative joint faulting (standard deviation = 0.060 in.) and largest variation in joint faulting, and largest percentage of slab cracking (5.8%). The large variability in joint faulting is attributed to the random entry-ways for water at joint corners. Section 4, with joint seal (hot pour rubber) in transverse and one longitudinal joint (inner joint) has developed less joint faulting with lower variability (0.047 in.), and less slab cracking (1.4%) than section 3. No slab cracking has developed in section 5 (doweled), which is attributed to the reduction in pumping due to a uniform joint deflection.

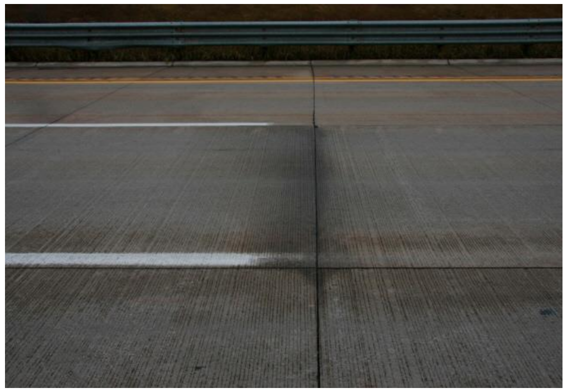
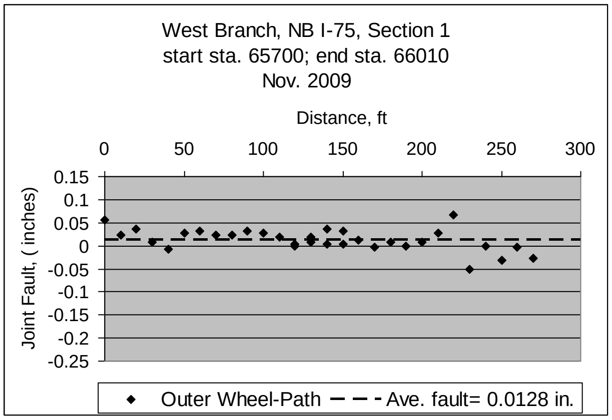


Figure A4. Pumping in super elevation part of section 1 sloping to the west.

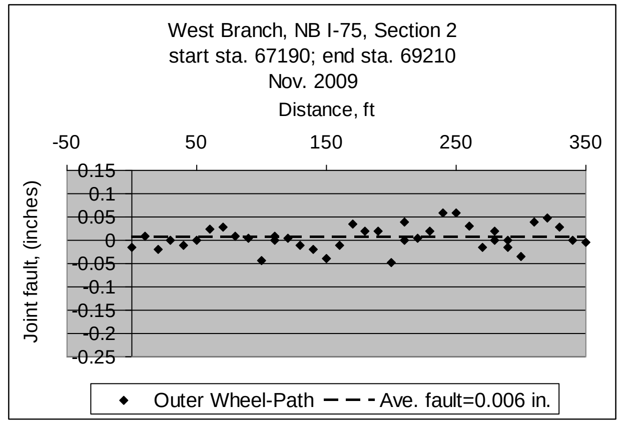
(a) (b)

Figure A5 (a-b). (a) Section 3 (station 699+86) with corner-break due to HMA pumping erosion at locations of water entry point along joint edge corner (b).

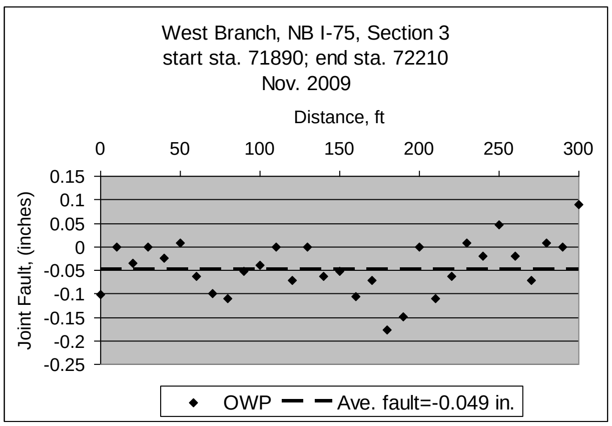
Figure A6. Full slab length longitudinal cracking at mid-panel (Section 3).



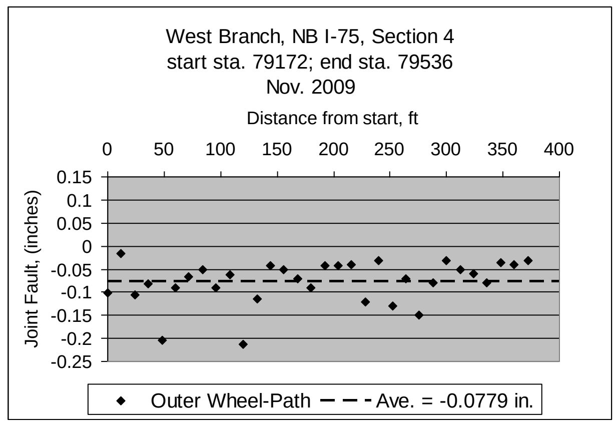
(a)



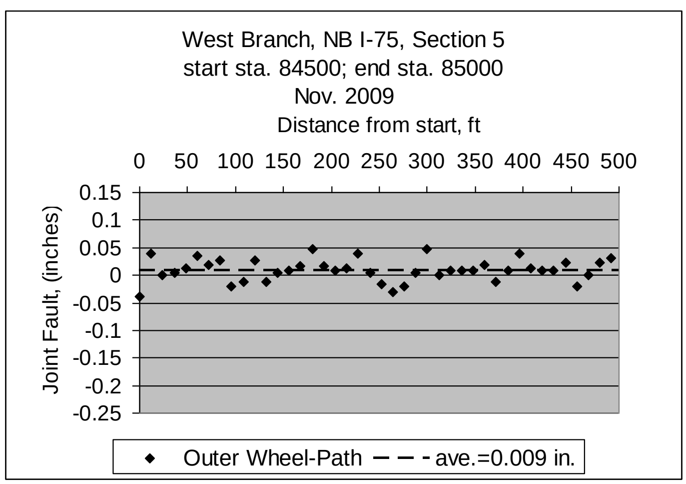
(b)



(c)



(d)



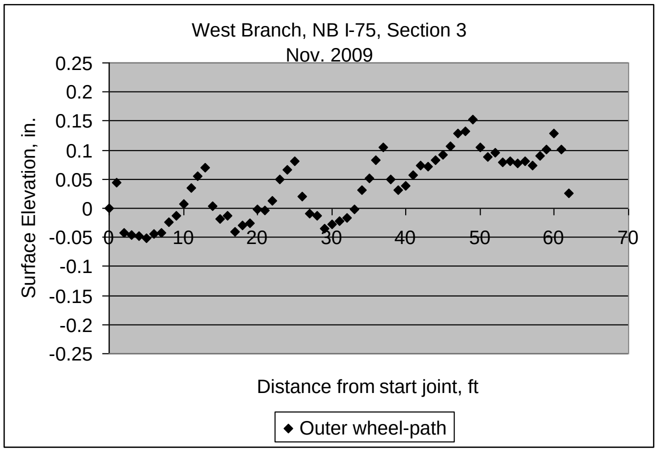
(e)

Figure A7 (a-e). Joint fault results in the outer wheel-path for Sections 1-5 in Year 6 (November 2009).

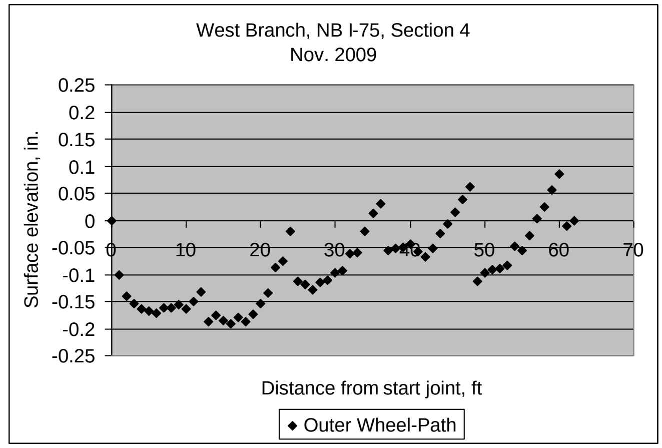
A joint core in section 3 (Figure A8) shows pumping erosion has developed in the concrete overlay and in the interlayer. Concrete joint erosion is substantial. Thus, shear-load transfer by aggregate interlock is questionable, and therefore, joint-load transfer calculations are not realistic. Therefore, the average deflection values (mils/9,000 lbf) are presented in Table 3.1 (Chapter 3) for the D0 values, deflection underneath the load-plate, for loading before the joint (BJT) and after the joint (AJT). These tests were conducted on three slabs within each test section that had no visible cracking. The outer corner D0 values BJT and AJT are slightly larger in sections 3, 4 and 5 as compared to sections 1 and 2, where moisture entrapment along outer corners is less. This suggests that erosion is happening. Surface elevation profiles for 5 consecutive slabs illustrate that slab joint faulting and slab rotation from pumping is reduced in the doweled section (Figure A9 (a-c)).

(a) (b)

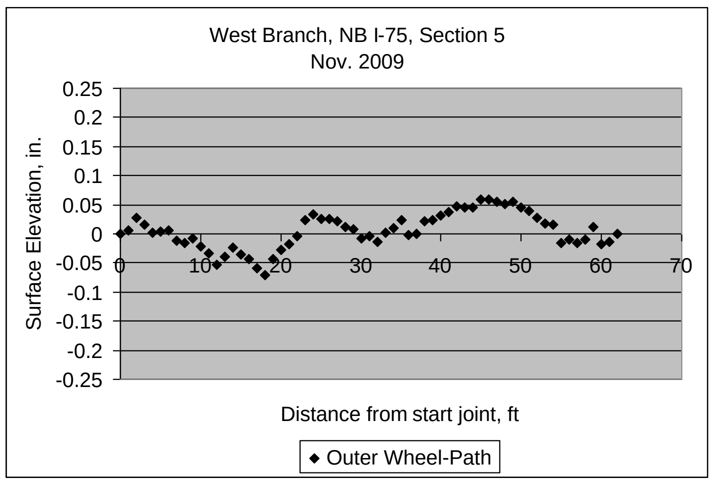
Figure A8 (a-b). Extensive joint concrete and base erosion along outer edge (Section 3).



(a)



(b)

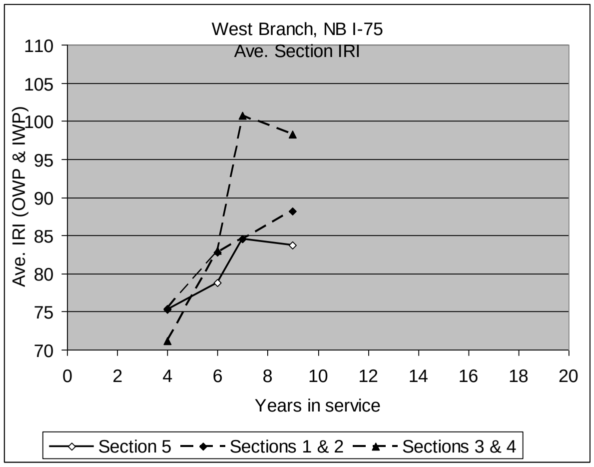


(c)

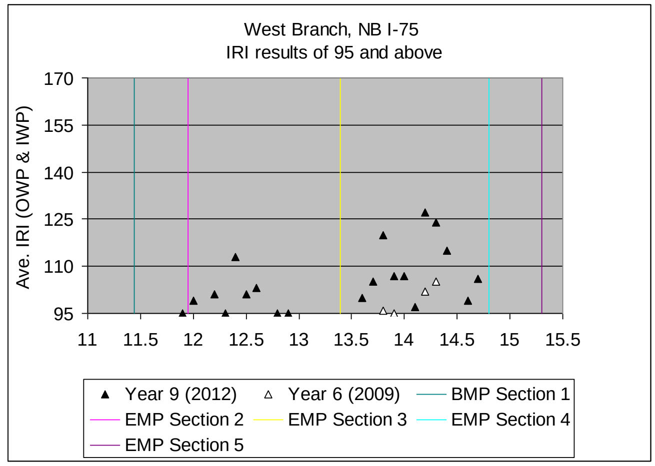
Figure A9 (a-c). Dipstick surface profiles for 5 consecutive slabs in November 2009.

2.3 Development of Pavement Roughness

Pavement roughness increase over time, as measured by the International Roughness Index, IRI, has been correlated empirically to pumping related distress in JPCP pavements (NCHRP 1-37a). Thus, IRI increase over time was found to be a good measure of distress development. MDOT's Pavement Management System, PMS, database provides IRI values for every tenth mile along the outer and inner wheel-paths. The results in figure A10a show a rapid increase in yearly average section IRI for sections 3 and 4, while increase in pavement roughness is lower for the sections with less pumping. Individual IRI results in the fair range (95-170) are good indicators of locations where pumping is most severe (Figure A10b).



(a)



(b)

Figure A10 (a-b). IRI results for West Branch, NB I-75.

2.4 Loss of Joint Support Causing Top-Down Longitudinal Cracking

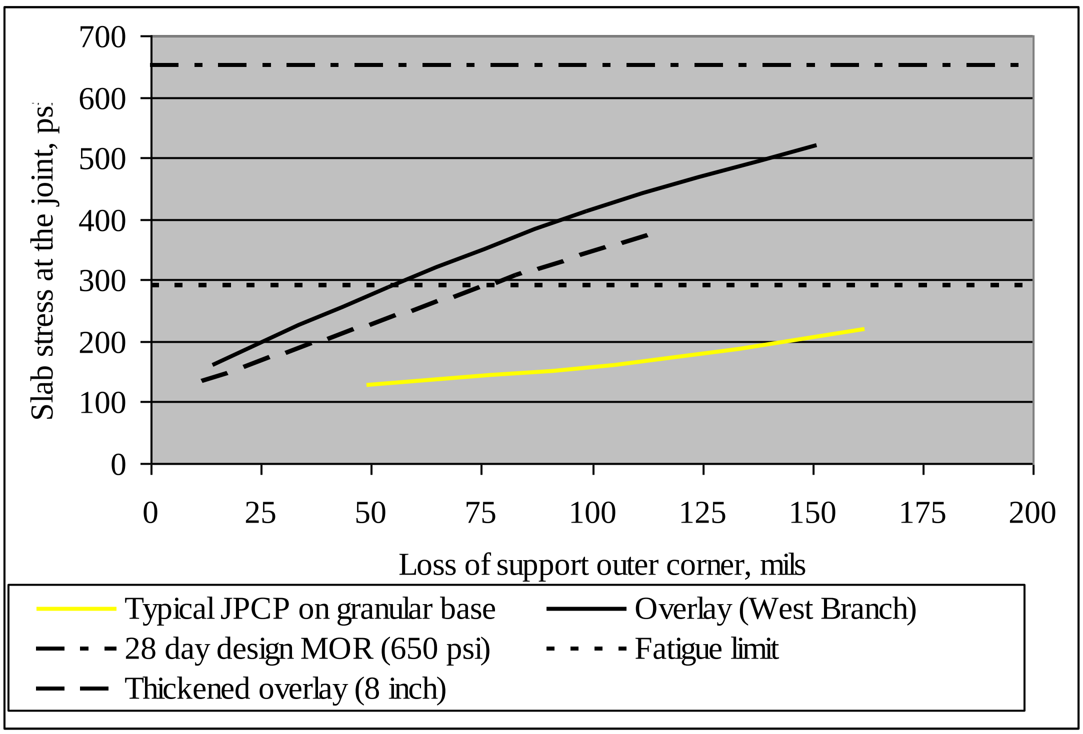
Section 3 has been exposed to the worst case scenario of pumping as a result of no joint seals. This section developed longitudinal cracking at the transverse joints in year one (Figure A11), rapidly propagating into full length cracks such as shown in figure A6. During the field investigation in November of 2009 a core taken at the transverse joint over a partial length crack illustrates that longitudinal cracking is top-down (Figure A12), consistent with loss of joint support at the outer pavement edge.

Figure A11. Start of longitudinal cracking at joints in section 3 within year 1 (October 2004).

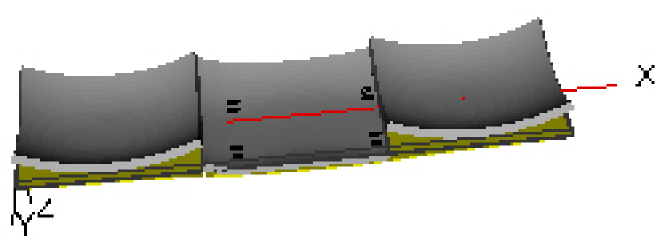
Figure A12. Top-Down longitudinal cracking starting at transverse joint station 709+08 (section 3)

According to finite element predictions a JPCP overlay is more sensitive to axle loading if loss of joint support has developed along the outer joint edge as compared to a JPCP on open-graded drainage course, OGDC (Figure A13). These simulations were run for the undoweled West Branch project to simulate longitudinal cracking for Section 3 (12 ft joint spacing, 6 inch slab thickness; 1 inch HMA-SL and 9 inch slab thickness for the old concrete slab). The comparison is a JPCP on OGDC (i.e. 15 ft joint spacing; 10 inch slab thickness; un-doweled joints; 4 inch OGDC; 12 inch sub-base). For this case a 50 ksi modulus was assumed for slab base/subbase. For both cases a sub-grade k-value of 150 psi/inch was assumed. To model loss of slab support along outer edge at transverse joints an increasing curling uplift was applied (i.e. negative temperature differential). Joint loading consisted of a typical Michigan truck axle configuration (i.e. dual tire axle loading with axle spacing of 9 ft). One set of tires is placed near outer joint corner, where HMA erosion is most pronounced. For the case of the West Branch overlay no aggregate interlock was assumed since joint cores show substantial concrete erosion at the joint (Figure A8a). For both pavements three-consecutive slabs were modeled with axle loading in the middle slab (Figure A13b)

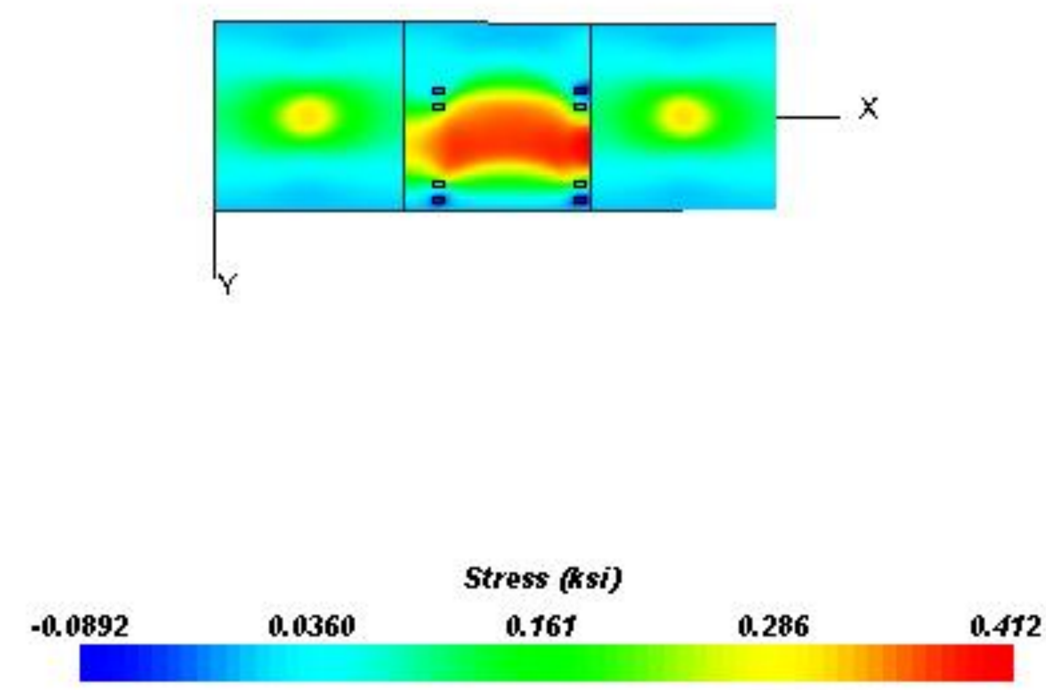
Longitudinal slab stresses at the transverse joint are much more sensitive to loss of joint support in an overlay than in a JPCP on OGDC as seen from stress simulations (figure A13a) as they quickly reach the fatigue region (above 300 psi in flexure stress) for a typical 28-day design strength (650 psi). Slab stresses at the joint are maximum at the slab top surface. A typical stress contour plot (figure A13c) illustrates that slab stresses are maximum at the joint between the two axle-wheels in the y-direction (i.e. parallel to the transverse joint). Increasing slab thickness from the current 6 inches to 8 inches has only minor effect. It is not practical to counteract loss of slab support by increasing slab thickness.



(a)



(b)



(c)

Figure A13 (a-c). EverFE slab stress predictions for top surface at transverse joints versus increasing loss of joint support at outer corners (a); slab deflections (b) and contour plot (c) for the 6 inch un-bonded overlay for a joint corner gap of 100 mils prior to loading.

3.0 Summary of Major Findings for West Branch

- Pumping erosion due to insufficient drainage was found to be the major cause of distress that has developed in Sections 3 and 4. Construction related factors are the main cause. 1) The leaching trench drain was omitted at the outside shoulder during construction as soil boring tests showed sand soil condition, and 2) the old dense-graded HMA shoulder was not removed prior to construction of the overlay. 3) The open-graded HMA (SL) was found to extend into the shoulder, butting up against the old HMA, thus preventing trapped water from draining vertically. 4) The concrete overlay in the shoulder was about ¼ inch higher than the outside lane. 5) The outside lane-shoulder joint was left unsealed. Thus, the open-graded HMA has served as a reservoir for trapped water.

- Dowels in Section 5 were found to be effective in reducing pumping effects (joint faulting and slab rotation) although erosion in the HMA is developing along the outer edge.

- Sections 1 and 2 have not developed distress in the truck lane due to being constructed on a super elevation sloping downward in the direction of inside lanes and on a steep uphill slope.

- Pavement roughness as measured by IRI is increasing rapidly for Sections 3 and 4. Due to a strong correlation between increasing roughness and pumping distress such as joint/crack faulting, the increase in IRI is a good parameter for determining the need for repair. Individual IRI data-points, which are calculated every 0.1 mile in the OWP and IWP on a yearly or bi-yearly basis, can be used to identify the locations within a project that have poor drainage and thus need further investigation.

Recommendation

- It is recommended to create a vertical drainage path along the outside lane-shoulder. This will slow the pumping mechanism and reduce further deterioration.

APPENDIX B AIR-VOID TESTING BY ASTM C457

Table B1 Air-Void test results for the concrete field samples

Project	Station Number	Point count results			Linear traverse results				
		Total air, %	paste content, %	Infilling	Entrained air, %	Entrapped air, %	Total air, %	Spacing factor(in.)	comment
US 131 NB (Rockford)-1	27+631.3	2.92	27.49	insignificant	2.01	0.79	2.80	0.0082	Poor F-T performance
US 131 NB (Rockford)-2	27+703	3.00	23.30	insignificant	2.7	0.15	2.85	0.0057	
US 131 SB (Rockford)-1	34+024.5	2.08	24.53	insignificant	1.9	0.24	2.14	0.0075	
US 131 SB (Rockford)-2	34+256.7	3.15	23.95	insignificant	2.08	1.32	3.40	0.0095	
US131 NB (Plainwell) (stain)-1	11+030.1 (top)	5.08	30.60	1.25%	3.85	1.13	4.98	0.0067	Stained specimens have more infillings
US131 NB (Plainwell) (stain)-2	11+030.1 (mid)	5.15	26.80	1.00%	4.23	1.28	5.51	0.0056	
US131 NB (Plainwell) (no stain)-1	10+990.8	4.31	25.20	0.33%	3.21	0.93	4.14	0.0056	
US131 NB (Plainwell) (no stain)-2	11+319.5	3.00	25.60	0.42%	3.07	0.31	3.38	0.0053	
US131 NB (Kalamazoo)-1	1122+03	5.92	24.10	0.83%	4.85	1.3	6.15	0.0062	Good air void distribution
US131 NB (Kalamazoo)-2	1120+15	6.23	23.30	-	4.86	2.03	6.89	0.0062	
US131 SB (Kalamazoo)-1	1125+18	3.77	24.10	1.00%	3.62	0.57	4.19	0.0072	
US131 SB (Kalamazoo)-2	1123+03	5.69	23.40	0.75%	3.85	1.55	5.40	0.0078	
I96 EB (Portland)-1	861+91	10.08	21.60	0.67%	9.34	1.55	10.89	0.0036	Moderate to servere air void clustering
I96 EB (Portland)-2	861+49	6.46	23.20	0.75%	6.89	0.34	7.23	0.0040	
I96 WB (Portland)-1	1193+26	8.23	23.60	insignificant	7.28	2.21	9.49	0.0040	moderate air void clustering
I75 NB (Westbranch)-1	792+20	6.31	21.50	insignificant	4.56	2.2	6.76	0.0047	Good air void distribution
I75 NB (Westbranch)-2	708+35	6.38	23.70	insignificant	5.29	1.8	7.09	0.0060	
US10 WB-1	617+42	4.85	26.70	insignificant	4.37	0.67	5.04	0.0050	Good air void distribution
US10 WB-2	659+97	5.92	25.10	0.85%	3.86	2.29	6.15	0.0068	
I69 NB (Charlotte)-1	87+730	3.44	27.90	insignificant	2.65	0.60	3.25	0.0077	Moderate air void clustering
I69 NB(Charlotte)-2	87+726.1	3.23	24.70	insignificant	2.88	0.66	3.54	0.0083	
US23 NB (north of M59)-1	43+983.3	6.88	22.80	insignificant	5.52	1.96	7.48	0.0042	Good air void distribution
US23 NB (north of M59)-2	43+997.4	6.50	22.20	insignificant	5.08	1.78	6.86	0.0056	