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Dowel Bar Retrofit Performance in Wisconsin

FINAL REPORT

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Research Study # WI-98-05

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16. Abstract

In 1999, WisDOT constructed test sections on I-39 to evaluate the dowel bar retrofit (DBR) rehabilitation technique for faulted concrete pavement slabs. Two years later, mortar deterioration and debonding were noted in the dowel slots. In response to this early distress, additional test sections were constructed on STH 13 to evaluate various mortar materials. The I-39 project was repaired, and test sections from both the I-39 and STH 13 projects were monitored between 2001 and 2007 for distress (PDI), pavement smoothness (IRI), and load transfer efficiency (LTE) between adjacent slabs. Additional DBR projects on USH 45, STH 21, and USH 18/151 were also surveyed in 2010.

Six years after repairs were made on the I-39 project, distressed dowel slots were noted again, and additional repairs were made. The entire project was eventually overlaid with HMA. Prior to the overlay, however, IRI values were low for sections with DBR. Smoothness varied for control sections that were diamond ground only. The control sections with a 3-in HMA overlay and with diamond grinding only had the roughest ride in the driving lane. DBR sections had consistently better LTE values than non-doweled sections. Among the STH 13 test sections, mortar with 100 percent extension ratio had more debonding and surface deterioration than mortar with lower extension ratios (60 or 80 percent). Mortar and joint deterioration occurred on the USH 45, STH 21, and USH 18/151 projects, but areas of very good performance were noted as well.

While DBR is a more expensive rehabilitation option than diamond grinding or HMA overlay, it addresses the root cause of slab faulting and provides the longest service life. If slab faulting is severe in the driving lane and not in the passing lane of a multi-lane highway, it may be possible to perform DBR in the driving lane only. Additionally, use of quality materials and attention to details during construction are critical for long-term performance of DBR projects.

17. Key Words Dowel bar retrofit, DBR, diamond grinding, load transfer efficiency, slab faulting, concrete pavement rehabilitation		**18. Distribution Statement** Distribution unlimited, approved for public release	
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1. Introduction

Prior to the mid-1970s, most Portland cement concrete (PCC) pavements constructed in Wisconsin incorporated steel dowel bars at joints to maintain load transfer between adjacent slabs. Later in the 1970s, it was hypothesized that the forces of aggregate interlock at PCC joints would be enough to provide adequate load transfer between adjacent slabs, and PCC pavements were constructed without dowel bars. The aggregate interlock was not strong enough, however, and slabs in these pavements quickly became faulted. In the mid-1980s, the Wisconsin Department of Transportation (WisDOT) returned to the practice of using dowel bars during the construction of PCC pavements.

This left 10 to 15 years' worth of non-doweled PCC pavements that needed rehabilitation when slab faulting caused the ride to be too rough for travelers. Diamond-grinding and/or hot mix asphalt (HMA) overlays were the rehabilitation strategies typically used, but slab faulting and associated distresses soon returned.

In the 1990s, dowel bar retrofit (DBR) was an emerging rehabilitation strategy for non-doweled PCC pavements that were in good structural condition but had experienced slab faulting. This technique involved placing dowel bars in sawed slots at PCC joints, and placing mortar or concrete mixture in the slots to cast the dowels in place. To keep the cost of this process low, the retrofit was only performed in the wheel paths, where load transfer is most critical.

In 1999, WisDOT began a research project to evaluate the DBR rehabilitation technique. Several test sections were constructed on a large DBR project on Interstate Highway 39. Within two years, mortar deterioration and spalling were noted in the test sections and along the length of the project. In some areas, the mortar loss was severe. This early distress prompted the issuance of a moratorium for DBR projects in Wisconsin. The distressed dowel slots on I-39 were repaired or, in areas with particularly severe distress, overlaid with HMA.

Because the primary distress noted on the I-39 project involved the quality of the dowel slot fill material, further research of mortar used for DBR was proposed. In 2001, several test sections with different mortar materials were constructed on a DBR project on State Trunk Highway (STH) 13 that was let prior to the moratorium. Based on good early performance of these test sections, the DBR moratorium was lifted in 2004, and a performance warranty provision was created to ensure quality DBR construction.

Many additional roadways in Wisconsin have received DBR rehabilitation, three of which were monitored as part of this study. Test sections were constructed on U.S. Highway (USH) 45 in 2002. Several DBR projects on USH 18/151 and STH 21 were also monitored, although formal test sections were not included at these sites. DBR on USH 18/151 was performed between 1999 and 2009, and in 2004 on STH 21.

The DBR test sections on I-39 were monitored until 2007, when the entire length of the DBR project was overlaid with HMA. The STH 13, USH 45, STH 21 and USH 18/151 projects were monitored until 2010. This report contains documentation of the performance of the projects’ test sections, a description of current WisDOT DBR practices, and recommendations for future DBR construction in Wisconsin.

2. Study Description

2.1 Motivation

As described in the Introduction, this research study was initially created to evaluate aspects of the DBR construction process and to determine whether DBR is a cost-effective rehabilitation technique for faulted non-doweled PCC pavements. Additional test sections evaluating specific types of dowel bar slot fill material were constructed after early distresses were noted in the mortar used in the initial test sections. Results of this study are important, as there are several hundred additional lane-miles of non-doweled PCC pavement in Wisconsin that will eventually need rehabilitation to restore ride quality.

2.2 Objectives

The following objectives were defined at the beginning of this study:

1. Conduct field evaluations of test and control sections to document the relative performance characteristics of each;
2. Evaluate DBR construction procedures and identify those that provide optimum results;
3. Determine if DBR is a suitable and cost-effective rehabilitation method for non-doweled concrete pavements; and
4. Review the current WisDOT DBR guidelines and procedures and determine their suitability for future projects.

2.3 Project Locations

The I-39 DBR test sections were proposed as part of a 100 lane-mile DBR construction project on the state's central north-south interstate corridor. The overall project extended from the Columbia/Marquette County line to STH 54 in Portage County. The roadway is a four-lane divided limited access interstate expressway. The test sections were constructed in Waushara and Marquette Counties (Figure 1).

The STH 13 test sections were integrated into a DBR rehabilitation project in the city of Marshfield in Wood County. The project extended from 1125 ft north of USH 10 (southern limit) to 1643 ft north of 26th Street (northern limit). The highway is a four-lane divided urban roadway. The test section location is shown in Figure 2.

The USH 45 test sections were constructed on a ¾-mile long project in the city of Clintonville in Waupaca County. The highway is a four-lane urban roadway with a center median. The location of this project and test sections is shown in Figure 3.

The STH 21 DBR rehabilitation project extended from the city of Redgranite to STH 49 in Waushara County. The roadway is a two-lane urban highway in the west and a two-lane rural highway in the east. The location of this project is shown in Figure 4.

The evaluation area on USH 18/151 is located between Dodgeville in Iowa County and Mount Horeb in Dane County. Approximately 20 miles of non-doweled PCC pavement were rehabilitated during multiple DBR projects over a 10-year period. The location of this evaluation area is shown in Figure 5.

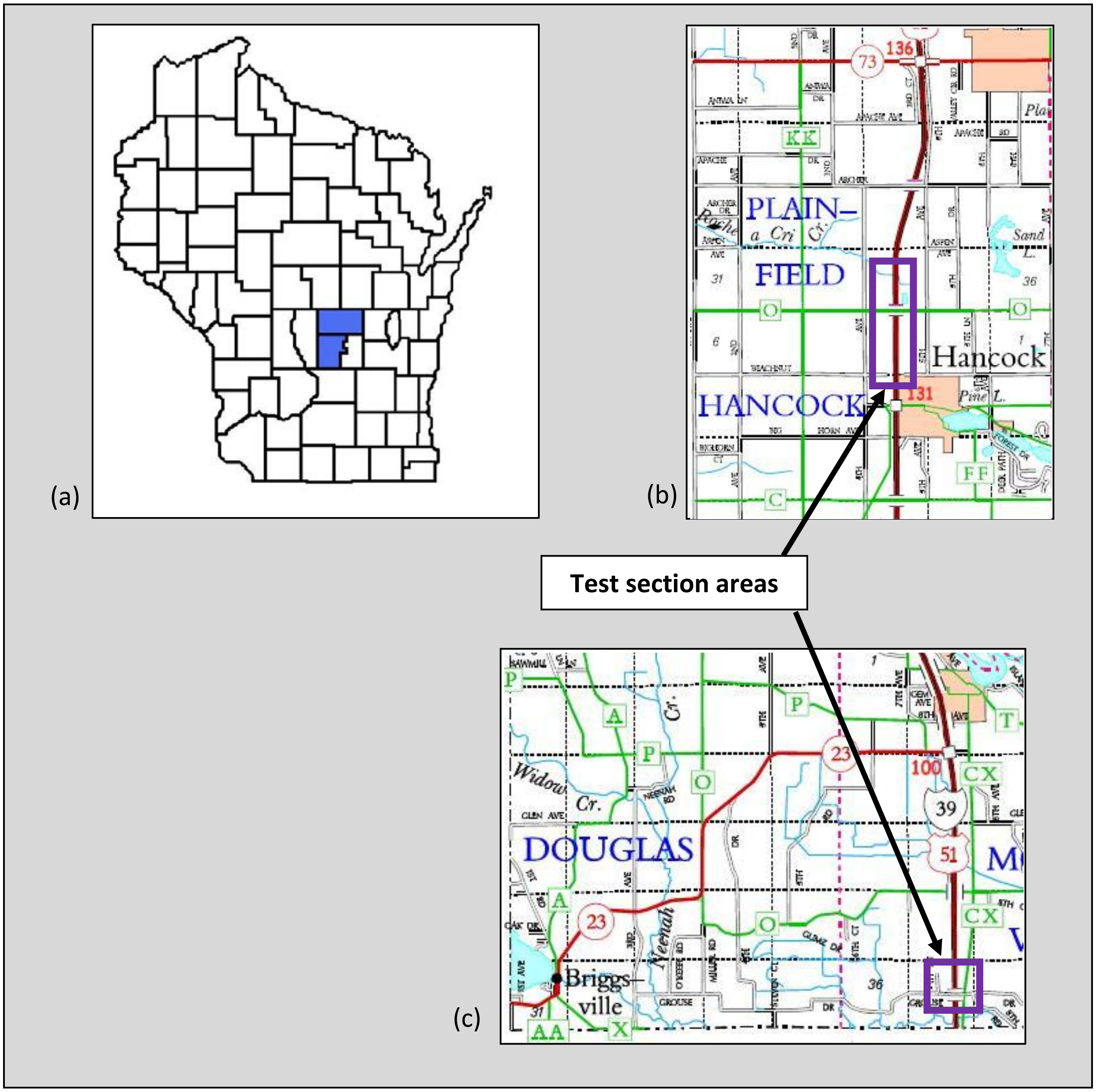

Figure 1. Location of I-39 test sections: (a) Waushara and Marquette Counties; (b) test section area in Waushara County; and (c) test section area in Marquette County.

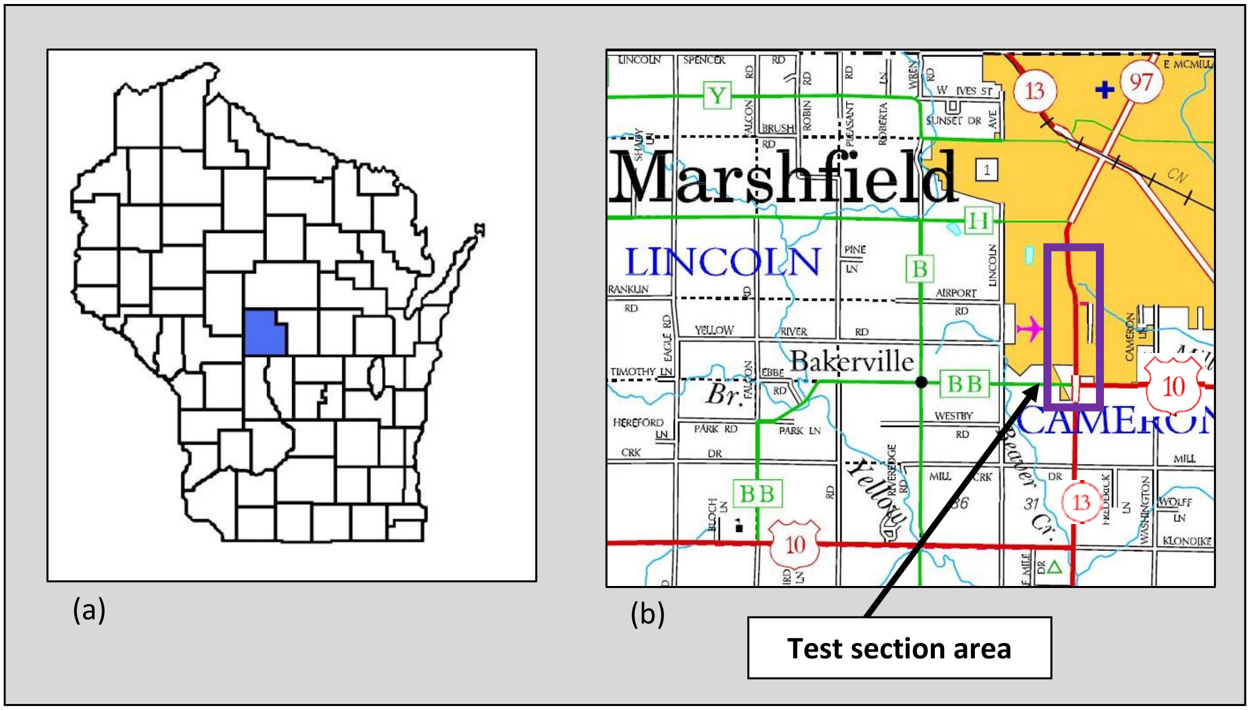

Figure 2. Location of STH 13 test sections: (a) Wood County; and (b) test section area.

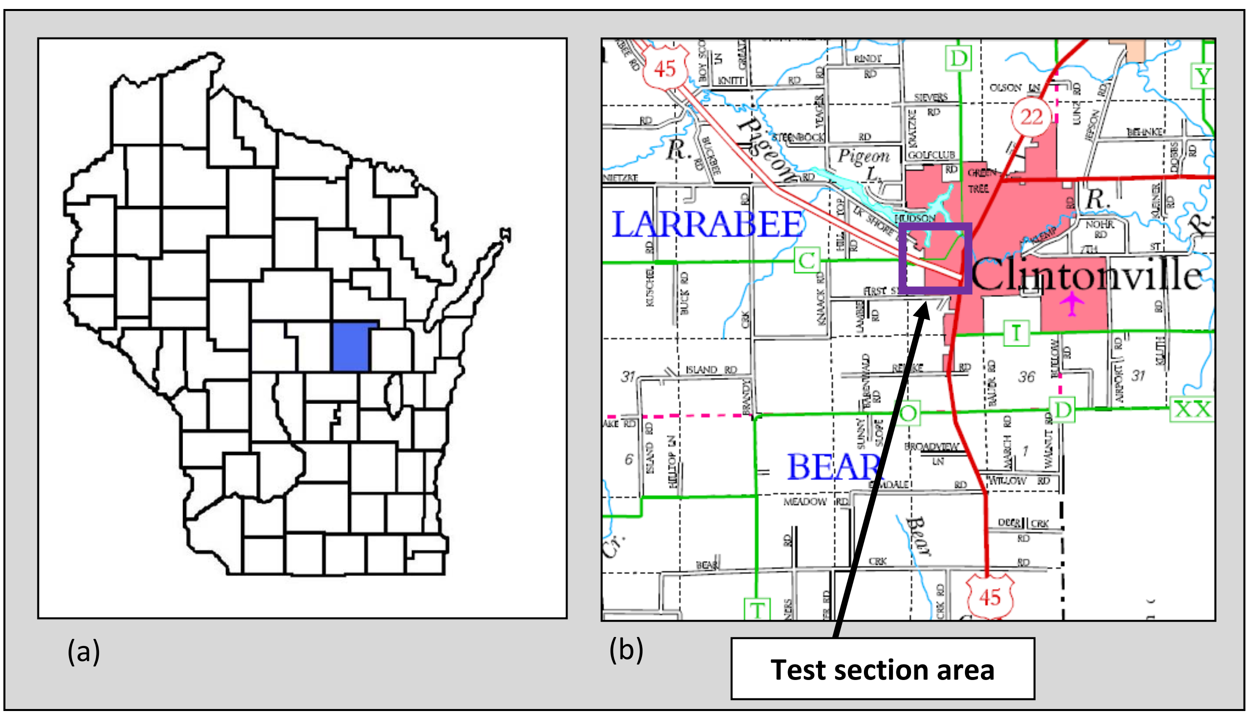

Figure 3. Location of USH 45 test sections: (a) Waupaca County; and (b) test section area.

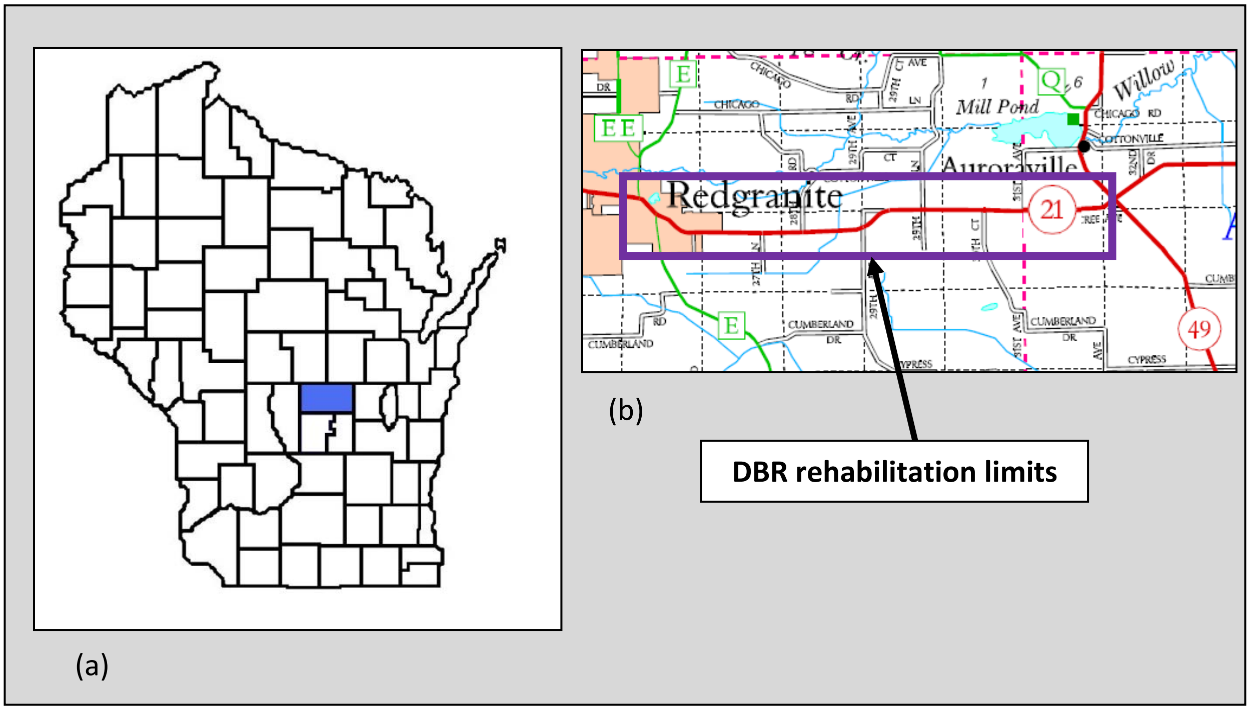

Figure 4. Location of STH 21 evaluation area: (a) Waushara County; and (b) limits of DBR rehabilitation.

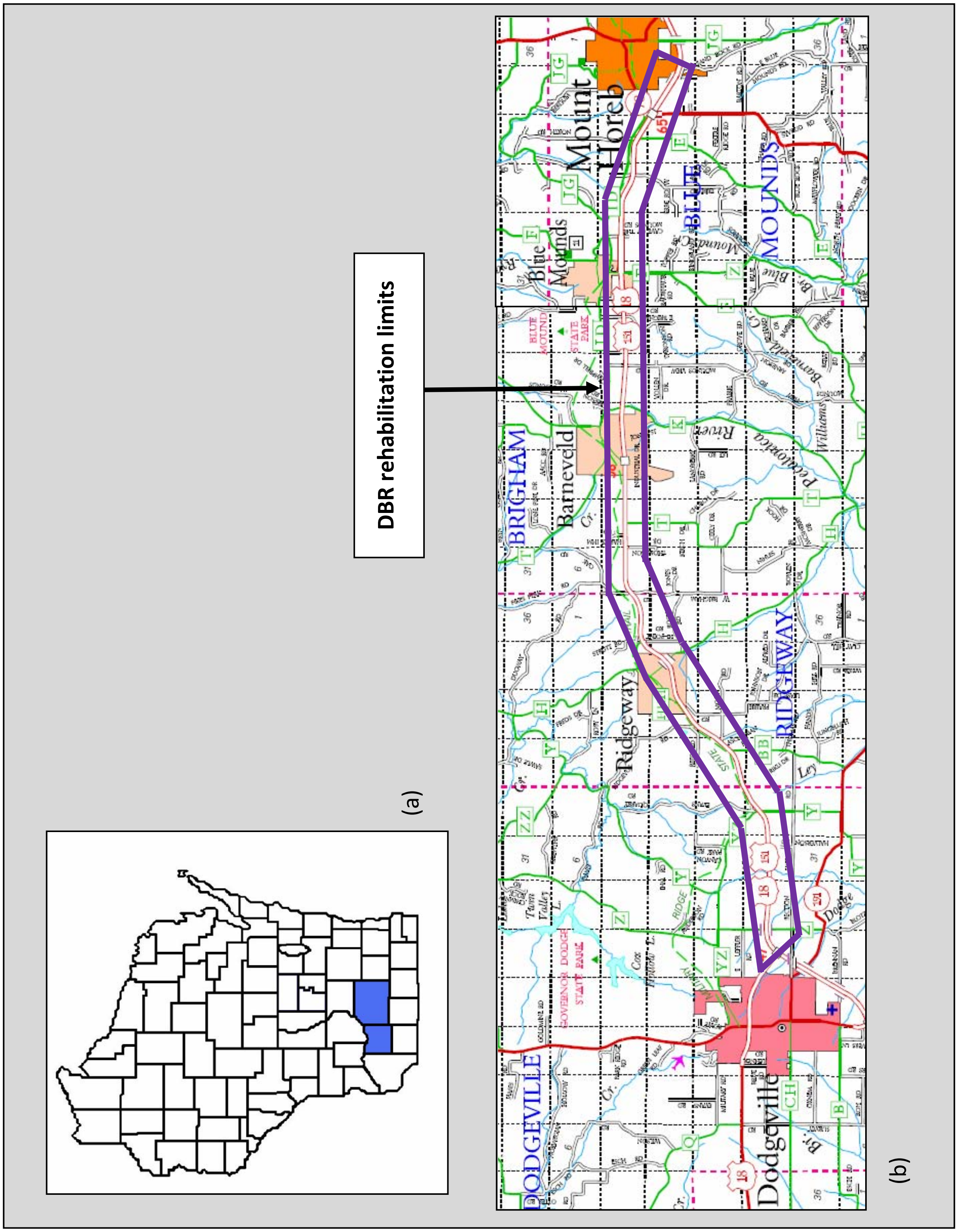

Figure 5. Location of USH 18/151 evaluation area: (a) Iowa and Dane Counties; and (b) limits of DBR rehabilitation.

2.4 Test Section Descriptions

2.4.1 Interstate 39

Four DBR design strategies were studied at the I-39 test site. Redundant test sections were constructed for some of the designs, resulting in a total of ten test sections. The driving and passing lanes (DL and PL) in all test sections were diamond ground after dowel installation. Two control sections were constructed: one with a 3-in HMA overlay, and one with only diamond grinding in both lanes. Test section details are provided in Table 1. The test section layout is shown in Appendix A.

Table 1. I-39 Test Section Descriptions

Test Section	Length (ft)	County	Direction	Dowel Length (in)	DBR in DL	DBR in PL	Mortar Type	Extension Rate
1A	1000	Marquette	NB	18	Y	N	Thoroc 10-60	100%
1B	1000	Waushara	NB	18	Y	N	Thoroc 10-60	100%
1C	1000	Waushara	NB	18	Y	N	Thoroc 10-60	100%
1D	1000	Waushara	SB	18	Y	N	Thoroc 10-60	100%
2A	1000	Marquette	NB	18	Y	Y	Thoroc 10-60	100%
2B	1000	Waushara	NB	18	Y	Y	Thoroc 10-60	100%
3A	1000	Waushara	SB	12	Y	N	Thoroc 10-60	100%
3B	125	Waushara	SB	12	Y	N	Thoroc 10-60	100%
4A	2000	Waushara	SB	15	Y	N	Thoroc 10-60	100%
4B	125	Waushara	SB	15	Y	N	Thoroc 10-60	100%
C1	1000	Waushara	NB	3-in HMA overlay				
C2	1000	Waushara	NB	Diamond grind only				

2.4.2 STH 13

Fifteen test sections were constructed at the STH 13 test site. These sections investigated several mortar mixture designs used to fill the dowel bar slots. Three mortar brands were studied along with an air-entrained concrete mixture specified by the Minnesota DOT (MnDOT). Various extension rates were tested; this parameter is the ratio of the weight of coarse aggregate to the weight of mortar mix. Most test sections were constructed with 18-in long, 1.25-in diameter epoxy-coated (E.C.) steel dowel bars. In one test section, 15-in long, 1.25-in diameter stainless steel (S.S.) clad dowel bars were used. Three control sections were constructed (designated C1, C2, and C3); these sections were diamond-ground only (no DBR). Details of the test sections are provided in Table 2. The test section layout is shown in Appendix A.

Table 2. STH 13 Test Section Descriptions

Test Section	Length (ft)	Mortar Type	Extension Rate	Dowel Bars	Sealed Joints
A	2264	MnDOT 3U18	N/A	E.C.	Y
B	2272	MnDOT 3U18	N/A	E.C.	N
C	2256	AHT	60%	E.C.	N
D	2263	AHT	60%	E.C.	Y
E	1921	AHT	100%	E.C.	Y
F	769	AHT	100%	E.C.	N
G	305	Thoroc 10-60	60%	E.C.	N
H	2263	Thoroc 10-60	60%	E.C.	Y
I	550	Tamms Speed Crete 2028	100%	E.C.	Y
J	2787	Tamms Speed Crete 2028	80%	E.C.	Y
K	1190	Thoroc 10-60	60%	E.C.	N
L	3392	Tamms Speed Crete 2028	80%	E.C.	N
M	1136	Tamms Speed Crete 2028	80%	S.S.	N
N	3337	AHT	100%	E.C.	Y
O	1396	Thoroc 10-60	60%	E.C.	N
C1	3797	No DBR			N
C2	3797	No DBR			N
C3	525	No DBR			N

2.4.3 USH 45

The DBR test sections constructed on USH 45 utilized two brands of mortar. Tamms Speed Crete was tested at two extension rates (60 and 80 percent), and Master Builder's Set 45 was tested at 60 percent extension. Two curing methods were tested; the first was according to the WisDOT standard specification, and the second was according to the MnDOT specification. The WisDOT curing procedures specify a water-based, wax-based curing compound, while the MnDOT specification calls for a poly-alphamethylstyrene (AMS) curing compound. In the cases of test section pairs 1 & 2 and 7 & 8, the DBR slots in the right and left wheel paths were cured using these different methods. Details of the test sections are provided in Table 3.

Table 3. USH 45 Test Section Descriptions

Test Section	Length (ft)	Direction	Lane, Wheel Path	Mortar Type	Extension Rate	Water (pts)	Cure Method Specification
1	3250	WB	DL, right	50 lbs Tamms Speed Crete	60%	5.25	WisDOT
2	3250	WB	DL, left	50 lbs Tamms Speed Crete	60%	5.25	MnDOT
3	3250	WB	PL, right	50 lbs Tamms Speed Crete	80%	6.00	WisDOT
4	3250	WB	PL, left	50 lbs Tamms Speed Crete	80%	6.00	WisDOT
5	3250	EB	DL, right	50 lbs Tamms Speed Crete	80%	6.00	WisDOT
6	3250	EB	DL, left	50 lbs Tamms Speed Crete	80%	6.00	WisDOT
7	3200	EB	PL, right	50 lbs Master Builder's Set 45	60%	4.00	MnDOT
8	3200	EB	PL, left	50 lbs Master Builder's Set 45	60%	4.00	WisDOT

3. Construction

The overall 100 lane-mile DBR rehabilitation project on I-39 was constructed in several phases during 1999 and 2000. The project phases were let under three state project I.D.s: 1160-00-60, 1160-01-61, and 1160-01-62. Existing pavement consisted of 9 inches of non-doweled jointed plain concrete pavement (JPCP) over 6 inches of crushed aggregate base course (CABC). Joints were skewed and randomly spaced. The test and control sections were constructed in 1999. Gang saws were used to cut outlines for three dowel bar slots into the wheel paths, and the PCC material was removed with a jackhammer. Dowel bars were placed in the slots using chairs to maintain the appropriate height and straight placement. Foam board was inserted at the joint in each slot to maintain the joint opening. Dowel slots were filled with ThoRoc 10-60C mortar mix at 100 percent extension rate. After the mortar had cured, transverse joints were re-sawed to remove all patching material within the joint. After all slots were finished, the entire lane was diamond-ground to restore a smooth ride. Existing joint sealant was left in place or replaced if necessary. More information is available in the Department's Report of Early Distress for the I-39 project. [*1*]

The STH 13 DBR rehabilitation project was constructed in the summer of 2001 under state project I.D. 1620-00-60. Existing pavement in control sections 1 and 2 consisted of a 2.75- to 3.875-in bonded concrete overlay (1986 construction) over 9 inches of non-doweled JPCP (1949 construction). Joints in these lanes were perpendicular and randomly spaced. Existing pavement in the driving lanes adjacent to the control sections was constructed in 1986 with 8 inches of non-doweled JPCP over 6 inches of CABC over an 18-in granular subbase. Joints in this area were also perpendicular and randomly spaced. The remainder of the test section pavement north of station 248+54 and south of station 210+66 (see Figure A-2 in Appendix A) was constructed with new 8-in JPCP over 6 inches of CABC over 18 inches of granular subbase. The joints in these pavement sections were also randomly spaced but were skewed 6:1, right-hand forward. DBR construction methods similar to those described above were used for this project. Various mortar materials were used for filling the dowel slots, and joints were sealed or left unsealed as described in Section 2.4.2. More detailed information on the STH 13 DBR construction process is available in the Construction and One-Year Performance report published for this study. [*2*]

DBR construction on USH 45 took place in 2004 under state project I.D. 1142-00-60. The existing pavement, constructed in 1988, was 9 inches of non-doweled JPCP. Joints were perpendicular and spaced at 20 ft. Joints were sealed during initial construction and were re-sealed after DBR construction.

The STH 21 DBR rehabilitation project was constructed in 2004 under state project I.D. 6180-04-60. The existing pavement was constructed in 1987 and consisted of 8 inches of non-doweled JPCP over 6 inches of CABC. Joints were skewed and randomly spaced. Joints were sealed during initial construction and were re-sealed after DBR construction. The project contract included a 3-year warranty provision for the DBR construction and materials.

DBR construction on USH 18/151 took place under several state rehabilitation projects between 1999 and 2009. Existing concrete pavement in these areas was constructed between 1982 and 1985 (westbound lanes) and from 1988 to 1989 (eastbound lanes). Joints were skewed, randomly spaced, and sealed.

Using the WisDOT Pavement Information Files, performance data were gathered for the construction projects described above. Pavement distress index (PDI) and international roughness index (IRI) were noted before and after DBR construction took place. These data are summarized in Table 4. Data were not available for the STH 13 and USH 18/151 projects. The pavement smoothness, as measured by IRI, was improved for all projects after DBR was completed. The PDI was improved in most cases.

Table 4. PDI and IRI Before and After DBR Rehabilitation

		PDI		IRI	
Highway	**Direction**	**Before**	**After**	**Before**	**After**
I-39 (Waushara County)	NB	14	0	1.84	0.67
I-39 (Waushara County)	SB	12	0	1.57	0.75
I-39 (Marquette County)	NB	24	0	1.36	0.85
USH 45	NB	6	6	1.65	0.94
USH 45	SB	6	4	3.52	1.28
STH 21	EB	6	0	2.32	0.90
STH 21	WB	6	0	2.10	0.93
STH 13 USH 18/151		Data not available			

4. Test Section Performance

4.1 I-39

In 2001, approximately two years after DBR construction on I-39, severe mortar deterioration and material loss was noted in the dowel bar slots. The distresses were noted along the entire length of the construction project. Because of this early distress, a moratorium was issued for DBR projects in Wisconsin. The problem was investigated and reported in a Department Report of Early Distress. [*1*] The moratorium was lifted in 2004, when the Department implemented a performance warranty program for DBR projects (see Section 6). Many deteriorated dowel bar slots on the I-39 DBR project were repaired by sandblasting the slots to remove inferior mortar material and refilling the slots with an epoxy-based concrete fill material (EP35). A section in the southbound lanes with particularly severe mortar distress was overlaid with HMA. Due to continued mortar deterioration, a second repair project was completed on the I-39 DBR project in the fall of 2006 and spring of 2007. The test sections remained intact and were surveyed until 2007, when the entire length of the original DBR project was overlaid with HMA.

4.1.1 Freeze-Thaw Testing

Freeze-thaw testing was conducted in 2001 according to ASTM C 666 on samples of dowel slot fill material collected from cores of the I-39 test section pavement. Test results were mixed, with some samples showing little or no mass loss after 600 freeze-thaw cycles, while five of the eleven cores tested completely disintegrated (100 percent mass loss) before the test was complete. [*1*]

Tests conducted on samples of the same mortar fill material but with a lower extension rate (80 versus 100 percent) performed better. It was concluded that lack of freeze-thaw durability was a major factor contributing to the early distress noted on the I-39 test sections. [*1*]

4.1.2 Automated Performance Surveys

Automated pavement performance surveys of the I-39 test sections were conducted annually from 2001 to 2007. Surveys for PDI and IRI were performed by the WisDOT Pavement Data Unit, using the Department's automated video surveying and profiling equipment. Data from the 2007 automated PDI and IRI surveys are shown in Figures 6 and 7, respectively. Data for all test years are provided in Appendix B. Note that surveys were not performed in the passing lanes for test sections 1D, 3A, 3B, 4A, and 4B. The test sections had been in service for approximately eight years in 2007.

Results from the PDI survey indicated that very little distress was present in any of the test sections after eight years in service. However, some DBR distresses, such as mortar debonding and deterioration, are difficult to measure with the automated survey equipment. In-person surveys showed that the epoxy-based patch material was holding up well, but further mortar deterioration was taking place in the dowel slots; because of this, dowel slot repairs, and eventually an HMA overlay, were completed as described above.

The pavement in control section 1 (3-in HMA overlay) displayed the highest level of distress in 2007. The relatively high PDI value (28.0) indicates that many of the faulted PCC slabs had reflected through the HMA pavement, resulting in cracks at the joint locations (Figure 6).

The automated IRI survey showed that the test sections had smooth riding surfaces after eight years in service, with an IRI value of less than 1.0 m/km in most surveyed test sections (Figure 7). The PLs in test sections 1C and 2B had comparatively high IRI values (1.34 and 1.44 m/km, respectively). The PL in section 1C did not have DBR rehabilitation, and the PL in section 2B did have DBR; the section 2B PL did not match observed trends in other sections. Other surveyed PLs with (2A) and without (1A, 1B) DBR performed as well as the DLs with DBR. It is possible that isolated joints or other bumps in the 1C and 2B PLs resulted in relatively high IRI values. It was also noted in a Washington State study that DBR rehabilitation was more effective at maintaining a smooth riding surface if the initial faulting was lower. [*3*] It is possible that faulting in section 2B was higher at the time of DBR, resulting in higher IRI measurements after eight years in service.

Both control sections had higher IRI values than the test sections (Figure 7). The rough ride in control section 1 is likely a result of cracks that were also apparent in the PDI survey results. Control section 2, which received diamond grinding only as a rehabilitation strategy, also had higher IRI levels than the test sections. It is unknown why IRI values were higher in section C2 than in the PLs of sections 1A and 1B, which also received diamond grinding only. The control sections' DL IRI results were higher than in the PLs (Figure 7). This outcome is typical, as the DL is subjected to more truck traffic (i.e. heavier loads) than the PL.

An objective of this study was to determine whether DBR in the driving lane and diamond-grinding only in the passing lane was as effective as DBR in both lanes. For this analysis, it was necessary to look at IRI data for test sections that were surveyed in both the driving and passing lanes: 1A, 1B, 1C, 2A, and 2B. Control section 2 was also evaluated, as it had diamond grinding in both lanes. The PL in section C2 had lower IRI values, and therefore less faulting, than the DL. This is expected, as the PL is not subjected to as many heavy truck loadings as the DL. The DL and PL in sections 1A and 1B performed at approximately equal levels, indicating that using DBR in the DL and diamond grinding in the PL is an effective strategy. However, the IRI in the PL of section 1C was relatively high, suggesting that diamond grinding only in the PL is not effective. The DL and PL performed approximately equally in section 2A, but the PL had a high IRI value in section 2B; this data suggests that DBR in both lanes is not necessary. Because of the conflicting data after eight years in service, it is difficult to determine whether DBR in the DL and diamond grinding in the PL is as effective a rehabilitation strategy as DBR in both lanes. A longer-term study would possibly highlight more clearly any differences among these test sections. This is not an option in this case, however, as the project has been overlaid with HMA.

Test sections 3A, 3B, 4A, and 4B had DBR performed using shorter dowel lengths (12 and 15 inches). After eight years in service, these sections had similar PDI and IRI performance. This suggests that shorter dowels could provide the same level of protection against joint faulting as 18-in long dowels, particularly in DBR projects (where dowel bars can be visually centered across the joints, as opposed to new construction projects, where joints are sawed after the dowel bars have been covered).

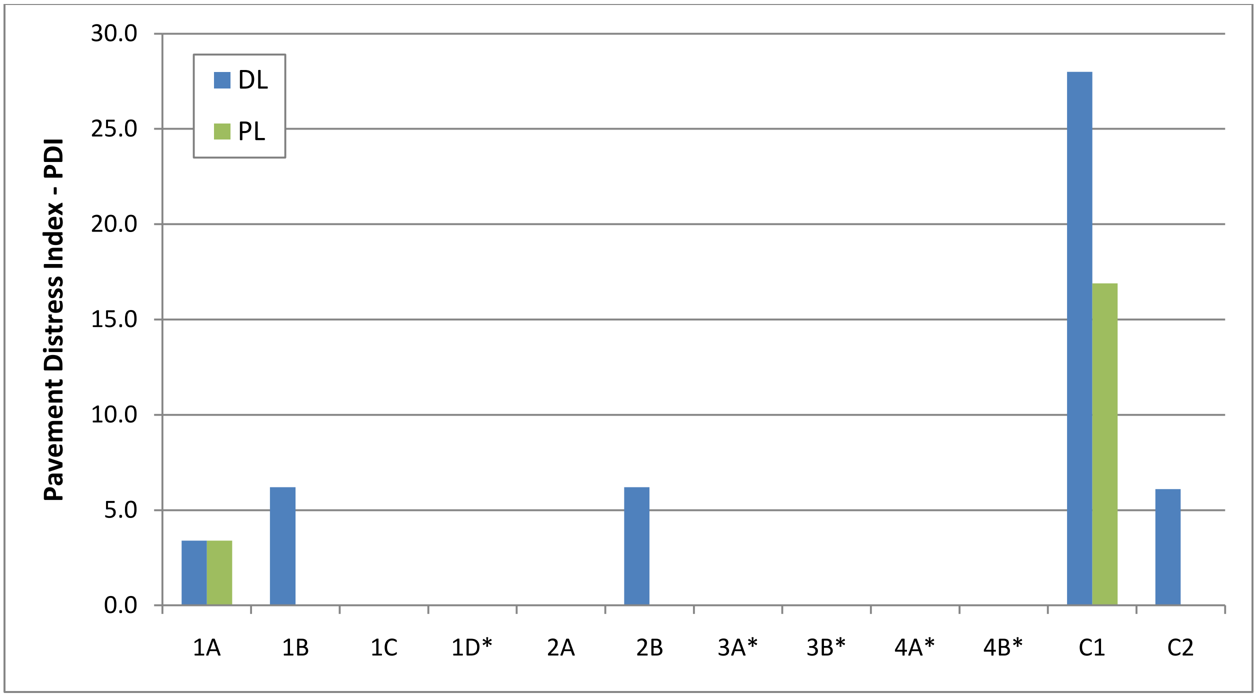

Figure 6. Pavement distress index results for I-39 test sections, 2007. *Denotes section was not surveyed in the PL.

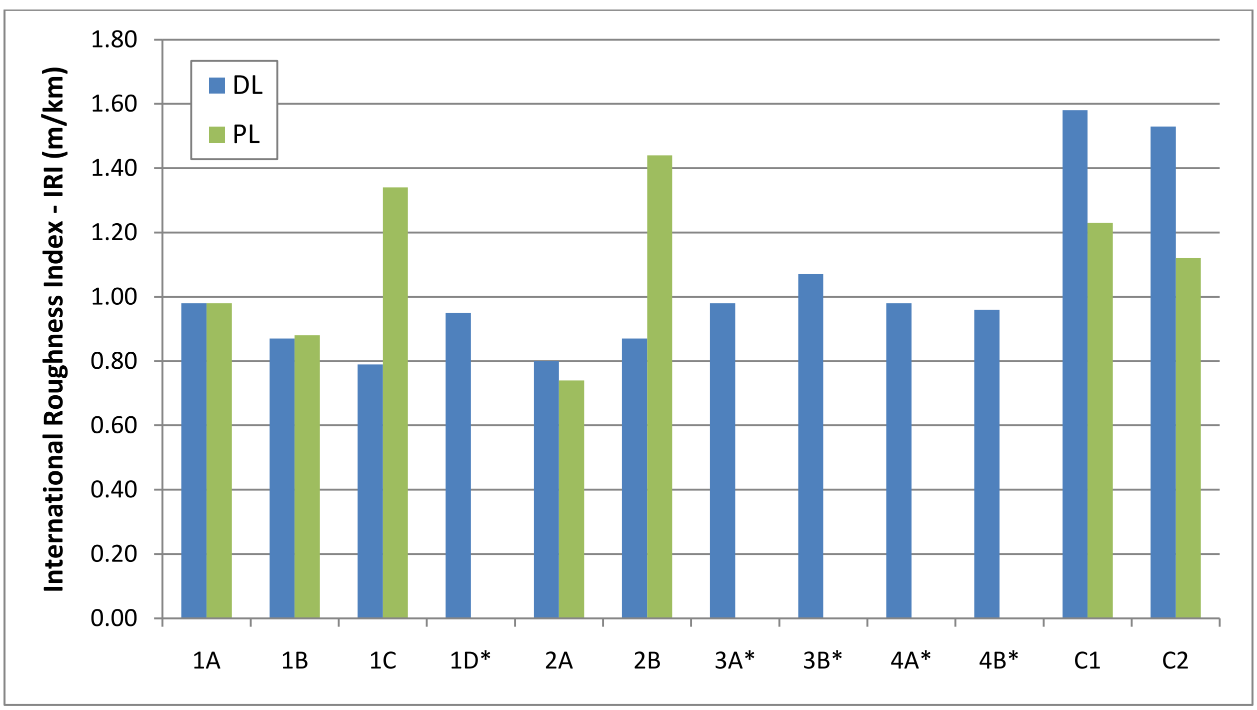

Figure 7. International roughness index results for I-39 test sections, 2007. *Denotes section was not surveyed in the PL.

4.1.3 Load Transfer Efficiency

Falling weight deflectometer (FWD) data was collected several times on the I-39 test sections to determine load transfer efficiency (LTE) of the joints. LTE is the ratio of slab deflection measured on the approach and leave slabs due to a load applied at the joint (Eq. 1). Slab deflection was measured with the WisDOT FWD equipment.

$$LTE = \frac{\Delta_a}{\Delta_l} \times 100\% \qquad \text{Eq. 1}$$

where: Δ_a = deflection of the approach slab

Δ_l = deflection of the leave slab

LTE is an indicator of whether two adjacent slabs are acting together and uniformly sharing traffic loads. If $\Delta_a = \Delta_l$, then LTE = 100 percent, which indicates that the two slabs are deflecting as one, and traffic loads are smoothly transferred between slabs. If $\Delta_a < \Delta_l$, then LTE < 100 percent; loads are not shared equally between slabs. When LTE is low (less than 70 percent), the unequal load transition can result in joint faulting.

Deflections for LTE calculation were measured in June 1999 (prior to the DBR operation), March 2001, July 2001, and May 2003. These data are presented in Figure 8 and Tables 4 through 5, respectively.

LTE testing conducted in June 1999 was performed in the northbound lanes of I-39, in the area of future test sections 1B, 1C, 2B, C1, and C2. Testing took place between 10:00 a.m. and 2:00 p.m., with temperatures ranging from 66 to 76°F. Four tests were conducted at each joint using FWD loadings of 5, 9, 14, and 20 kips; an average LTE was calculated for each joint tested. Most measurements were taken in the right wheel path (RWP) of the DL, and some were taken at the center of the DL. The majority of LTE measurements were in the 25 to 50 percent range, which indicates poor load transfer between adjacent slabs (Figure 8).

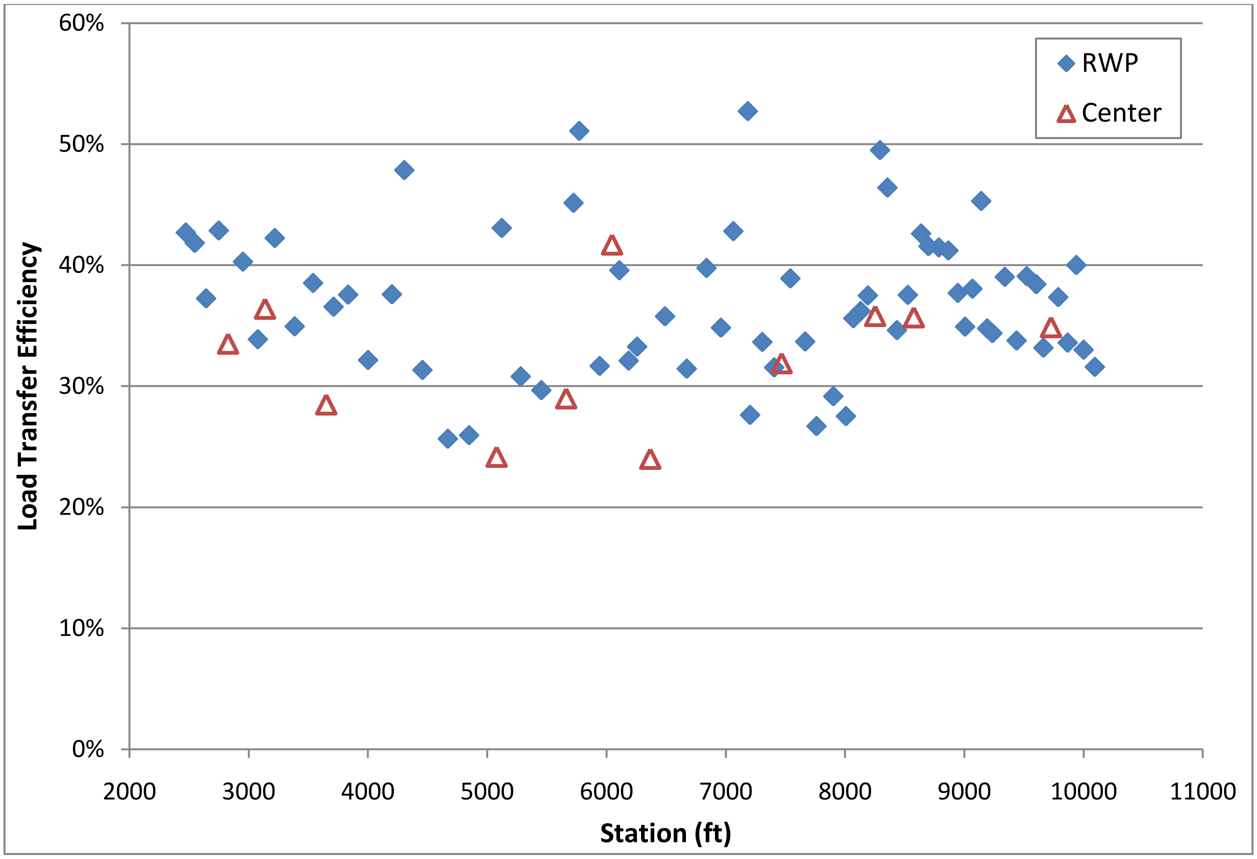

Figure 8. Load transfer efficiency measured at I-39 in June 1999 (prior to DBR).

Tests for LTE were repeated in March 2001, after mortar distress was noted in the dowel slots. Five areas were selected, and two to twelve distressed joints were tested under several FWD loadings in each area. Tests were conducted at either the right or left wheel path (RWP or LWP) location in the northbound and southbound (NB and SB) DLs and PLs. An average LTE was calculated for each test area; results from these tests are provided in Table 5. [*1*] Average LTE values were above 80 percent at all test locations, indicating that, despite mortar distress, the dowels provided good load transfer between slabs. LTE was greatly improved compared to before dowel bars were installed (Figure 8). Air and pavement temperature data were not available for the test date, but it was suspected that the subbase was frozen; this could lead to higher LTE values. [*1*]

Table 5. Load Transfer Efficiency, I-39, March 2001. Adapted from [*1*]

Test Area	Direction	Lane	Position	Doweled?	Average LTE (%)
1	NB	DL	RWP	Y	82
2	SB	PL	LWP	Y	88
3	SB	DL	RWP	Y	81
4	SB	DL	RWP	Y	87
5	SB	DL	RWP	Y	88

Follow-up testing was conducted in July 2001. Eight areas were selected, and two to twelve distressed joints were tested under four FWD loadings (5, 9, 12, and 20 kips) in each area. In addition, one non-doweled test area with no joint distress was tested. Tests were conducted in the RWP in the NB and SB DLs. An average LTE was calculated for each test area; results from these tests are provided in Table 6. [*1*] Average LTE values for doweled joints were in the 70 to 95 percent range, indicating adequate to very good load transfer. The average LTE for the non-doweled joints was 60 percent, which is lower than desired. Therefore, despite the distresses noted in the dowel slots, joints with DBR were performing better than non-doweled joints.

Table 6. Load Transfer Efficiency, I-39, July 2001. Adapted from [*1*]

Test Area	Direction	Doweled?	Average LTE (%)
1	NB	Y	81
2	NB	Y	81
3	NB	Y	94
4	NB	Y	92
5	SB	Y	74
6	SB	Y	72
7	SB	Y	82
8	NB	N	60

A final series of LTE testing was conducted in May 2003, after repairs had been made to deteriorated dowel slots. Tests were performed at three to five joints in six of the twelve DBR test and control sections. Four FWD loadings (5, 9, 12, and 20 kips) were applied to joints in either the RWP or the LWP in the DL or PL. An average LTE value was calculated for each test section (Table 7). The average LTE for the joints with DBR ranged from 75 to 83 percent, while the non-doweled joints had an LTE of approximately 30 percent. Therefore, after four years in service, the sections that received DBR out-performed the control section with a rehabilitation strategy of diamond-grinding only (no dowels). It is also interesting to note that joints in test section 3A, which had 12-in long dowels installed, had lower LTE values than joints with longer (15-in or 18-in) dowels.

Table 7. Load Transfer Efficiency, I-39, May 2003

Test Section	Direction	Lane	Position	Doweled?	Average LTE (%)
1B	NB	DL	RWP	Y	83
1D	SB	DL	RWP	Y	80
2B	NB	DL	RWP	Y	80
2B	NB	PL	LWP	Y	83
3A	SB	DL	RWP	Y	75
4A	SB	DL	RWP	Y	83
1B	NB	PL	LWP	N	27
C2	NB	DL	RWP	N	32
C2	NB	PL	LWP	N	33

4.2 STH 13

4.2.1 Freeze-Thaw Testing

Freeze-thaw testing was conducted according to ASTM C 666 on samples of the various mortar fill materials taken from cores of the STH 13 pavement. After 300 cycles, the MnDOT 3U18 samples and the samples of Tamms Speed Crete at 80 percent extension had no mass loss, and the Tamms Speed Crete at 100 percent extension had only 3 percent mass loss. The ThoRoc 1060 and AHT mortars at 60 percent extension performed the worst, with 20 to 35 percent mass loss. [*2*]

4.2.2 Automated Performance Surveys

Automated pavement performance surveys of the STH 13 test sections were conducted annually from 2001 to 2007. (No survey was conducted in 2005.) Surveys for PDI and IRI were performed by the Pavement Data Unit, using the Department's automated video surveying and profiling equipment.

Results from the PDI and IRI surveys are shown in Tables 8 and 9, respectively. Test sections A and M (MnDOT 3U18 concrete mix and Tamms Speed Crete at 80 percent extension) had relatively high 2007 PDI values of 26.2 and 18.8, respectively (Table 8). Multiple slab corner cracks, along with mortar deterioration, resulted in the high PDI measured in test section A. The PDI reported for test sections N and O, and control sections 2 and 3 was greater than 10.0. Because these four sections were combined into one survey section, however, it is difficult to determine whether one of the sections performed worse than the others. Test sections B through L had relatively low PDI values in 2007, indicating good performance of the DBR sections.

It should be noted that it is difficult to obtain reliable results from automated distress surveys of DBR projects. Many of the distresses that could cause problems in the dowel slots, such as debonding and thin shrinkage cracks, are difficult to discern in the automated survey. Equipment and software upgrades implemented by WisDOT in 2009 provide more detailed imaging, which may increase the accuracy of automated DBR surveys. However, in-person visual surveys are still recommended to provide the most comprehensive documentation of DBR distresses.

Results of the IRI survey showed that all test sections maintained a smooth riding surface after six years in service. The range of IRI values noted among the various test sections was 0.60 to 1.67 m/km (Table 9). This range represents only small differences in ride quality among test sections, and the values indicate good performance overall.

Table 8. PDI Survey Results for STH 13 Test Sections

Test Section	2001	2002	2003	2004	2005	2006	2007
A	6.2	6.7	20.0	20.9	No Data Taken	20.0	26.2
B	0.0	0.0	3.4	3.4		6.1	3.4
C	7.3	0.0	3.4	4.5		9.4	9.4
D	9.5	6.2	0.0	0.0		3.4	3.4
E	6.2	0.0	3.4	3.4		3.4	3.4
F, G, H	6.2	0.0	3.4	0.0		3.4	3.4
I, J, K	6.2	0.0	3.4	9.9		3.4	3.4
L	0.0	0.0	6.1	9.4		3.4	6.1
M	0.0	0.0	11.9	17.8		11.9	18.8
N, O, C2, C3	10.1	6.2	10.4	7.3		9.4	10.4

Table 9. IRI Survey Results for STH 13 Test Sections

Test Section	2001	2002	2003	2004	2005	2006	2007
A	1.18	1.10	1.03	1.15	No Data Taken	1.58	1.39
B	1.44	1.44	0.99	0.99		1.07	1.04
C	1.26	1.50	1.33	1.20		1.36	1.39
D	1.03	1.15	0.93	1.03		1.34	1.25
E	0.60	1.39	1.04	1.09		1.29	1.44
F, G, H	1.40	1.56	1.17	1.14		1.48	1.47
I, J, K	0.77	1.33	0.90	1.07		1.23	1.17
L	1.45	1.45	0.95	0.96		1.01	1.03
M	1.67	1.61	1.20	1.18		1.33	1.20
N, O, C2, C3	1.05	1.37	1.09	1.07		1.25	1.23

4.2.3 Visual Surveys

Visual distress surveys were performed for the STH 13 test sections in April 2007 and April 2010, six and nine years, respectively, after the DBR rehabilitation had been performed. Dowel bar slots were examined to identify distresses: shrinkage/debonding, cracking, surface deterioration, and mortar loss. The slabs were also inspected for cracking that originated at dowel bar slots (corner cracking), and joint deterioration. Results of these surveys are shown in Tables 10 and 11.

Test sections A and B (MnDOT 3U18 concrete mix) had the worst performance, with minor or significant debonding and mortar cracking after six years in service (Table 10) and mortar deterioration after nine years in service (Table 11). Slab corner cracks that originated at dowel slots were also noted in test section A. It is possible that the concrete used to fill dowel slots was not properly cured, which could result in the shrinkage and cracking noted in the slot fill material. It is also possible that the concrete mixtures were properly cured but had greater shrinkage susceptibility than expected. After six years in service, the slot surfaces were in good condition, with no deterioration or material loss (Table 10). This was not the case after nine years in service, however, at which time significant mortar deterioration was noted (Table 11).

Surface deterioration and mortar loss were significant problems in test sections E, F, and N (AHT mortar at 100 percent extension). It is suspected that 100 percent is too high an extension rate for the AHT mortar; test sections C and D, with AHT at 60 percent extension, performed better in the mortar loss and deterioration categories.

Test sections with very good or excellent mortar performance after nine years in service included sections G, M, and O, which used ThoRoc mortar at 60 percent extension and Tamms Speed Crete at 80 percent extension. No distresses were noted in these sections after six years in service, and only isolated instances of joint deterioration (test section G) and mortar loss (test section O) were noted after nine years in service.[1] Test section L (Tamms Speed Crete at 80 percent extension) also performed well, with isolated instances of mortar debonding after six and nine years in service. It should also be noted that none of these test sections had sealed joints, which might indicate that joint sealing is not necessary for good DBR performance.

Test section K (ThoRoc mortar at 60 percent extension) also had no distresses in 2007 (Table 10), but minor mortar deterioration was noted in 2010 (Table 11). Other materials that showed good performance were AHT at 60 percent extension and Tamms Speed Crete at 80 and 100 percent extension (sections D, H, I and J).

Sealed or unsealed joints did not impact performance of the DBR test sections. In addition, the test section that used stainless steel dowel bars (section M), had excellent performance. Furthermore, use of the stainless steel material would have a positive effect on long-term performance. The non-corrosive material is typically intended to provide extended protection to the dowel at the joint location; protection would also be provided in areas of mortar loss or deterioration.

Control sections 1 and 2 (diamond grinding only) were also evaluated during the visual distress survey in April 2010. After nine years in service, both of these sections were performing well, and little to no noticeable faulting was noted.

[1] The automated survey results presented in Section 4.2.2 showed that test section M had a comparatively high PDI value of 18.8 in 2007, while the visual survey showed no evidence of distress in this section's dowel bar slots. These contradictory results may be due to the problems associated with automatic surveying of DBR distresses, as discussed in Section 4.2.2. It is also possible that other distresses were noted in the mainline pavement slabs during the PDI survey that were not recorded during the visual survey.

Table 10. Distresses Noted in STH 13 Test Sections, April 2007

Test Section	A MnDOT 3U18	B MnDOT 3U18	C AHT 60%	D AHT 60%	E AHT 100%	F AHT 100%	G ThoRoc 60%	H ThoRoc 60%	I Tamms 100%	J Tamms 80%	K ThoRoc 60%	L Tamms 80%	M Tamms 80% (S.S. bars)	N AHT 100%	O ThoRoc 60%
Mortar Shrinkage/ Debonding	^	#	+						+	+		+			
Mortar Cracks	^	#	+												
Mortar Surface Deterioration					^	^		+						^	
Mortar Loss				+	+	^								^	
Slab Corner Cracking	X														
Joint Deterioration					^										

Table 11. Distresses Noted in STH 13 Test Sections, April 2010

Test Section	A MnDOT 3U18	B MnDOT 3U18	C AHT 60%	D AHT 60%	E AHT 100%	F AHT 100%	G ThoRoc 60%	H ThoRoc 60%	I Tamms 100%	J Tamms 80%	K ThoRoc 60%	L Tamms 80%	M Tamms 80% (S.S. bars)	N AHT 100%	O ThoRoc 60%
Mortar Shrinkage/ Debonding	^	#	+						+	+		+			
Mortar Cracks	^	#	+												
Mortar Surface Deterioration	^	+			^	X		+			#			X	
Mortar Loss				^	+	^								^	+
Slab Corner Cracking	X								#						
Joint Deterioration					^		+	#	+						

	+	#	^	X
None	Isolated	Minor	Significant	Severe

4.2.4 Load Transfer Efficiency

Tests for LTE were performed on joints in the STH 13 test sections in May 2003, two years after the DBR rehabilitation was completed. Joint deflection was measured with the WisDOT FWD equipment, and LTE was calculated as described in Section 4.1.3. The time of day and weather conditions when testing took place were not recorded.

Tests were conducted at three to five joints in 8 of the 15 test sections and one of the three control sections. Four FWD loadings were used (5, 9, 12, and 20 kips). An average LTE was calculated for each section; these values are reported in Table 12. The test sections with DBR had average LTE values in the 80 to 95 percent range, which indicates very good load transfer between slabs. Interestingly, the control section joints that did not receive DBR rehabilitation also had very good load transfer, with an average LTE of 91 percent. Test sections A and B, which displayed mortar distress, still had LTE values in a good range.

Table 12. Load Transfer Efficiency, STH 13, May 2003

Test Section	Position	Doweled?	Average LTE (%)
A	RWP	Y	81
B	RWP	Y	87
C	RWP	Y	86
E	LWP	Y	94
I	LWP	Y	92
J	RWP	Y	86
K	RWP	Y	87
M	RWP	Y	85
C1	LWP	N	91

4.3 USH 45

The DBR test sections on USH 45 were inspected visually in April 2010, after six years in service. Excellent performance was noted in the test sections that used Tamms Speed Crete (60 and 80 percent extension) for the slot fill material (eastbound PL and both westbound lanes). A few instances of joint deterioration were noted in these areas, but these distresses were typically present between the dowel slots (Figure 9).

More distresses were present in the eastbound driving lane, where dowel slots were filled with Master Builder's Set 45 mortar at 60 percent extension. In these test sections, debonding and spalling of the mortar were noted in approximately 10 percent of the dowel slots.

Figure 9. Joint deterioration between dowel slots on USH 45, 2010.

4.4 STH 21

A visual survey was conducted for the entire length of the STH 21 DBR project in April 2010, after six years in service. Good performance was noted overall for the DBR slots and mortar material. Some mortar loss (less than two percent) was noted in the slots, but this is not likely to affect the pavement performance. More extensive mortar loss was noted in slots in a few isolated areas. This minor to moderate surface loss typically appeared in several slots in one joint, and for several joints in a row. It is therefore possible that these distresses were due to underperforming mortar batches or batches that had not been properly mixed and were inconsistent at the end. This highlights the need for careful attention to the quality of mortar that is placed in the dowel slots.

An additional DBR project on STH 21 at the intersection of I-39 was surveyed in April 2010. This project was constructed at the same time as the I-39 DBR project evaluated in this study and used the same mortar material (ThoRoc 10-60C at 100 percent extension). The project had been in service for ten years at the time of evaluation. Mortar in the dowel slots was performing poorly. There was evidence of mortar loss and slight to moderate deterioration. Some slots with mortar loss had been filled with an epoxy-based concrete repair material. The repair material appeared to be performing well.

4.5 USH 18/151

In April 2010, a visual survey was conducted along the entire length of the concrete pavement between Dodgeville and Mount Horeb. Mixed results were noted. Some areas had minor to severe joint spalling and deterioration at the joints, and other areas were performing very well. The newest DBR areas (2006 to 2009 rehabilitation) were performing well overall. Some of the older DBR sections (1999 and 2000 rehabilitation) had mortar deterioration and joint spalling, but a few of the oldest sections had good performance. Several areas that did not have DBR were also performing well; the joints were in good condition, and the ride was comfortable (i.e. little slab faulting). Overall, it was difficult to pinpoint specific factors that resulted in either good or poor DBR performance on this roadway.

4.6 Performance Summary

A summary of the overall performance of each DBR project surveyed is provided in Table 13. Good or very good performance was typically noted up to 10 years in service. After 10 years in service, more problems with mortar and joint deterioration were noted, although many areas still had good performance and would provide additional years of service.

Table 13. DBR Project Performance Summary

Location	**Years in Service**	**Performance**
I-39	8	Poor mortar performance. Good ride (IRI). Excellent LTE.
STH 13	9	Many areas with mortar and joint deterioration, mortar loss. Good ride (IRI). Excellent LTE.
USH 45	6	Very good performance.
STH 21	6	Good performance overall. Several instances of minor to moderate mortar loss at 2 to 10 consecutive joints.
STH 21	10	Poor mortar performance.
USH 18/151	1 to 4	Good performance.
USH 18/151	10	Mixed performance. Some very good areas and some areas with mortar deterioration and joint spalling.

5. Cost Considerations

To determine if DBR is a cost-effective rehabilitation method, an analysis was performed to compare the costs associated with four rehabilitation strategies for a faulted non-doweled PCC pavement:

1. Diamond grind only
2. 3-in HMA overlay
3. DBR and diamond grind
4. DBR in DL only; diamond grind in both lanes

For each scenario, a cost per project mile (including both the DL and PL) was established using recent WisDOT project bid data (averages from April 2009 to April 2010). Only the associated pavement rehabilitation costs were included; additional costs such as pavement markings and traffic control were assumed to be approximately equal for all three scenarios. In addition, a service life was defined based on Wisconsin experience or on national data.

5.1 Diamond Grind Only

The average bid cost for diamond grinding a PCC pavement was $3.06 per square yard (SY). For a 14-ft wide DL and a 12-ft wide PL, this translates to a cost of $46,700 per project mile.

Because diamond grinding does not address the structural causes of PCC slab faulting, faulted joints are almost guaranteed to occur again after diamond grinding. The service life of a diamond-ground, non-doweled pavement has been estimated to be 8 to 10 years. [*4*] The I-39 performance data for control section 2 (diamond-grinding only) showed an IRI value of 1.53 m/km in the driving lane after eight years in service. The average IRI for the driving lanes with DBR was 0.93 m/km. Therefore, the difference in these values (0.60 m/km) is likely due to joint faulting. Assuming 15-ft joint spacing, the average joint faulting in control section 2 after eight years in service was 0.11 in. Rehabilitation is recommended when average joint faulting reaches 0.15 in. [*4*] For purposes of this discussion, the service life of a diamond-ground non-doweled PCC pavement was estimated to be 10 years.

5.2 3-inch HMA Overlay

Several materials are bid separately for WisDOT HMA paving projects: HMA mixture, asphalt cement binder, and tack coat. For this analysis an E-3 HMA mixture[2] and PG 58-28 binder were used, as these are common HMA paving materials for highway overlay applications. In addition to the 14-ft DL and 12-ft PL, the cost of 3-ft shoulders on each side was also included, as the raised pavement elevation would require matching shoulder elevations.

The average bid costs for the HMA mixture, binder, and tack coat were $41.62 per ton, $276.30 per ton, and $2.39 per gallon, respectively. In addition to the original construction costs, one crack sealing operation for the HMA pavement was assumed to be required during the initial service life. The

[2] The WisDOT E-3 HMA mixture is designed for 3 million ESALs during the pavement's design life. [*5*]

Department's Pavement Maintenance Program defines a crack sealing cost of $4,900 per lane mile, or $9,800 per project mile in this scenario. With this information, the cost for a 3-in HMA overlay was calculated to be $186,900 per project mile.

As with diamond grinding, an HMA overlay does not address the structural issues behind slab faulting, and excessive roughness in an HMA pavement over non-doweled PCC is likely due to slab faulting. The IRI measurements for control section 1 (3-in HMA overlay) in the I-39 test project were typically about the same as those for the diamond-ground control section. Therefore, these types of pavements would likely need rehabilitation at the same time. A service life of 10 years was also estimated for a 3-in HMA overlay.

5.3 DBR and Diamond Grind

The average DBR bid cost was $31.30 per dowel bar installed. Assuming 15-ft joint spacing and installation of 12 dowels per joint, this results in a cost of $132,200 per project mile. Adding the $46,700 per project mile cost of the diamond grinding operation results in a total cost of $178,900 per project mile for DBR.

The estimated service life of DBR construction varies. Literature has indicated approximately 20 years of service for a pavement initially in good condition (other than joint faulting) to 10 years for a pavement in poorer initial condition. Many of Wisconsin's non-doweled PCC pavements remain in good condition with the exception of joint faulting. In some of this study’s DBR test section pavements, however, distresses in the mortar and at joints have been noted after ten years in service or earlier. Therefore, a service life of 15 years was estimated for DBR construction in this discussion.

5.4 DBR in Driving Lane Only

Performing DBR in the DL only reduces the number of dowels per joint to six. The cost of the DBR operation is therefore $66,100 per project mile. Adding the $46,700 per project mile cost of diamond grinding in both lanes results in a total cost of $112,800 per project mile for this scenario.

As discussed above, the service life of the DBR in the DL was 15 years. The diamond grind in the PL would have an estimated service life of 10 years, at which point both lanes would require an additional diamond grinding operation.

5.5 Discussion

The costs and service lives determined in the previous section are summarized in Table 14. A 3-in HMA overlay requires the greatest initial investment, followed closely by the DBR and diamond grind option in both lanes. Additionally, because the service life for the HMA overlay is 10 years while the DBR operation provides 15 years of service, DBR is more cost effective than the HMA overlay.

Although the diamond grind rehabilitation option provides only two-thirds the service life of DBR, the initial cost of diamond grinding is four times less than for DBR. However, there are additional factors to consider if repeated diamond grinding operations are used to maintain a smooth pavement surface:

1. Joint faulting slowly recurs after diamond grinding, resulting in an uncomfortable ride and increased vehicle wear for several years before the next diamond grind operation is performed.
2. Additional operations and highway closures required by repeated diamond grinding result in increased traffic control, pavement marking, and user delay costs.
3. Repeated diamond grinding reduces the structural thickness of the PCC pavement.
4. The final rehabilitation method for nearly all PCC pavements is an HMA overlay. Overlaying a diamond-ground non-doweled pavement will eventually result in reflective cracking and a poor ride, because the slabs will continue to fault under the HMA overlay.

These issues are similar for the PL in the scenario where only the DL received DBR rehabilitation. However, recurring joint faulting could be less of a problem in the PL because of less frequent truck loadings. (This hypothesis was not confirmed in the performance data discussed in Section 4.1.2.) If faulting was significantly worse in the DL than in the PL at the time that rehabilitation is necessary, it is possible that future joint faulting in the PL would also be limited, and DBR in the DL only would be a cost-effective option.

None of the concerns described above is a problem with the DBR operation in both lanes. If slab faulting is a problem in both lanes at the time of rehabilitation, it is more prudent and likely more cost-effective in the long term to use the DBR rehabilitation strategy rather than repeated diamond grinding operations or DBR in the DL only. DBR addresses the root structural cause of slab faulting and provides excellent LTE, as discussed in Sections 4.1.3 and 4.2.4. When the mortar fill material is properly mixed, placed, and cured, DBR results in smoother, longer-lasting pavement.

Table 14. Rehabilitation Option Initial Cost and Service Life

Rehabilitation Method	Initial Cost per Project Mile	Service Life (Years)
Diamond grind	$46,700	10
3-in HMA overlay	$186,900	10
DBR and diamond grind	$178,900	15
DBR in DL only	$112,800	15 (DL) 10 (PL)

6. Current Practices

As mentioned previously, a moratorium was issued for DBR rehabilitation projects after the early mortar deterioration was noted on the I-39 project. This moratorium was lifted in 2004. From that time forward, all DBR projects have been constructed under a performance warranty that guarantees the condition of the dowel slots for three years. If the dowel slots and surrounding concrete remain in good condition for three years, it is anticipated that the pavement will provide satisfactory performance for the remaining expected service life.

The distresses covered under the DBR performance warranty are shown in Table 15. If any distress is noted in the DBR project and exceeds the threshold level, the contractor must perform the remedial action described in the final column of Table 15. In addition, the contractor is responsible for repairs of slots where the foam core board used to maintain the joint shifts during placement of the mortar, such as in Figure 10. The department surveys the DBR projects at least twice during the three-year warranty period to determine if any distresses are present.

Use of the warranty provision for DBR projects helps ensure quality workmanship during construction. This type of guarantee is valuable, as use of quality materials and sound construction of dowel slots are critical to long-term performance of DBR projects. This correlation between superior workmanship and long-term performance has also been noted for DBR rehabilitation projects in Washington State. [*3*]

Table 15. DBR Warranty Distress Levels and Remedial Actions

Distress Type	**Threshold Level**	**Remedial Action**
Distressed joints within the DBR slot	• Spalling of 1 inch or greater on more than 10% of joints per 0.1 mile segment Or • Spalling of 2 inches or greater on 1% of the joints per 0.1 mile segment	Remove and replace retrofit dowel bar
Cracking in existing concrete pavement between slots or across slab to pavement edge (corner crack)	Greater than 1% of joints per lane mile	Standard full-depth concrete repair of pavement
Loss of surface and concrete patch material within dowel bar slot	(a) Loss of material greater than ½-inch but less than 1 inch on more than 1% of joints per lane mile (b) Loss of material of 1 inch or greater on more than 1% of joints per lane mile	(a) Surface treatment as approved by the engineer (b) Remove and replace retrofit dowel bar
Debonding of patch material from existing concrete on any slot surface	Debonding on any surface on more than 1% of joints per lane mile	Remove and replace retrofit dowel bar
Breakup or dislodgement of patch material within slot	One or more cracks in greater than 1% of the joints per lane mile	Remove and replace retrofit dowel bar

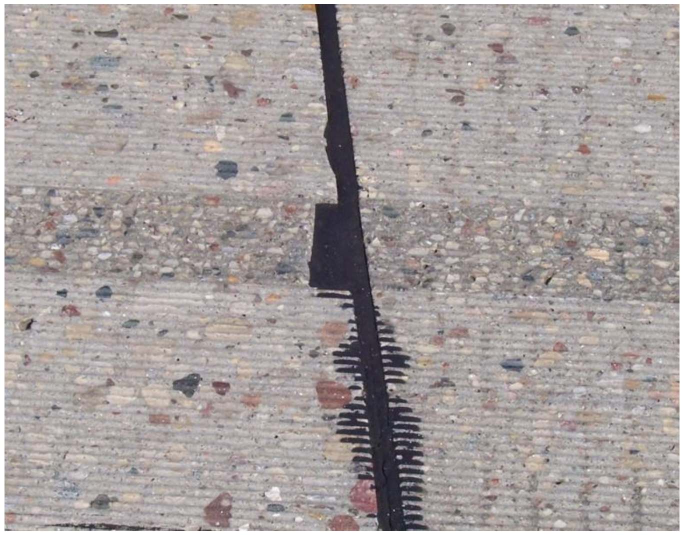

Figure 10. Misalignment of foam core board on STH 21, 2007.

7. Summary and Conclusions

DBR was performed on several sets of test sections in this study. Test sections on I-39 and STH 13 were surveyed for PDI and IRI between 2001 and 2007, and tests for LTE between adjacent pavement slabs were also performed. Visual pavement surveys were performed in 2010 for DBR pavement on STH 13, USH 45, STH 21, and USH 18/151.

Dowel slots in the I-39 test sections and in other areas along the overall construction project had mortar deterioration after only two years in service. This problem was addressed by replacing the deteriorated mortar material and eventually overlaying the project with HMA. Results of the PDI survey showed little to no distress in the diamond-ground control section. The control section that had been overlaid with HMA had a higher PDI, which was attributed to reflective cracking caused by faulting at the underlying joints.

Automated IRI measurements showed that the test sections were smoother after eight years in service than the control sections. The two control sections, which received rehabilitation treatments of an HMA overlay (C1) and diamond grinding only (C2), experienced a recurrence of joint faulting. It was difficult to determine whether performance was consistently acceptable in test sections that received DBR in the DL only.

Tests for LTE on the I-39 test sections showed that the DBR rehabilitation strategy greatly improved load transfer between adjacent pavement slabs. Prior to DBR, the average LTE ranged from 25 to 50 percent; after DBR, the range increased to 70 to 95 percent.

The STH 13 test sections were constructed to investigate different mortar materials for filling of dowel bar slots. Materials that showed excellent performance (no debonding, cracking, or material loss) included ThoRoc mortar at 60 percent extension and Tamms Speed Crete at 80 percent extension. Mortar mixtures that were extended by 100 percent showed distresses, particularly mortar loss and debonding.

LTE testing on the STH 13 test sections showed good load transfer in all test sections; the range was 80 to 95 percent. One control section that did not have DBR also showed good load transfer (91 percent). In this case, aggregate interlock forces were strong enough to provide load transfer between adjacent slabs.

Mixed DBR performance was noted during visual surveys of areas on USH 45, STH 21, and USH 18/151. Some sections had very good performance after six to ten years in service. Distresses such as mortar deterioration and spalling at the joints were noted in other areas, however. If these distresses were a result of poor materials or faulty construction, it is possible to reduce their occurrence in the future by enforcing superior construction practices.

It was determined that DBR is a cost-effective rehabilitation strategy; this technique addresses the root cause of slab faulting and should maintain load transfer for approximately 15 years. Other rehabilitation methods considered included diamond grinding only and HMA overlay. These techniques had lower initial costs but would need to be repeated after approximately 10 years in service. Performing DBR in the DL only is a lower-cost option for pavements that do not have severe slab faulting in the PL.

As was apparent in both the I-39 and STH 13 test sections, use of quality materials and proper attention to detail during construction is critical for long-term performance of DBR projects. To increase the probability of good performance, WisDOT currently applies a warranty provision to all DBR construction. It is the contractor's responsibility to remedy distresses that occur in and around dowel slots for three years after construction. If good performance is guaranteed for three years, it is anticipated that the DBR will perform well in the long-term.

8. Recommendations

When properly constructed, DBR has been shown to effectively maintain load transfer at PCC pavement joints and reduce slab faulting and associated distresses. It is a cost-effective rehabilitation strategy, as it addresses the structural cause of slab faulting and improves load transfer between adjacent slabs. This rehabilitation strategy can extend the life of a non-doweled PCC pavement by approximately 15 years. DBR is therefore considered a viable rehabilitation method for faulted non-doweled PCC pavement that is otherwise in good condition.

If a correction for slab faulting is required in both the driving and passing lanes, it is recommended that DBR be performed in both lanes, followed by diamond grinding in both lanes. If slab faulting needs to be addressed in the driving lane but not in the passing lane, DBR in the driving lane only followed by diamond grinding in both lanes is a valid option.

It is recommended that WisDOT continue its policy to place a performance warranty on all DBR construction. This guarantee reduces the possibility that early deterioration of mortar in the dowel slots will occur as it did on the I-39 test sections in this study.

Although all new PCC pavements constructed in Wisconsin use dowel bars to maintain load transfer between slabs, there are still a number of remaining non-doweled pavements that are DBR candidates; several hundred lane-miles of non-doweled PCC pavement are still in service. In addition, the DBR technique can be used to prevent faulting at transverse cracks that develop in PCC slabs.

9. References

1. Wilson, J., and Toepel, A. “Report on Early Distress (RED): Retrofit Dowel Bars on I-39.” Report no. RED-05-01. Wisconsin Department of Transportation. Jan. 2002.

2. Bischoff, D., and Toepel, A. “Dowel Bar Retrofit: STH 13 Construction and One-Year Performance Report.” Report no. WI-07-02. Wisconsin Department of Transportation. Nov. 2002.

3. Pierce, L. M., Muench, S. T., and Mahoney, J. P. "Long-Term Performance of Dowel Bar Retrofit in Washington State." Presentation at the 89th Annual Meeting of the Transportation Research Board. Jan. 2010.

4. Rao, S., Yu, H., Khazanovich, L., and Darter, M. "Longevity of Diamond-Ground Concrete Pavements." Presentation at the 78th Annual Meeting of the Transportation Research Board. Jan. 1999.

5. Wisconsin Department of Transportation Standard Specifications for Highway and Structure Construction. Part 4, Section 460, "Hot Mix Asphalt Pavement." 2010 Edition.

Appendix A Test Section Layouts

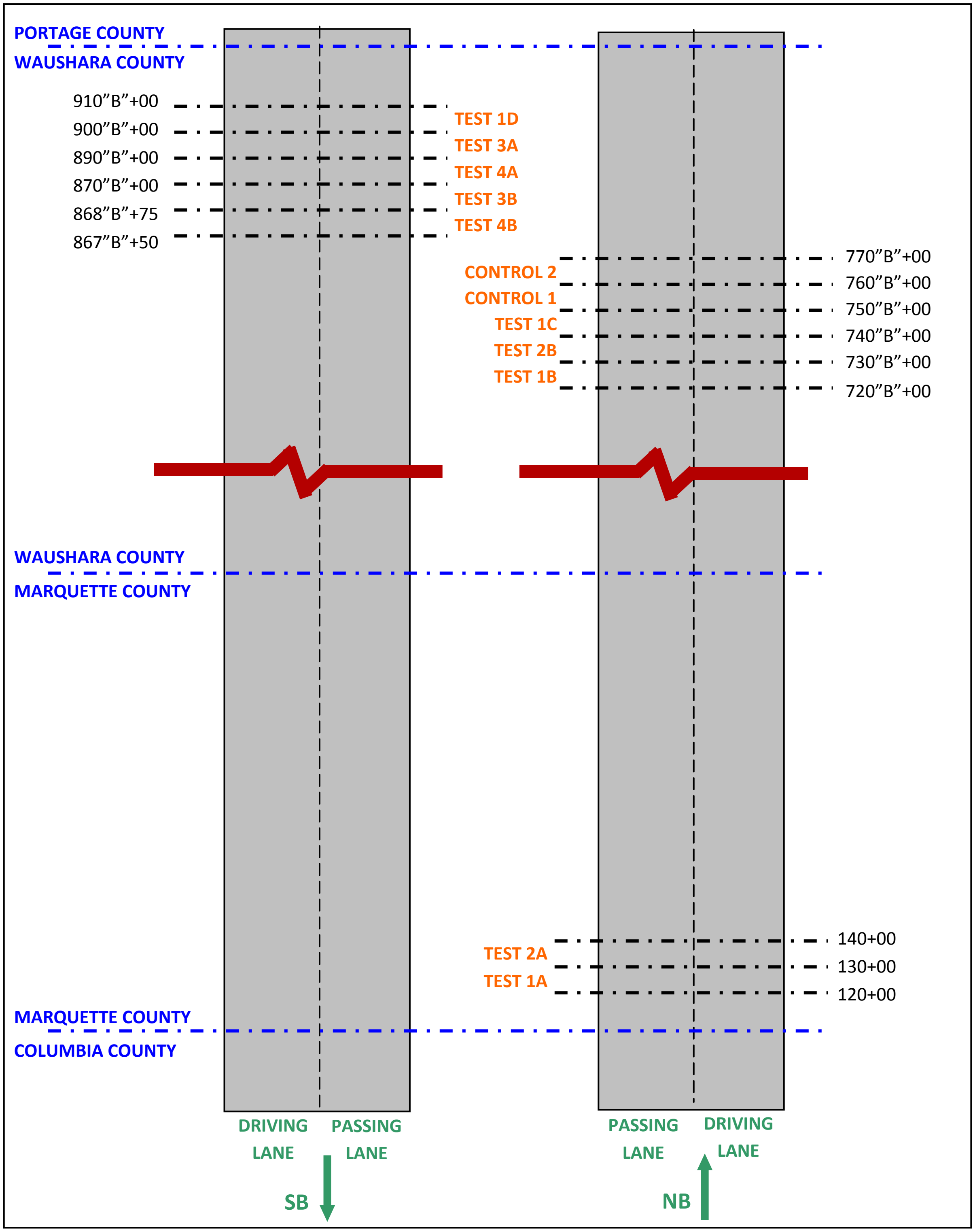

Figure A-1. I-39 test section layout. Note: not to scale.

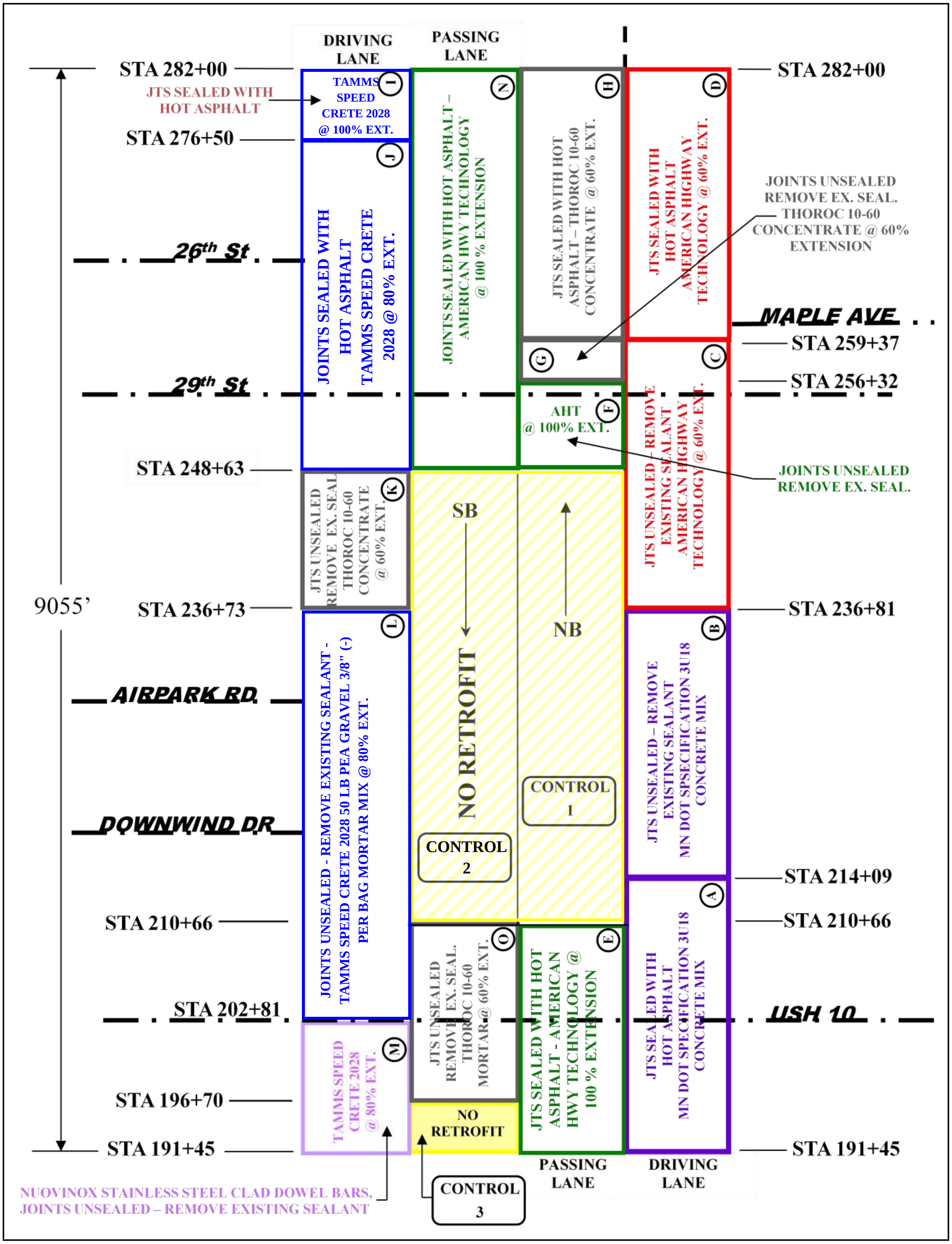

Figure A-2. STH 13 test section layout.

Appendix B I-39 Performance Data

Table B-1. PDI values for I-39 test sections.

Test Section	Lane	2001	2002	2003	2004	2005	2006	2007
1A	DL	0.0	0.0	0.0	*	0.0	0.0	3.4
1A	PL	0.0	0.0	3.4	*	3.4	3.4	3.4
1B	DL	0.0	0.0	3.8	6.2	6.2	6.2	6.2
1B	PL	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1C	DL	0.0	0.0	0.0	3.4	3.4	0.0	0.0
1C	PL	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1D	DL	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1D	PL	*	*	*	*	*	*	*
2A	DL	0.0	0.0	0.0	*	0.0	0.0	0.0
2A	PL	0.0	0.0	0.0	*	0.0	0.0	0.0
2B	DL	0.0	0.0	0.0	0.0	0.0	3.4	6.2
2B	PL	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3A	DL	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3A	PL	*	*	*	*	*	*	*
3B	DL	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3B	PL	*	*	*	*	*	*	*
4A	DL	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4A	PL	*	*	*	*	*	*	*
4B	DL	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4B	PL	*	*	*	*	*	*	*
Control 1	DL	16.9	16.9	22.7	28.0	28.0	28.0	28.0
Control 1	PL	16.9	16.9	16.9	16.9	16.9	16.9	16.9
Control 2	DL	0.0	0.0	0.0	3.4	6.1	6.1	6.1
Control 2	PL	0.0	0.0	0.0	0.0	0.0	0.0	0.0

*Survey not conducted in this lane.

Table B-2. IRI values for I-39 test sections.

Test Section	Lane	2001	2002	2003	2004	2005	2006	2007
1A	DL	0.68	0.77	1.33	*	1.07	1.20	0.98
1A	PL	1.20	0.60	1.55	*	1.10	0.92	0.98
1B	DL	0.62	0.44	0.96	0.85	0.76	0.96	0.87
1B	PL	0.74	0.60	1.78	0.98	0.69	0.90	0.88
1C	DL	0.71	0.46	1.04	1.14	0.96	1.06	0.79
1C	PL	0.90	0.55	1.91	0.88	0.84	0.96	1.34
1D	DL	0.84	0.88	1.36	1.25	0.88	1.20	0.95
1D	PL	*	*	*	*	*	*	*
2A	DL	1.18	0.79	1.18	*	0.98	1.15	0.80
2A	PL	0.88	0.63	1.52	*	1.01	0.90	0.74
2B	DL	0.65	0.46	0.98	0.96	0.90	1.12	0.87
2B	PL	0.82	0.49	1.88	0.95	0.82	0.84	1.44
3A	DL	0.93	1.01	1.25	1.14	0.85	1.10	0.98
3A	PL	*	*	*	*	*	*	*
3B	DL	0.68	0.87	1.23	1.09	0.58	0.76	1.07
3B	PL	*	*	*	*	*	*	*
4A	DL	0.80	0.88	1.29	0.88	0.66	0.85	0.98
4A	PL	*	*	*	*	*	*	*
4B	DL	0.71	0.87	1.01	1.07	0.63	0.79	0.96
4B	PL	*	*	*	*	*	*	*
Control 1	DL	0.55	0.66	1.50	1.67	1.47	1.44	1.58
Control 1	PL	0.51	0.54	1.82	0.99	1.20	1.18	1.23
Control 2	DL	1.07	0.69	1.34	1.61	1.29	1.45	1.53
Control 2	PL	0.85	0.57	1.48	0.96	0.80	0.88	1.12

*Survey not conducted in this lane.