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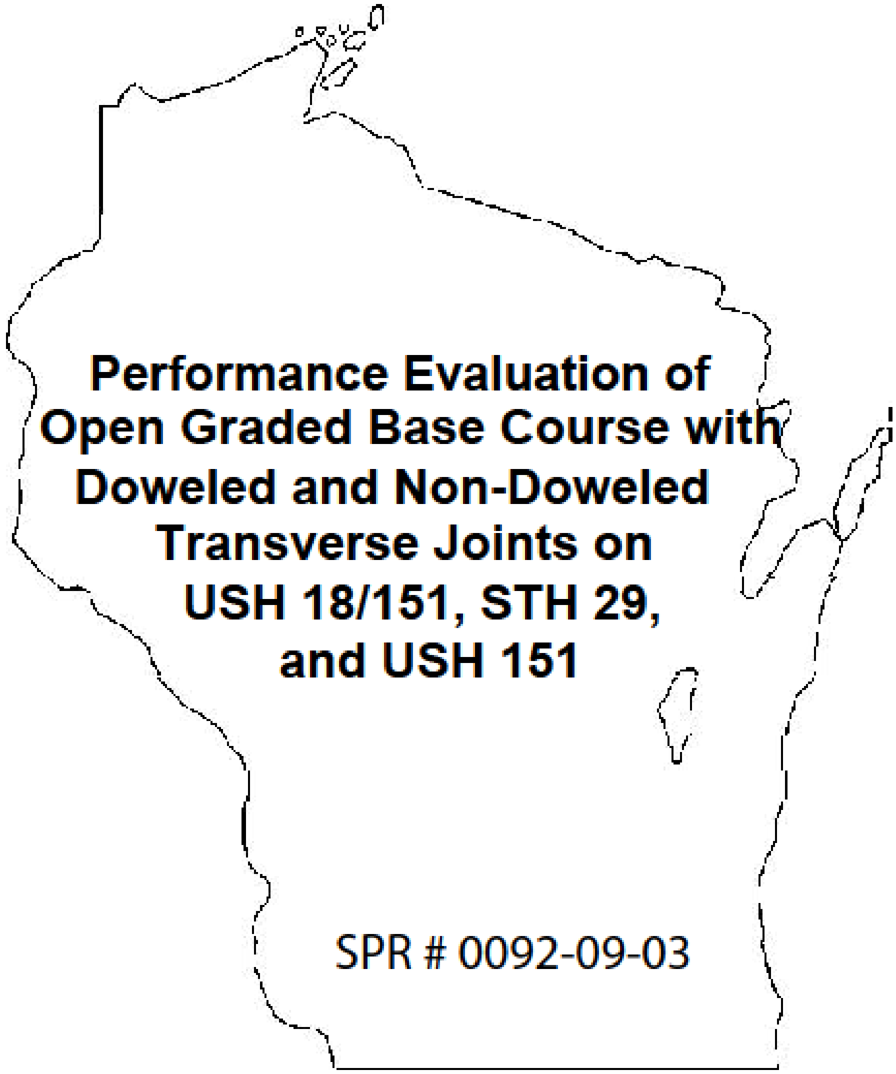

Performance Evaluation of Open Graded Base Course with Doweled and Non-Doweled Transverse Joints on USH 18/151, STH 29, and USH 151

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16. Abstract

The objectives of this study were to investigate the performance of 20-year old doweled/non-doweled and dense-graded/permeable base test sections on three concrete pavement segments in Wisconsin: USH 18/151 in Iowa and Dane counties, STH 29 in Brown County, and USH 151 in Columbia and Dane Counties. Five pavement bases were placed including: dense graded, asphalt-stabilized permeable, cement-stabilized permeable, and untreated permeable having two gradation sizes.

USH 18/151 test sections had similar performance (PDI) for doweled unsealed pavement on dense and permeable base. Distresses common to all segments included slight to moderate distressed joints/cracks and slight transverse faulting. Asphalt-stabilized permeable base had no slab breakup or surface distresses, however it measured a greater severity of distressed joints and cracks. Non-doweled sections having asphalt-stabilized permeable base and Transverse Inter Channel drains had better performance and ride than the other non-doweled sections. IRI was generally higher on non-doweled pavements, but many doweled sections had an equal roughness to non-doweled sections. Sealed non-doweled joints produced a better performing pavement, however, sealant did not appear to improve ride.

STH 29 unsealed sections performed better than the median PDI for the sealed sections. The sealed doweled pavement did perform a little better than the non-doweled section, but the opposite occurred on the non-doweled sections. Sealed doweled joints had a smoother ride than the other combinations.

USH 151 test sections found the finer-graded New Jersey permeable base had the smoothest ride when compared to other permeable sections. Asphalt-stabilized permeable base had the roughest ride, and unstabilized and cement-stabilized permeable bases had intermediate values.

The average hydraulic conductivity for the unstabilized permeable base was 17,481 feet per day and there appears little variation due to doweling or joint sealant. Deflection load transfer results indicate expected high average values for the doweled sections and fair to poor values for the non-doweled sections. Slab support ratios indicate variable results based on base type and joint reinforcement/sealant.

Life-cycle cost analysis found dense-graded base was the least cost among all base alternatives, with a total estimated present-worth life-cycle cost of $665,133 per roadway mile. Untreated and asphalt-stabilized permeable bases were more expensive by 13% and 27%, respectively. Other factors in selecting dense-graded base over permeable base include project drainage conditions set forth in the FDM guidelines an anticipated increase in pavement surface roughness.

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EXECUTIVE SUMMARY

The objectives of this study are to investigate the performance of doweled/non-doweled and open-graded/dense-graded base test sections on three concrete pavement segments in Wisconsin. A field evaluation was conducted from 20-year old pavement on USH 18/151 in Iowa and Dane Counties (17 test sections), STH 29 in Brown County (4 test sections), and USH 151 in Columbia and Dane Counties (4 test sections). This data allowed a comparison of unique features of each section to determine effects between subgrade support, drainability, load transfer, joint sealant, and overall performance.

The USH 18/151 test sections were constructed in 1988 with 9-inch thick PCC pavement and 7 unique design factors across Iowa and Dane County. Test sections in Iowa County were non-doweled, while those in Dane County were doweled. Both sealed and unsealed transverse joints were constructed in each county. There were 5 unique pavement bases, including: asphalt stabilized permeable base (ASOG), cement-stabilized permeable base (CSOG), unstabilized permeable base (OGBC), dense graded, and Transverse Inter Channel (TIC) drains on dense-graded base. Except for the TIC drain system, the remaining four base types were constructed in both the non-doweled Iowa County sections and doweled Dane County sections.

Test sections on STH 29 in Brown County and USH 151 in Columbia County were a more simplified experimental design than USH 18/151. STH 29 was constructed in 1988 with 10-inch thick PCC pavement over a 4-inch upper permeable aggregate base and a 4-inch lower dense aggregate subbase. Joints were both sealed and unsealed in two non-doweled sections and two doweled sections.

Constructed in 1991, the all-doweled USH 151 Columbia County project has 10-inch PCC pavement over 5 unique bases: ASOG, CSOG, unstabilized OGBC, dense graded, and unstabilized finer-graded New Jersey OGBC with 50% passing the #4 sieve. Asphalt concrete sections were also included in this project. Drainage pipe on STH 29 and USH 151 were 6-inch diameter, unlike USH 18/151 with 4-inch pipe diameter.

Data were collected for the Pavement Distress Index (PDI), International Roughness Index (IRI), Falling Weight Deflectometer (FWD), and water drainage to evaluate pavement performance, support conditions, and water permeability through the base course. Both automated and manual pavement condition data surveys were conducted for each test section. First, semi-automated electronic survey were collected for transverse faulting and ride quality with IRI measurements in both wheel paths. Pavement condition was manually measured for traditional PCC pavement distresses, including slab breakup, distressed joints and cracks, joint crack filling, patching, surface distress, longitudinal joint distress and distortion, and transverse faulting.

The doweled sections of USH 18/151 had similar performance (PDI) for doweled unsealed pavement on both dense and permeable base. Distresses common to all segments included slight to moderate distressed joints/cracks and slight transverse faulting. ASOG had no slab breakup or surface distresses, however it measured a greater severity of distressed joints and cracks. The dense-graded base section had the roughest ride when compared to all open-graded doweled sections. There was little difference in ride among the open-graded sections. In summary, doweled pavement on asphalt-stabilized open graded bases had the lowest measured composite distresses, while the open-graded bases had a lower surface roughness.

For non-doweled sections on USH 18/151, the CSOG, ASOG, and TIC drains had the least amount of distress. DGBC and untreated OGBC had the highest composite measure of pavement distress. ASOG base and TIC drains had the smoothest ride, while untreated OGBC and CSOG had the rougher surface smoothness. Therefore, non-doweled sections having ASOG base and TIC drains had better performance and ride than the other non-doweled sections.

USH 151 had doweled 10-thick PCC, unsealed skewed transverse joints, paved over a 4-inch top permeable base (untreated with two gradations, cement-stabilized, and asphalt-stabilized) and 4-inch lower dense base. All permeable base types had nearly the same performance among the different bases with slight distressed joints/cracks. Minor differences were found with untreated OGBC with 10% of slab area having slab breakup and surface distresses, and ASOG having slight transverse faulting. The finer New Jersey open-graded base had the smoothest ride when compared to other open-graded sections. ASOG base had the roughest ride, and unstabilized OGBC and CSOG bases had intermediate values. In summary, the much finer-graded New Jersey base had less composite distresses and a smoother ride.

Pooled data from the three projects found that non-doweled pavement generally has a higher distress level than doweled; however, when two non-doweled outliers are removed, the difference is less pronounced. The extent of transverse faulting was equal among all test sections, however, the severity was higher for non-doweled joints with about half of those sections rated a level 2 (¼ to ½ inch). All doweled sections were either at or less than 0.02 inches. IRI was generally higher on non-doweled pavements, but many doweled sections had an equal roughness to non-doweled sections.

USH 18/151 sealed non-doweled joints produced a better performing pavement than unsealed joints; however, sealant did not appear to have a consistent effect on ride. On two doweled dense-graded sections, sealant slightly outperformed the unsealed section, with minor patching the prominent distress for the unsealed section. Both sections had identical extent and severity levels for slab breakup, distressed joints/cracks, surface distress, longitudinal distress, and transverse faulting.

STH 29 unsealed sections for doweled/non-doweled joints performed better than the median PDI for the sealed sections. The sealed doweled pavement did perform slightly better than the non-doweled section, but the opposite occurred on the non-doweled sections. Sealed doweled joints had a smoother ride than the other combinations. Sealed/non-doweled joints produced the roughest ride, and as expected, non-doweled joints, whether sealed or unsealed, had the highest IRI values.

The average hydraulic conductivity for the unstabilized OGBC was 17,481 feet per day (fpd), exceeding the desired minimum rate of 1,000 fpd. The average hydraulic conductivity for the cement-stabilized permeable base CSOG was 15,129 fpd and there was a substantial variation due to joint sealant, with the sealed section having a hydraulic conductivity of 21,212 fpd and the unsealed sections averaging 12,087 fpd. The average hydraulic conductivity for the ASOG was 8,471 fpd which was significantly lower than the untreated OGBC and CSOG sections. There appeared to be a slight variation due to doweling with the doweled section having a hydraulic conductivity of 5,920 fpd and the non-doweled sections averaging 9,747 fpd.

The results provided for STH 29 Brown County indicate adequate drainage capacity in all sections. The data disclosed a significant variation due to doweling but little variation due to joint sealant. The average hydraulic conductivity for the unstabilized permeable base (OGPB) sections without dowels is 2,817 fpd and 13,637 fpd for the doweled test sections. Results for USH 151 Columbia-Dane Counties indicate adequate drainage capacity in only the CSOG base section, with a calculated hydraulic conductivity of 10,697 fpd. The base layers in the remaining three test sections would not accept water, indicating a complete blockage of the layer. The reason for this condition is unknown.

The deflection load transfer results indicate expected high average values for the doweled sections and fair to poor values for the non-doweled sections. For USH 18/151, the overall average load transfer values for the doweled and non-doweled sections were 94.8% and 40.9%, respectively. For the non-doweled sections, the overall average load transfer values for the sealed and unsealed sections were 45.1% and 38.5%, respectively. For the doweled sections, the overall average load transfer values for the sealed and unsealed sections were 96.0% and 94.7%, respectively. For STH 29, the overall average load transfer values for the doweled and non-doweled sections were 93.0% and 17.9%, respectively. Little variation was noted for the sealed and unsealed sections. For USH 151, the overall average load transfer value for the doweled sections was 98.3%.

The slab support ratios indicate variable results based on base type, joint reinforcement and joint sealant. For USH 18/151 Iowa-Dane Counties, all corner support ratios suggest full support is maintained. The edge support ratios generally indicate full support is maintained with the exception of three doweled and unsealed sections; namely sections 10a (SSRe=0.58), 13 (SSRe=0.54) and 14 (SSRe=0.67). These reduced values (< 0.75) suggest support problems due to densification of the base layers which is not normally expected for doweled sections. For the STH 29 sections, reduced edge support is noted for non-doweled section 2 (SSRe=0.73) and doweled section 3 (SSRe=0.69) and reduced corner support is noted for doweled section 3 (SSRc=0.63). While these values are near the trigger value of 0.75, indicating only minor loss of support, it is interesting to note that these are the sealed sections. The results from USH 151 Columbia-Dane Counties indicates support problems under all edges and corners, with SSR values ranging from a low of 0.16 to a high of 0.66.

A life-cycle cost analysis was performed to quantify costs of comparable sections for the various base types. The analysis began by identifying the stage or time in pavement life when rehabilitation activities would occur using performance models, then estimating a cost for each rehabilitation. The analysis found that dense-graded base was the least overall cost among all base alternatives, with a total estimated 65-year present-worth life-cycle cost of $665,133 per roadway mile. Open-graded permeable bases were more expensive, with the estimated cost of untreated open-graded base at $748,843 and asphalt-stabilized open-graded base at $844,810. These costs translate to increases of 13% for untreated open-grade base and 27% for asphalt-stabilized open-graded base. When only cost is considered, the dense-graded base is the recommended choice. Rehabilitation cost for dense-graded base was more than the permeable base, but first construction cost was the primary determinant. Another factor in choosing dense-graded base over open-graded base is the drainage conditions on the project as set forth in the FDM guidelines. Also, ride performance is another factor, where the dense-graded base sections had a good performing IRI ranging from 119 to 135 inches per mile, and permeable sections having an IRI of approximately 100 inches per mile.

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CHAPTER 1 INTRODUCTION

1.1 Background

Until the late 1980s, Portland cement concrete (PCC) pavement in Wisconsin was constructed as either jointed plain concrete pavement (JPCP) or continuously reinforced concrete pavement (CRCP). Use of CRCP was discontinued because of its high initial cost. Use of JPCP was questioned because of severe transverse joint faulting that occurred at many locations. It was proposed that using dowel bars to provide load transfer at joints and/or eliminating free water and erodible material beneath the slabs would alleviate the faulting problem. All PCC pavements since 1987 have been constructed as JPCP with doweled joints, and many utilize open-graded base course (OGBC) to provide a drained pavement structure. However, it has not been proven whether dowels, OGBC, or a combination of both provide the best protection against joint faulting and other pavement distress. In 1988, 17 test sections were constructed on USH 18/151 in Iowa and Dane Counties to study the effects of dense and open graded base courses (stabilized and non-stabilized), several drain systems, and doweled and non-doweled transverse joints. A performance report was written after the pavement had been in service for 10 years (Rutkowski et al. 1998). The major conclusions of this report were that dowels and asphalt-stabilized OGBC provided the greatest protection against joint faulting, but use of dowels and asphalt-stabilized OGBC in combination did not provide significantly better performance than using either of these measures separately.

Additionally, test sections to evaluate doweled/non-doweled performance were constructed on STH 29 in Brown County. Constructed in 1988, this pavement cross-section consists of a 10-inch JPCP over a 4-inch permeable aggregate base and a 4-inch aggregate subbase. The joints are non-doweled in two test sections and doweled in the other two sections. For all four sections, the joints are skewed and variably spaced in a repeating 12-13-19-18 ft pattern. The STH 29 project is part of an original Wisconsin experimental section that investigated a number of different design features, including joint sealant and dowel bars. In this study, two sections contained pre-formed sealant and two sections are unsealed.

Other test sections were constructed around the state. In 1991, the doweled USH 151 Columbia County project had 5 test sections, each with a unique base: asphalt stabilized permeable, cement stabilized permeable, unstabilized permeable, dense graded, and New Jersey permeable. A dense-graded section was constructed at the STH 73 interchange, but did not have equivalent traffic loading and structural section to the mainline test sections.

After nearly 20 years of service, performance differences among these test sections may now be apparent. Designing a field data collection plan and analyzing the data will allow more definite conclusions to be drawn.

1.2 Problem Statement

This study evaluated the 20-year performance characteristics of 3 concrete pavement test segments in Wisconsin, including 17 test sections constructed on USH 18/151 in Dane and Iowa Counties; 4 test sections constructed on STH 29 in Brown County; and 4 test sections on USH 151 in Columbia County. Performance results of test sections constructed with multiple combinations of doweled and non-doweled joints; cement, lean concrete, asphalt, and non-stabilized OGBC; pipe/aggregate longitudinal, interchannel transverse, and wrapped trench/pipe edge drains; and sealed and unsealed transverse joints. At this time, it is unclear what factors, or combination of factors, influence actual performance, as measured by the PDI and IRI.

1.3 Objective

The objective of this study is to investigate the performance of test sections on three concrete pavement segments in Wisconsin: (1) 17 test sections along USH 18/151 in Iowa and Dane counties, (2) 4 test sections along STH 29 in Brown County, and (3) 4 test sections along USH 151 in Columbia County. The following analytical tools are used, including:

(a) WisDOT Pavement Surface Distress Survey Manual and PDI;
(b) International Roughness index (IRI);
(c) Falling Weight Deflectometer (FWD) testing to evaluate support conditions;
(d) Permeability testing to measure water flow through the permeable base course; and
(e) Data analysis and modeling.

Initially, the scope of the project was limited to USH 18/151 in Iowa and Dane Counties. WHRP amended the initial scope of research to include more testing and evaluation of PCC test pavements. In the report by Rutkowski et al. (1988), there were several other research segments constructed from 1987 to 1991 to evaluate the performance of both PCC and AC pavements. These segments were reviewed to identify candidate test sections for evaluation in this study. Since the principal objective of the initial USH 18/151 research was to evaluate dowel/non-dowel and base performance, there were 3 candidate segments from the 1998 report that were best suited for this study, including: STH 29, Brown County; USH 14, Dane County; and USH 151, Columbia County.

With the approval of additional funds, the study was expanded to allow the testing of STH 29 in Brown County. USH 14 was overlayed with hot-mix asphalt in 2007, precluding a detailed data collection and analysis. The WisDOT Kuab 2-m FWD was made available for this study soon after the addition of STH 29 project, allowing cost savings to be applied to a third project, USH 151 in Columbia County.

1.4 Benefits

The potential benefits of this study include:

- Enhancing WisDOT PCC pavement design that result in pavements providing a high level of performance at the lowest cost;
- Augmenting results from previous studies of dowel bars and drained pavement structures; and
- Supplementing technological developments and the knowledge base on this topic.

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

A comprehensive literature review was conducted to identify factors affecting PCC performance in doweled and non-doweled pavements having varying base conditions. The literature review was conducted using the Transportation Research Information Services (TRIS), general web-based search, and published documents related specifically to these test sections.

Several recent literature sources were reviewed to understand the effect of doweled or undoweled transverse joints, and related design elements, to actual pavement performance. Literature were identified with the assistance of the TRIS database and WisDOT research reports. Literature sources were divided into Wisconsin DOT, other DOTs, and national studies (e.g., FHWA, NCHRP).

2.2 Wisconsin DOT Studies

A summary of studies for Wisconsin DOT are provided in Table 2.1. These include two reports directly related to this study, along with two other reports that evaluated design elements. Reports directly related to the test sections in this study include those by Weiss (1992), Rutkowski (1992a; 1993), Crovetti (1995), and Rutkowski et al. (1998).

The reports by Weiss (1992) and Rutkowski (1992a; 1993) evaluated four PCC and three HMA test sections along USH 151 and STH 73 in Dane and Columbia Counties. The initial reports by Weiss (1992) and Rutkowski (1992a; 1993) were published in a series of phases (Phase II, III, and IV) to coincide with the FHWA Open-Grade Base Course National Open House. This FHWA demonstration project focused on the research and development of OGBC as an alternative to DGBC. Analysis of FWD data found no substantial variation in the measured load transfer efficiencies (LTE) between five PCC test sections constructed with different base types including DGBC, non-stabilized OGBC, non-stabilized New Jersey OGBC, asphalt-stabilized OGBC, and cement-stabilized OGBC. LTE showed 90% load transfer for the OGBC test sections. The substitution of OGBC for DGBC adds approximately \$55,000 to \$110,000 per mile to the structural cost depending upon base course gradation and material stabilization. The substitution of asphalt-stabilized OGBC for the standard HMA paving system adds approximately \$44,000 per mile to the structural cost (Rutkowski 1992a).

Table 2.1 Literature Summary of Wisconsin DOT Studies

Reference (1)	Key Findings (2)
Weiss, 1992; Rutkowski 1992a, 1993	• Evaluated test sections along USH 151 and STH 73 in Dane and Columbia Counties. • After 4 years pavement service, there was no substantial variation in the measured load transfer efficiencies between sections constructed with different base types (DGBC, non-stabilized OGBC, non-stabilized New Jersey OGBC, asphalt-stabilized OGBC, and cement-stabilized OGBC). • Substitution of OGBC for DGBC adds approximately $55,000 to $110,000 per mile to the structural cost. • Substitution of asphalt-stablized OGBC for the standard HMA paving system adds approximately $44,000 per mile to the structural cost
Crovetti, 1995; Rutkowski et al. 1998	• Dowels and asphalt-stabilized OGBC provided the greatest protection against joint faulting. • Use of dowels and asphalt-stabilized OGBC in combination did not provide significantly better performance than using either of these measures separately.
Wen and Chen, 2007	• Thick concrete slabs result in lower initial pavement roughness than thin slabs. • Pavements located in urban areas have higher initial pavement roughness than those in rural areas. • There is no statistically significant difference of initial pavement roughness resulting from dowel bar placement methods, either dowel baskets or inserted dowel bars. • There are no differences in initial pavement roughness resulting from base types, including CABC, OGBC, and OGBC2, except that for rural pavements, OGBC2 results in statistically higher initial pavement roughness than does CABC. • Joint spacing is not a statistically significant factor affecting initial pavement roughness. • Longer paving projects in urban areas result in lower initial pavement roughness.
Crovetti, 2006	• Fiber-reinforced polymer (FRP) composite dowels may not be a practical alternative to conventional epoxy coated steel dowels due to their reduced rigidity, which results in lower deflection load transfer capacities at transverse joints. • Reduced placements of solid stainless steel dowels also indicate reduced load transfer capacity and increased IRI values as compared to similarly designed sections incorporating epoxy coated dowels. • Reduced doweling in the driving lane wheel paths also is detrimental to performance for most constructed test sections. • Sections constructed with variable slab geometry and drainage designs indicate that one-way surface and base drainage designs are performing as well as or better than standard crowned pavements with two-way base drainage. • Drainage capacity of the base layer, constructed with open graded number 1 stone, appears sufficient to handle all infiltrated water.

A separate report published by Rutkowski (1992b) evaluated other test sections in the state constructed with variable design elements, including USH 18/151 in Dane and Iowa Counties, USH 14 in Dane County, STH 164 in Waukesha County, STH 50 in Kenosha County, and STH 29 in Brown County. In 1988, 17 test sections were constructed on USH 18/151 to study the effects of dense and open graded base courses (stabilized and non-stabilized), several drain systems, and doweled and non-doweled transverse joints. The key finding from this report was that OGBC appears to provide better pavement performance than the standard base course system after five years of faulting experience on one project (USH 14, Dane County) of the drained pavements study. Otherwise, a pavement structure that places the OGBC directly on the subgrade has resulted in the same pavement performance as the standard DGBC system. The average faulting of both type of base course systems was similar after four years of monitoring. The OGBC test section measured a diminished level of average faulting (0.05 inches) compared to standard DGBC sections (0.13 inches). There was no apparent benefit to the installation of an edge drain at the outside edge of the outside shoulder of a DGBC structure, and it was recommended that this type of edge drain installation be discontinued.

The 1992 report also observed that pavement width may have had an effect on performance (Rutkowski 1992b). For example, on USH 18/151, Iowa County, pavement sections were constructed with 14-foot wide driving lanes. The other project pavements used as the basis for early faulting distress were constructed with 12-foot driving lane slabs. Based on conclusions from Rutkowski (1992b), the lack of early faulting in 14-foot wide driving lane pavements can possibly be explained by a "wider slabs theory". The 12-foot wide slabs constructed in 1983 allowed the outer wheel path to be approximately 3 feet from the outside edge of the pavement. This is thought to promote faulting (pumping of fines) at the pavement/base course interface of the transverse joint. The most intense faulting takes place in the outer lane. The pavements constructed in 1988 had a 14-foot wide outside slab allowing the shoulder stripe to be placed 2 feet in from the edge of the pavement. The outer wheel path is then 5 feet from the outside edge of the pavement. It was theorized that this greater distance to the pavement outer edge at the transverse joint places less stress due to loading on the outside corner and that the faulting (pumping) mechanism is diminished to a large degree. This may have resulted in a lower rate of faulting on this pavement configuration. Based on the this report, the 1988 test sections with dense graded base did not have the intensity of faulting seen in the test sections of the 1983 pavements at similar age (Rutkowski 1992b).

Rutkowski (1992) also observed that it was not possible to differentiate between the provision or absence of dowel load transfer systems and transverse joint sealing systems (USH 18/151 Iowa and Dane Counties) on the basis of pavement distress index or average transverse joint faulting. It could not be stated that the provision or absence of dowel load transfer and transverse joint sealing would be superior. The average faulting values are extremely low on all test sections. It was not possible to determine a benefit for load transfer devices after four years of transverse joint fault and pavement distress index monitoring. There was only a slight difference in average faulting or pavement distress index for test sections constructed with the provision or absence of dowel load transfer devices at transverse joints for either OGBC, DGBC, or TIC drain systems. The addition of retrofit edge drains to a dense graded base course pavement structure ground to profile had not prevented the development of subsequent faulting. Between 75 and 100% of the original degree of faulting has returned to the pavement transverse joints in four years or less. Retrofit edge drains have not proven to be an effective short term rehabilitation method. It was recommended that the use of retrofit edge drains to prevent renewed faulting be discontinued as a rehabilitation method (Rutkowksi 1992).

A report by Rutkowski et al. (1998) provided a comprehensive background and evaluation of concrete and asphalt pavement test sections constructed throughout the state, while a report by Crovetti (1995) addressed the USH 18/151 test sections in Iowa and Dane Counties. The purpose of the reports was to document performance after the pavement had been in service for 7 to 10 years. The major conclusions of these reports were that dowels and asphalt-stabilized OGBC provided the greatest protection against joint faulting, but use of dowels and asphalt-stabilized OGBC in combination did not provide significantly better performance than using either of these measures separately. However, both reports recommended that a 20-year performance of these pavement segments be evaluated, providing the motivation for this study.

In the report by Rutkowski et al. (1998), PCC and AC pavement test segments constructed from 1987 to 1991 were listed. Tables 2.2 and 2.3 provide the characteristics for those segments, where Table 2.2 presents primary PCC projects constructed in 1987 and 1988, and Table 2.3 presents secondary PCC and AC projects constructed in 1988. Primary projects were designed to compare various formats of positive drainage features, as well as dowel/non-dowel and sealant design features. Secondary PCC and asphaltic concrete (AC) segments researched positive drainage concepts, but on a less comprehensive scale.

Table 2.2 Primary PCC Projects constructed in 1987 and 1988 (Rutkowski et al. 1998)

PROJECTS	Total Sect	Base Course WisDOT #1 Stabilization AS	CS	NS	#2	DBG	Transverse Joints Unseal	Seal	TIC Drains	Edge Drains Std	Fin	Wrapped Pipe	None	Dowel Trans Joints	Grind Surface
PCC Drain Study															
STH 14	3	1				2		3			2	1			
USH 18-151 Iowa County	13	2	2	2		7	6	7	2	6	1-Retro	1-Retro	3	1	3
STH 29	4			4			1	3			4			2	
STH 50	6					6	3	3			2	2	2		
STH 164	6					6	3	3				2	4		
Dowel Study PCC USH 18-151 Dane County	6	1	1	1		3	5	1		3			3	6	

Notes:
Sect ... control/test sections
AS ... asphalt
CS ... cement
NS ... nonstabilized
DGB ... dense graded
#1 ... permeability = 10,000 feet/day
#2 ... permeability = 500 feet/day
TIC ... transverse joint drains
Std ... standard
Fin ... fin-type
Retro ... retrofitted

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Table 2.3 Secondary PCC and AC Projects constructed in 1991 (Rutkowski et al. 1998)

PROJECTS	Total Sect	Base Course Wis DOT#1 Stabilization AS	CS	NS	#2	DGB	Transverse Joints NJ	No Seal	Seal	TIC	Edge Drains Std	Fin	None	Retro	Day Light	Grind Surf
PCC																
IH 43 Ozaukee Co	10					10			10	6	5		3		2	10
USH 151 Columbia Co	5*	1	1	1		1	1	5			4		1			
IH 43 Walworth Co	4					4			4				1	3		4
AC																
STH 60 Washington Co	4	1		1	1	1					3		1			
STH 167 Washington Co	2			1		1				1	2					
USH 151 Columbia Co	2*	1				1					1		1			
STH 73 Columbia Co	2	1									1		1			

Notes: oth AC and PCC sections.
Sect … control/test sections
AS … asphalt
CS … cement
NS … nonstabilized
DGB … dense graded
NJ … New Jersey gradation
X … quantity unknown
TIC … transverse joint drains
Std … standard
Fin … fin-type
Retro … retrofitted
#1 … permeability = 10,000 feet/day
#2 … permeability = 500 feet/day

file: TestSect1.xls 1

A Wisconsin DOT study by Wen and Chen (2007) analyzed the design and construction factors affecting initial pavement roughness. Initial IRI of 90 concrete pavements constructed in Wisconsin from 2000 to 2004 were analyzed using multiple regression methods. The factors considered in this study included concrete pavement slab thickness, project location, dowel bar placement, joint spacing, base type, and pavement length. The factors affecting initial pavement roughness were identified. Thicker concrete slabs result in lower initial pavement roughness than do thinner concrete slabs. Pavements located in urban areas have higher initial pavement roughness than those in rural areas. There is no statistically significant difference of initial pavement roughness resulting from dowel bar placement methods, either dowel baskets or inserted dowel bars. There are no differences in initial pavement roughness resulting from base types, including CABC, OGBC, and OGBC2, except that for rural pavements, OGBC2 results in statistically higher initial pavement roughness than does CABC. Joint spacing is not a statistically significant factor affecting initial pavement roughness. Longer paving projects in urban areas result in lower initial pavement roughness.

A Wisconsin DOT study by Crovetti (2006) evaluated alternate pavement designs targeted at reducing the initial construction costs of concrete pavements without compromising pavement performance. Test sections were constructed with alternate dowel materials, reduced dowel placements, variable thickness concrete slabs and alternate surface and subsurface drainage details. Performance data were collected out to 5 and 7 years after construction. The study results indicate that fiber reinforced polymer (FRP) composite dowels may not be a practical alternative to conventional epoxy coated steel dowels due to their reduced rigidity, which results in lower deflection load transfer capacities at transverse joints. Ride quality measures also indicate higher IRI values on sections constructed with FRP composite dowels. Study results for sections constructed with reduced placements of solid stainless steel dowels also indicate reduced load transfer capacity and increased IRI values as compared to similarly designed sections incorporating epoxy coated dowels. Reduced doweling in the driving lane wheel paths also is shown to be detrimental to performance for most constructed test sections. The performance of sections with reduced doweling in the passing lane wheel paths indicates that this alternate may be justifiable to maintain performance trends similar to those exhibited by the driving lane with standard dowel placements. Performance data from sections constructed with variable slab geometry and drainage designs indicate that one-way surface and base drainage designs are performing as well as or better than standard crowned pavements with two-way base drainage. The drainage capacity of the base layer, constructed with open graded number 1 stone, appears sufficient to handle all infiltrated water.

2.3 Other Agency Studies

A summary of studies from DOTs other than Wisconsin are provided in Table 2.4. A brief paragraph summary of each source is provided.

Table 2.4 Literature Summary of DOTs other than Wisconsin

Reference (1)	Key Findings (2)
Chen et al., 2008, Texas DOT	• Premature asphalt pavement failure was attributed to disintegration of the cement-stablized base layer. • Asphalt pavement failure was attributed to two primary factors: 1) a very coarse gradation of the aggregate used in the cement-stablized layer which produced a mix prone to segregation during placement; and 2) the cement-stabilized layer was placed in 2 lifts, which were not well bonded together. • Another contributing factor was the lack of bond between the cement-stabilized base and the asphalt pavement surface layer.
Rahman et al., 2008, Kansas DOT	• Key pavement distresses (deformation and roughness) are insensitive to the subgrade modulus. • Asphalt-stabilized base was used, and it was determined that base layer thickness has more influence on the total pavement deformation than the subbase layer. • The influence of subgrade modulus on the slab thickness is insignificant.
Gisi et al., 2007, Kansas DOT	• Pavement drainage is critical to performance. • Both daylighted and partially daylighted drainage systems of various configurations can perform as well as a system using a positive drainage system of pipes and outlets. • Both systems do not have the inherent problems of a pipe system clogging. • Winter freeze condition can affect the outflow of water from the base and this condition may not be desirable in harsh freezing environments. • Drainable PCC sections with permeable asphalt treated base (PATB) have performed the best.
Elfino and Hossain, 2007, Virginia DOT	• Lack of positive drainage along with heavily loaded truck traffic resulted in premature failure. • Water entering the pavement because of poor joint sealing was trapped in the open-graded drainage layer, and led to severe faulting, midslab cracks, pumping, and eventual failure of the pavement.
Sargand et al., 2006, Ohio DOT and FHWA	• Multiple base types in Ohio and North Carolina were evaluated including granular, lean concrete, asphalt treated, cement treated, and permeable asphalt treated. • Type of base had little impact on subgrade moisture. • The choice of base depends chiefly on three requirements: appropriate stiffness, sufficient permeability, and good constructability.
Chowdhury and Hossain, 1999, Kansas DOT	• Three FWD tests per mile are recommended for the network-level evaluation. • The decrease in the structural number values obtained from the models developed in this study was about 33% higher than the KDOT design assumption. • The Bayesian regression models developed are very similar in form to the classical regression models and yielded statistically similar results when tested on a different set of pavements. However, the Bayesian regression models appeared to give slightly better results for some pavements during testing.

Although a TxDOT study by Chen et al. (2008) evaluated asphalt pavement performance, it did highlight the degradation of a cement-stablized base treatment and the effect on pavement performance. After only 2 months in service, the frontage road of U.S. 290 in Houston, Texas, developed a series of depressions that caused a very poor ride. The main cause of the premature failure was attributed to disintegration of the cement treated base (CTB) layer. This was attributed to two primary factors: 1) a very coarse gradation of the aggregate used in the CTB layer which produced a mix prone to segregation during placement; and 2) the CTB layer was placed in 2 lifts, which were not well bonded together. Another contributing factor was the lack of bond between the CTB and the hot mix asphalt (HMA) surface layer. Secondary factors include high air voids in the HMA layer and low HMA layer thickness. The material, when prepared carefully in the lab at the design cement content, passed the strength requirement of 2.07 MPa. But this coarse mix appears to have been difficult to place correctly in the field. The coarsely graded aggregate used on this project appears to be prone to segregation, either during placement or compaction. The ground penetration radar results (with confirmation by core samples) indicated that most of the problems were at the bottom of the upper CTB lift. The CTB was placed in 2 lifts and very poor condition was found between the CTB layers. This problem was coupled with a thin, porous, and poorly bonded HMA layer that permitted moisture to enter the CTB layer. Similar failures have also been reported recently on other CTB projects in Houston.

A Kansas DOT study by Rahman et al. (2008) evaluated the effect of variation of the subgrade resilient moduli on the pavement design using the MEPDG analysis. Subgrade modulus values were obtained from three test sections on two routes in Kansas with an Intelligent Compaction (IC) roller and from the deflection tests using a Falling Weight Deflectometer (FWD). The deflection data was used to backcalculate the subgrade moduli using an elastic layer analysis backcalculation program and the Boussinesq's equation. The pavement design analysis for various subgrade moduli was done with the MEPDG v1.0 software. The results show that the predicted total pavement deformation and roughness are sensitive to the subgrade modulus for flexible pavements. For JPCP, the key distresses are insensitive to the subgrade modulus. Asphalt base thickness has more influence on the total pavement deformation than the foundation layer. However, truck traffic plays an even more significant role in controlling this distress. The influence of subgrade modulus on the slab thickness in the JPCP design is insignificant. The "target" subgrade modulus for intelligent compaction control can be derived well before construction based on the soil type and asphalt base thickness and using the M-EPDG analysis. Achievement of this modulus in the field will lead to a reliable pavement structure for a given design period.

A TRB proceedings paper by Gisi et al. (2007) discusses KDOT experience with drainage of six in-service concrete projects. The study found that pavement drainage is critical to performance. Since 1988, the Kansas Department of Transportation (KDOT) has been using a drainable base layer as an option for the PCC pavements. However, the majority of PCC pavements in Kansas do not incorporate a drainable base because the traffic volume is low to medium. Four of these projects were the experimental sections chosen from the Kansas SPS-2 project located on I-70 and incorporate a permeable asphalt treated base (PATB) layer with edge drains. The other two projects, US-50 and US-400, had daylighted drainable base layers. These projects also incorporated some alternative drainage designs and instrumentation for drainage monitoring. Both daylighted and partially daylighted drainage systems of various configurations can perform as well as a system using a positive drainage system of pipes and outlets. Both systems do not have the inherent problems of a pipe system clogging. However, the winter freeze condition can affect the outflow of water from the base and this condition may not be desirable in harsh freezing environments. On the SPS-2 project, the drainable PCC sections with permeable asphalt treated base (PATB) have performed the best. These sections were built smoother and remained so after 13 years of service. Kansas experience has also reinforced the need for an acceptable separator layer in the drainable PCC design.

In a Virginia DOT study by Elfino and Hossain (2007), field and laboratory forensic investigations were used to identify the failure mechanism of a jointed plain concrete pavement with a subsurface drainage system in Virginia. Similar to many states' practice, this subsurface drainage system consists of open-graded drainage layer and edge drains to provide positive drainage for the pavement. The investigation included a review of construction practices and pavement performance records, a visual distress survey, nondestructive testing using a falling weight deflectometer, roughness measurements using a profiler, coring and boring for materials testing, observation wells, subgrade soil classification, mineralogy, determination of concrete compressive strength, edge drain camera inspection, and slab removal. On the basis of the investigation, it was concluded that lack of positive drainage along with heavily loaded truck traffic resulted in premature failure. The water entering the pavement because of poor joint sealing was trapped in the open-graded drainage layer; this led to severe faulting, midslab cracks, pumping, and eventual failure of the pavement.

An Ohio DOT study by Sargand et al. (2006) investigated how base materials should be properly selected for specific types of pavement, not only considering the performance of individual layers but also how they interact in the total pavement structure. Base types considered in this study included granular (GB), lean concrete (LCB), asphalt treated (ATB), cement treated (CTB), and permeable asphalt treated (PATB) bases as constructed under both asphalt and concrete pavements. The LTPP Seasonal Monitor Program (SMP) sites investigated for this report included four SMP sections in the North Carolina SPS-2 experiment on US52 and thirteen SMP sections in the SPS-1 and SPS-2 experiments on the Ohio SHRP Test Road on US23. The NC site contained two GB and two LCB sections, and the OH site contained eight GB, one ATB, two PATB, and two LCB sections. The NC sites are located in a wet-no-freeze zone and OH sites are located in a wet-freeze zone. Environmental data were collected via seasonal monitors and time domain reflectometry. The effects of service were measured by conducting surface profiles and FWD measurements. It was found that the type of base had little impact on subgrade moisture. The choice of base depends chiefly on three requirements: appropriate stiffness, sufficient permeability, and good constructability. Guidelines for the selection of base under flexible and rigid pavements were developed.

An earlier KDOT report by Chowdhury and Hossain (1999) developed a pavement rating attribute, known as the Pavement Structural Evaluation (PSE), using FWD tests and network-level distress surveys. These ratings are subjective and based on the condition of the pavement as indicated by the visual distresses and maintenance histories and the ability of the section to provide an adequate surface for the prevailing traffic. PSE is expected to be an indicator of the structural deficiency of the pavement sections. However, since KDOT does not collect any deflection data at the network level, the PSE computation process does not directly take into account any structural evaluation. The regression models proposed in this study predict the decrease in PSE values by taking into account the FWD data, age, thickness, and distress levels of pavements, and very closely approximate the current PSE ratings obtained at the district level. FWD data on approximately 20% of the KDOT network is needed for network level structural evaluation. This translates into 750 lane-miles (1207 lane-km) of FWD testing per year. Three FWD tests per mile are recommended for the network-level evaluation. This testing would also be necessary for using/updating the models developed in this study. The decrease in the structural number values obtained from the models developed in this study was about 33% higher than the KDOT design assumption. A parallel study at Kansas State University used the Bayesian Regression methodology developed by the Canadian Strategic Highway Research Program. The Bayesian regression models developed are very similar in form to the classical regression models and yielded statistically similar results when tested on a different set of pavements. However, the Bayesian regression models appeared to give slightly better results for some pavements during testing.

2.4 National and Other Studies

A summary of national studies, including FHWA, FAA, and NCHRP, along with studies from non-agency specific studies are provided in Table 2.5.

Table 2.5 Literature Summary of National and Other Studies

Reference (1)	Key Findings (2)
Hall and Crovetti, 2007, NCHRP	• The presence of subsurface pavement drainage could not be readily identified as having a positive impact on pavement performance. • Deflection response, roughness, rutting, faulting, and cracking were found to be influenced by the stiffness, rather than the drainability. • Best-performing pavements were those with bases that were neither too weak (non-stabilized) nor too stiff (lean concrete).
Prabhu et al., 2007	• Principal stresses that develop between the concrete panel-dowel interface were measured using 3-D finite element models. • When steel dowels are misaligned, more stress is developed.
Mallela et al., 2007, FAA	• Concrete pavements constructed over certain dense-graded bases have a higher risk of early-age, uncontrolled cracking.
Buch et al., 2006, NCHRP	• The performance of key design elements were investigated including slab thickness, base type, drainage, flexural strength, and slab width. • Base type was the most critical design factor affecting performance in terms of cracking and IRI. • Pavement sections with a permeable asphalt-treated base and in-pavement drainage performed better than those with a dense-graded aggregate base or a lean concrete base. • PCC slab thickness also played an important role in improving the cracking performance of the pavements. • PCC flexural strength and slab width have only marginal effects on performance at this time.
Jiang and Darter, 2005, FHWA	• No SPS-2 projects were built on certain subgrade types and in some climates. • Some SPS-2 sites had construction deviations, and significant materials data and traffic data are missing from other sites or sections.
Khazanovich and Gotlif, 2003, FHWA	• Load transfer efficiency (LTE) indexes and joint stiffnesses were calculated. • LTE depends on FWD load plate position and testing time. • It is recommended that FWD LTE testing be conducted in the early morning in cool weather to provide realistic estimation of LTE. • LTE of CRCP cracks was higher than LTE of joint in JPCP. • LTE of doweled joints was higher than non-doweled joints. • Non-doweled sections with a high level of LTE are less likely to develop significant faulting than sections with low LTE. • LTE from leave and approach side deflection testing data was found to be statistically different for a large number of JPCP sections.
Davids et al., 2003	• Finite element models in 3-D were created to understand stress interactions. • Dowel locking and slab-base shear transfer can significantly affect the stresses in slabs subjected to both uniform shrinkage and thermal gradients. • Joint load transfer is greatly reduced by dowel looseness. • Transverse joint mislocation can significantly reduce peak dowel shears, but has relatively little effect on total load transferred across the joint.

In a comprehensive NCHRP study by Hall and Crovetti (2007), LTPP SPS-1 and SPS-2 pavement sections were evaluated for load deflection and flow testing of pavement drainage systems. The study did not identify any aspect of the behavior or performance of the HMA and PCC pavement structures that could have been improved by the presence of subsurface pavement drainage. Instead, the measures of pavement behavior and performance analyzed for these pavements - namely, deflection response, roughness, rutting, faulting, and cracking - were found to be influenced by the stiffness, rather than the drainability, of the base layers (Hall and Crovetti 2007). Overall, the best-performing PCC pavements in the SPS-2 experiments were those with bases that were neither too weak (untreated aggregate) nor too stiff (lean concrete). These include the sections with drained permeable asphalt-treated base, but also the sections with undrained HMA base and cement-aggregate-mixture base.

A study by Prabhu et al. (2007) investigated the effects of dowel misalignment on the joint opening behavior and distress in concrete pavement joints. A key finding of the study was the ability to measure and identify principal stresses that develop between the concrete panel-dowel interface when the steel dowels are misaligned in transverse joints, suggesting that steel dowels carry considerable load transfer between adjacent panels. Three-dimensional (3-D) finite element models were created for computing the complex stress states and resulting damage in concrete pavement joints with misaligned dowels. The concrete pavement was modeled using a damage–plasticity material model, which uses concepts of damage–plasticity formulation in compression and cracking combined with damage elasticity in tension. The longitudinal bond between the steel dowel and the concrete was modeled in two parts. First, the longitudinal bond resulting from chemical adhesion, mechanical interlock, and static friction (in the aligned state) is modeled by means of spring elements. The nonlinear force–deformation relationship for the spring elements is derived from specific experimental results. Second, the longitudinal bond resulting from transverse interaction between steel dowels and the concrete pavement is modeled by surface-to-surface contact interaction elements and associated friction models. The 3-D finite element models are validated by the results of experimental investigations. These validated models provide significant insight into the 3-D stress states and principal stresses that develop in concrete pavement joints with misaligned dowels. They are used to evaluate analytically the effects of misalignment type, magnitude, uniformity, and distribution on the 3-D stress states and resulting damage in concrete pavements.

Mallela et al. (2007) reports that under certain circumstances concrete pavements constructed over certain types of bases have a higher risk of early-age, uncontrolled cracking. In some cases, this has resulted in the removal and replacement of up to 5% to 7% of the total number of slabs paved on a project. An investigation of nearly two dozen airfield pavement sections in the United States identified several plausible factors that act either independently or in concert with other factors and lead to this phenomenon. This study attempted to explain the interaction between factors that trigger slab movements (triggers) and key design, material, and construction factors (variants) that aggravate the impact of these movements on early cracking risk. On the basis of this study, guidelines for design, materials selection, and construction of rigid pavements on stabilized and drainable bases were developed to mitigate the impact of various factors on the early-age cracking phenomenon. Revisions were suggested to the FAA's specifications for lean concrete, cement-treated, and hot-mix asphalt bases. New specifications were developed for permeable bases that balance stability with drainability.

The relative effects of various design and site factors on the performance of JPCP were researched by Buch et al. (2006) in NCHRP Project 20-50 (10&16), "LTPP Data Analysis: Influence of Design and Construction Features on the Response and Performance of New Flexible and Rigid Pavements". The data used in this study were primarily drawn from Release 17 of DataPave. An SPS-2 experiment was designed to investigate the effects of slab thickness, base type, drainage, flexural strength, and slab width on the performance of JPCP. On the basis of the statistical analysis of 167 test sections, ranging in age from 5 to 12 years, it was concluded that base type was the most critical design factor affecting performance in terms of cracking and roughness as measured by the IRI. Pavement sections with a permeable asphalt-treated base and in-pavement drainage performed better than those with a dense-graded aggregate base or a lean concrete base. PCC slab thickness also played an important role in improving the cracking performance of the pavements. PCC flexural strength and slab width have only marginal effects on performance at this time.

An FHWA report by Jiang and Darter (2005) documented the first comprehensive review and evaluation of the SPS-2 experiment, "Strategic Study of Structural Factors for Jointed Plain Concrete Pavements (JPCP)". The main objective of this experiment is to determine the relative influence and long-term effectiveness of JPCP design features (including slab thickness, portland cement concrete flexural strength, base type and drainage, and slab width) and site conditions (traffic, subgrade type, climate) on performance. Thirteen SPS-2 projects have been constructed with one additional site under construction. At each site, there are 12 core sections plus various numbers of supplemental sections. The data availability and completeness for the SPS-2 experiment are good overall. A high percentage of the SPS-2 data are at level E--greater than 82% for all data types, and greater than 99% for many. However, a significant amount of data are still missing, especially traffic, distress and faulting surveys, and key materials testing data. These deficiencies need to be addressed before a comprehensive analysis of the SPS-2 experiment is conducted. Required experimental pavement design factors and site conditions were compared with the actual constructed values. Most SPS-2 sections follow the experiment design for the large majority of the design factors. When comparing designed versus constructed, eight SPS-2 projects can be characterized as good to excellent, four projects are considered poor to fair, and one new SPS-2 project does not yet have enough data in the IMS database to be evaluated. The evaluation has shown that several problems may limit the results that can be obtained from the SPS-2 experiments if not rectified. Specifically, no SPS-2 projects were built on certain subgrade types and in some climates. Some SPS-2 sites had construction deviations, and significant materials data and traffic data are missing from other sites or sections. One site has excessive early cracking that will limit its usefulness. However, even though the SPS-2 sections are relatively young (oldest project is 7.5 years) and a large majority show no or little distress, some interesting and important early trends have already been identified that will be very useful to the design and construction of JPCP. As time and traffic loadings accumulate, much more valuable performance data will be obtained. The Federal Highway Administration is conducting a concerted effort to obtain missing data. Recommendations for future analyses are provided in the last chapter of this report. Valuable information will be obtained from this experiment if these studies are carried out.

A paper by Davids et al. (2003) describes computer software, EverFE2.2, that can be used for 3-D finite element modeling. The software has the ability to model multiple-tied slabs or shoulders, model dowel misalignment or mislocation, treat nonlinear thermal or shrinkage gradients, and simulate nonlinear horizontal shear stress transfer between the slabs and base. The results of two parametric studies are reported in this paper. The first study considers the effects of dowel locking and slab-base shear transfer and demonstrates that these factors can significantly affect the stresses in slabs subjected to both uniform shrinkage and thermal gradients. The second study examines transverse joint mislocation and dowel looseness on joint load transfer. As expected, joint load transfer is greatly reduced by dowel looseness. However, while transverse joint mislocation can significantly reduce peak dowel shears, it has relatively little effect on total load transferred across the joint for the models considered.

An ERES Consultants study for the FHWA evaluated load transfer efficiency (LTE) of cracks and joints for rigid pavements included in the Long-Term Pavement Performance (LTPP) program (Khazanovich and Gotlif, 2003). This study presents the first systematic analysis of the deflection data collected under the LTPP program related to LTE. Representative LTE indexes and joint stiffnesses were calculated for all General Pavement Studies (GPS), Special Pavement Studies (SPS), and Seasonal Monitoring Program (SMP) rigid test sections. Data tables that include computed parameters were developed for inclusion in the LTPP Information Management System (IMS). Trend analysis was performed to evaluate the effect of design features and site conditions on LTE. One key finding was LTE is a complex parameter, which depends on many factors, including falling weight deflectometer (FWD) load plate position, and testing time, where FWD LTE testing must be conducted in the early morning in cool weather to provide realistic estimation of LTE. Another finding was that LTE of continuously reinforced concrete pavements (CRCP) cracks was found to be higher than LTE of joint in jointed concrete pavements (JCP). Also, LTE of doweled joints was found to be higher than LTE of nondoweled joints. Nondoweled sections with a high level of LTE are less likely to develop significant faulting than sections with low LTE. Finally, LTE from leave and approach side deflection testing data was found to be statistically different for a large number of JCP sections.

2.5 FWD Publications

FWD testing is an integral component of this research study. To understand limitations and lessons learned from previous field testing, several literature sources were reviewed. A brief review of three relevant sources are cited.

A synthesis by Alavi et al. (2008) reported on the state of the practice of FWD usage as it involves state DOTs using these devices to measure pavement deflections in response to a stationary dynamic load, similar to a passing wheel load. The data obtained are used to evaluate the structural capacity of pavements for research, design, rehabilitation, and pavement management practices. It is anticipated that this synthesis will provide useful information to support guidelines, advancing the state of the practice for state DOTs and other FWD users, as well as equipment manufacturers and others involved in pavement research, design, rehabilitation, and management. Based on a survey conducted for this report, 45 state highway agencies (SHAs) reported using 82 FWDs, produced by 3 different manufacturers. The importance of FWDs among SHAs appears to be reflected in the survey results, as it was noted that SHAs conduct FWD tests on up to 24 100 lane-km (15,000 lane miles) annually.

A publication by the FHWA (1997) describes that FWD load deflection-time data can be used to measure the dissipated work during the loading and unloading of the pavement structure from the FWD impact load. An important property of materials that defines the viscoelastic and inelastic characteristics of materials is the dissipated work or dissipated energy of the material. Dissipated energy has been used in the asphalt concrete fatigue area for many years by some agencies. This dissipated work should be related to the occurrence of selected surface distresses, especially for asphalt concrete-surfaced pavements. The deflection-time history data collected within the LTPP program represent an invaluable data source and critical data element that has yet to be thoroughly investigated and used to its full potential in pavement diagnostic studies. A limited study was undertaken to determine if there is any relationship between the dissipated work as measured with the FWD and levels of pavement distress. The study also shows some of the different parameters that can be used from the deflection-time data and the benefit of using these data for pavement diagnostic studies and pavement classifications.

A publication by the FHWA cautions that the FWD must be properly calibrated (FHWA 2002). To make the best possible decisions about where and when to conduct pavement rehabilitation work, State departments of transportation (DOTs) need extensive data on the structural condition of pavement. To measure the structural condition of pavement, most pavement engineers rely on FWD technology. FWDs "thump" the pavement and record information about its structure and integrity. But like all sophisticated tools, the FWD must be properly calibrated. If it is not, measurements will be inaccurate. Inaccuracy wastes precious budget dollars.

2.6 Summary

A comprehensive literature review was conducted to identify factors affecting PCC pavement performance. Table 2.6 summarizes primary findings by design element.

Table 2.6 Summary of Literature for PCC Pavement Performance by Design Element

Design Element (1)	Primary Findings (2)
Base Type	• Substitution of OGBC for DGBC adds approximately $55,000 to $110,000 per mile to the structural cost (1993 cost). • There are no differences in initial pavement roughness resulting from base types, including DGBC, OGBC, and OGBC2, except that for rural pavements, OGBC2 results in higher initial pavement roughness than DGBC. • Best-performing pavements were those with bases that were neither too weak (non-stabilized) nor too stiff (lean concrete). • Concrete pavements constructed over certain dense-graded bases have a higher risk of early-age, uncontrolled cracking.
Drainage	• Drainage capacity of the base layer, constructed with open graded number 1 stone, appears sufficient to handle all infiltrated water. • Both daylighted and partially daylighted drainage systems can perform as well as a system using a positive drainage system of pipes and outlets. • Drainable PCC sections with ASOG have performed the best. • The presence of subsurface pavement drainage could not be readily identified as having a positive impact on pavement performance. • Pavement sections with a permeable asphalt-treated base and in-pavement drainage performed better than those with a dense-graded aggregate base or a lean concrete base.
Dowels	• Dowels and ASOG provided the greatest protection against joint faulting. • Use of dowels and asphalt-stabilized OGBC in combination did not provide significantly better performance than using either of these measures separately.
Strength	• Key pavement distresses (deformation and roughness) are insensitive to the subgrade modulus. • When ASOG base was used it was determined that base layer thickness has more influence on the total pavement deformation than the subbase layer.
Deflection	• Deflection response, roughness, rutting, faulting, and cracking were found to be influenced by the stiffness, rather than the drainability. • LTE depends on FWD load plate position and testing time. • LTE of doweled joints was higher than non-doweled joints. • Non-doweled sections with a high level of LTE are less likely to develop significant faulting than sections with low LTE. • LTE from leave and approach side deflection testing data was found to be statistically different for a large number of JPCP sections.
Sealant	• Water entering the pavement because of poor joint sealing was trapped in the open-graded drainage layer, and led to severe faulting, mid-slab cracks, pumping, and eventual failure of the pavement.

CHAPTER 3 EXPERIMENTAL DESIGN

3.1 Introduction

A field experiment was designed to collect and analyze data from the 17 USH 18/151 test sections, 4 STH 29 test sections, and 4 USH 151 test sections. This data allowed a comparison of unique features of each section to determine effects between subgrade support, drainability, load transfer, joint seal, and overall performance.

The condition of each test location was documented by visual surveys in November 2008 and April 2009. County highway departments were also contacted to understand the current pavement condition and whether any pavement rehabilitation was planned. It was learned that USH 14 Dane County had been overlayed with hot-mix asphalt during the 2008 construction season. Also, it was learned that USH 18/151 test sections were planned for a dowel-bar retrofit in the 2009 construction season, thus, field research testing of these sections was scheduled prior to that project. Table 3.1 summarizes the condition of the three project test locations. Figure 3.1 illustrates a field survey of USH 18/151 in April 2009.

Table 3.1 Condition of Pavement Test Sections prior to Research Testing

Project Location (1)	Test Sections (2)	Lane Direction (3)	Condition (3)
USH 18/151 Iowa and Dane Counties	17	Eastbound, 14 sections Westbound, 3 sections	Westbound dowel-bar retrofit in 2004; Eastbound dowel-bar retrofit in summer 2009.
STH 29 Brown County	4	Eastbound	Original construction
USH 151 Columbia and Dane Counties	4	Westbound	Original construction

Figure 3.1 Field Verification of Test Sections on Eastbound USH 18/151

3.2 USH 18/151 Test Sections

The USH 18/151 test sections have multiple levels of individual factors, as illustrated in Table 3.2. A total of 17 sections were constructed with 7 unique design factors across Iowa and Dane County. In the bottom row of Table 3.2 are the total number of levels associated with each primary factor. A factorial experiment to isolate on each unique combination of factors would have required 256 test sections (2 x 4 x 2 x 1 x 4 x 2 x 2 = 256). Obviously, a full factorial design would not be feasible in highway construction projects. Since 17 sections were constructed with 7 unique factors, what was created 20 years ago was largely a fold-over design, where combinations of factors were simultaneously changed to reduce the overall number of combinations. Typical cross sections are illustrated in Appendix B.

Table 3.2 USH 18/151 Test Section Details

Test Section	Base Thickness, inches	Base Type	Subbase Thickness, inches	Subbase Type	Drain Design	Doweled Transverse Joints	Sealed Transverse Joints
1	4	NS	4	DGBC	PAD	No	Yes
2	4	NS	4	DGBC	PAD	No	No
3	4	CS	4	DGBC	PAD	No	Yes
4	4	CS	4	DGBC	PAD	No	No
5	4	AS	4	DGBC	PAD	No	Yes
6	4	AS	4	DGBC	PAD	No	No
7	--	--	6	DGBC	TIC	No	No
7a	--	--	6	DGBC	TIC	No	No
8	--	--	6	DGBC	None	No	Yes
9	--	--	6	DGBC	None	No	No
10	--	--	6	DGBC	TIC	Yes	No
11	4	CS	4	DGBC	PED	Yes	No
12	4	AS	4	DGBC	PED	Yes	No
13	4	NS	4	DGBC	PED	Yes	No
14	--	--	6	DGBC	None	Yes	No
15	--	--	6	DGBC	None	Yes	Yes
16	6	LCBC	--	--	None	Yes	No
LEVELS	2	4	2	1	4	2	2

Pavement thickness all sections = 10 inches;
NS, Non-Stabilized Open Graded Base Course;
CS, Cement-Stabilized Open Graded Base Course;
AS, Asphalt-Stabilized Open Graded Base Course;
LCBC, Lean Concrete Base Course;
DGBC, Dense Graded SubBase Course;
PAD, Pipe/Aggregate Longitudinal Drains;
TIC, Transverse InterChannel Transverse Joint Drains;
PED, Wrapped Trench with 4' Pipe Longitudinal Edge Drain;
None, No Edge Drains.

3.3 STH 29 and USH 151 Test Sections

Test sections on STH 29 in Brown County and USH 151 in Columbia County have a more simplified factorial design than USH 18/151. Four test sections to evaluate dowel/non-dowel performance, and sealed and unsealed joints, were constructed on STH 29 in Brown County. Constructed in 1988, this pavement cross-section consists of a 10-inch JPCP over a 4-inch permeable aggregate base and a 4-inch aggregate subbase. The joints are non-doweled in two test sections and doweled in the other two sections. Joints are sealed and unsealed within each doweled and non-doweled sections.

Constructed in 1991, the all-doweled USH 151 Columbia County project has 10-inch JPCP over 5 unique bases: asphalt stabilized, cement stabilized, non-stabilized, dense graded, and New Jersey graded. Asphalt concrete sections were also included in this project. Typical cross sections for STH 29 and USH 151 are illustrated in Appendix B.

3.4 Data Overlay

The specific location of each test section was identified with the traditional Reference Point (RP) system. As-built construction locations and collected performance data for each segment were overlaid on single project maps using MSN Maps Live™. Five data sets were overlaid on each project map, as listed in Table 3.3. These data sets include: (1) as-built test section end points BROWN, (2) WisDOT distress and profile data collected prior to field testing BLUE, (3) Reference Points for each Sequence Number RED, (4) the 0.1-mile WisDOT PDI location that is typically 0.3 to 0.4 miles from a Reference Point YELLOW, and (5) the proposed 0.1-mile test Research PDI location GREEN where Falling Weight Deflectometer (FWD) and permeability testing were to occur in summer 2009 that is a variable distance from the butt joint in the direction of traffic. During field testing, the location of the 0.1-mile Research PDI segment was chosen using the beginning location of FWD test sites then projecting ahead 528 feet.

Table 3.3 Data Sets overlaid on Maps

Color (1)	Data Set (2)	Notes (3)
BROWN	Construction as-built sections	Construction stationing was used to identify paving butt joints (end points)
BLUE	WisDOT distress and profile data	Each project was measured using the Pathway Van in November 2008 or April 2009.
RED	Reference Points for each Sequence Number	The endpoints for a section of pavement where performance is measured.
YELLOW	WisDOT PDI	The 528-foot segement where performance distresses and roughness are measured. This is located 0.3 to 0.4 miles from a starting Reference Point, such as an intersection.
GREEN	Research PDI	A 528-foot segment of pavement where the performance of the test section was measured. This is located at the start of FWD testing to 528 feet ahead in the direction of traffic.

Figures 3.2 through 3.6 provide the overlay maps for each project for USH 18/151, STH 29/32, and USH 151. Each of these maps has the overall project length, along with the 5 data sets. Circle symbols designate end points of each data set feature.

Figures 3.2 through 3.4 illustrate USH 18/151 in Iowa and Dane Counties. These figures indicate a disagreeement between the construction butt joints (Contractor) and RP locations. Many of the construction joints were found between intersections, while the RP descriptors generally originated and terminated at intersections.

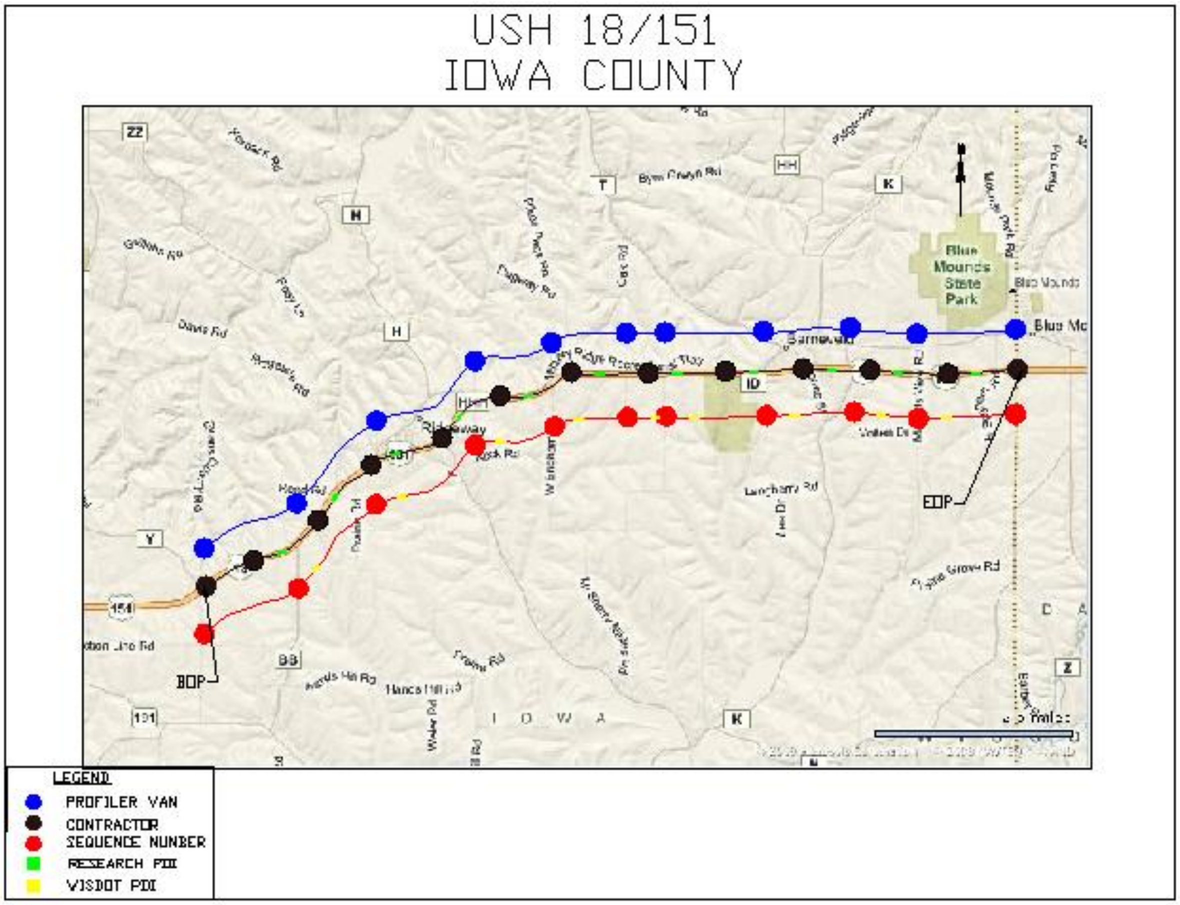

Figure 3.2 Test Sections for USH 18/151 in Iowa County

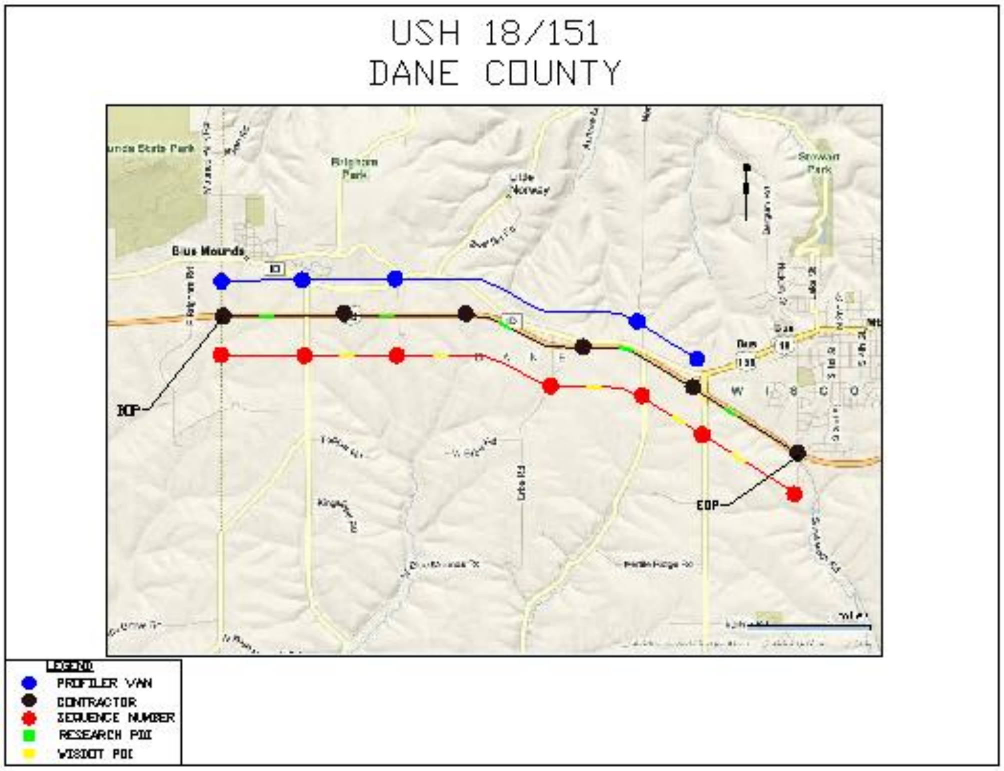

Figure 3.3 Test Sections for USH 18/151 in Dane County

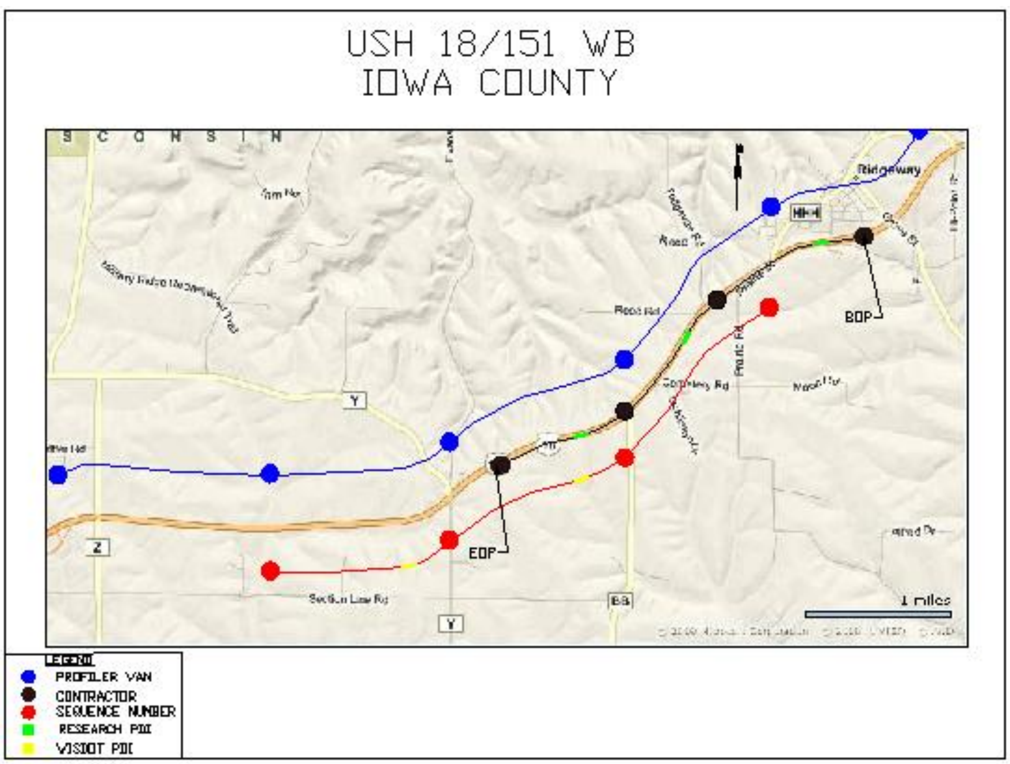

Figure 3.4 Test Sections for USH 18/151 Westbound in Iowa County

In Figure 3.5, the STH 29/32 project indicates a general agreement between the construction butt joints and the RP and Sequence Number termini. Figure 3.6 illustrates the USH 151 project in Columbia and Dane Counties.

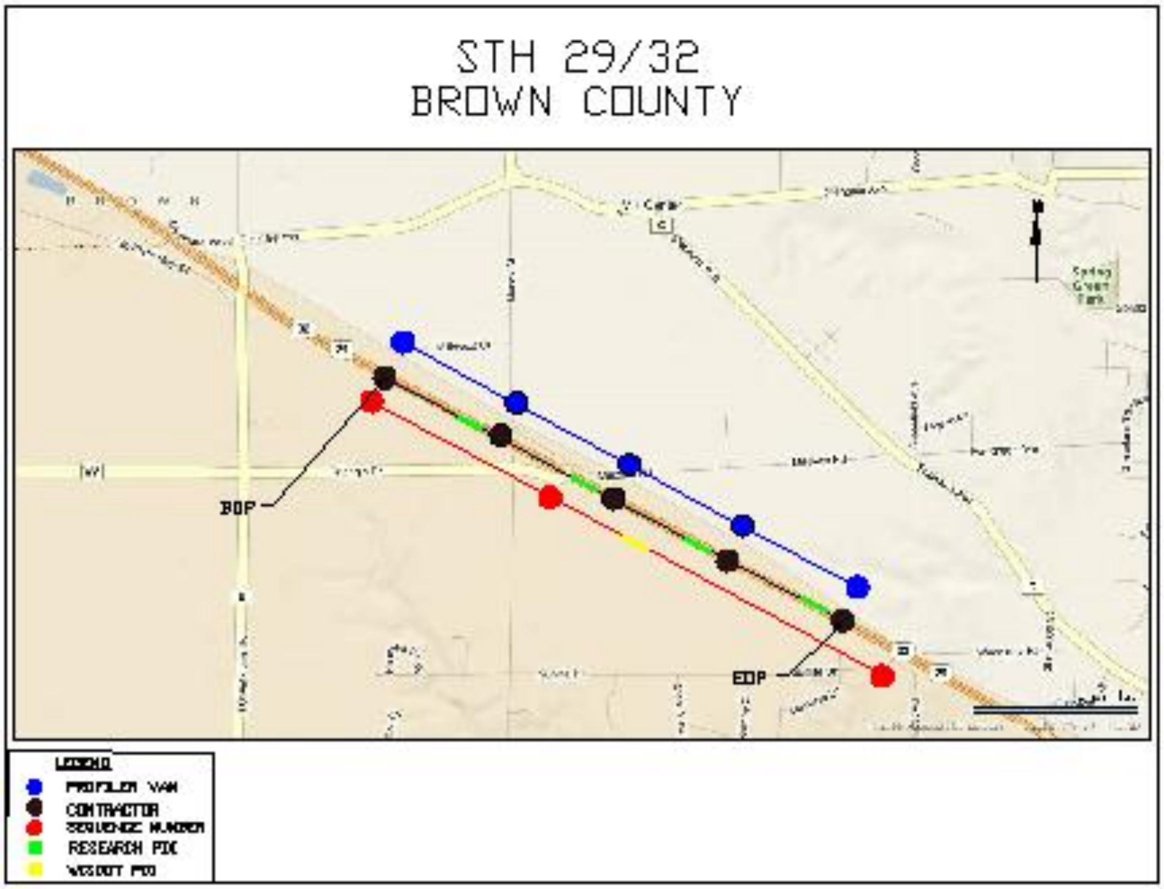

Figure 3.5 Test Sections on STH 29/32 in Brown County

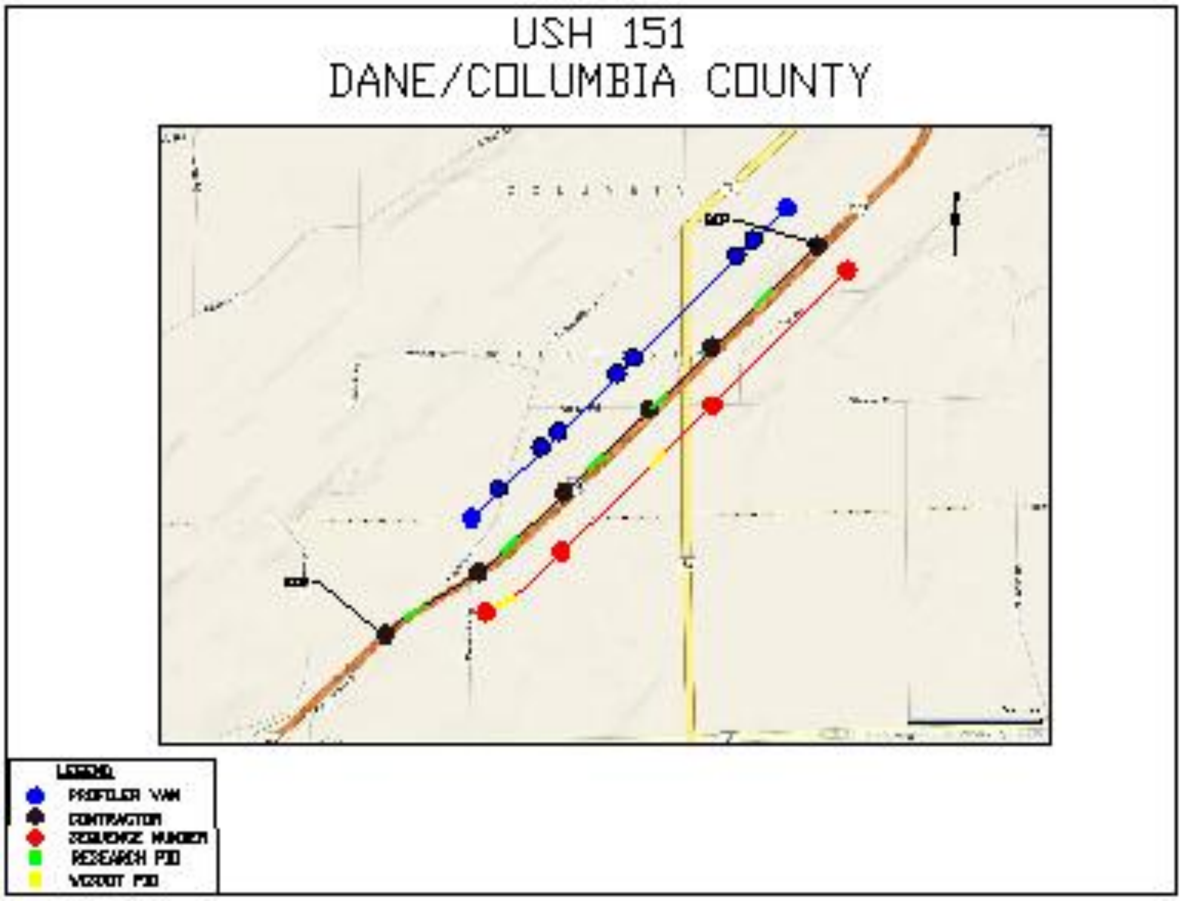

Figure 3.6 Test Sections for USH 151 Dane and Columbia Counties

These overlay maps indicated that the existing RP and Sequence Number system did not align with the physical test sections. In an attempt to overcome the disagreement, a series of tables were used to adjust the RP with the actual as-built sections. The Research PDI location was treated as the contolling location for the adjustment, since these 0.1-mile segments were to be used to conduct FWD testing and water permeability testing. Later, during field testing, the Research PDI locations were adjusted for traffic control and safety concerns. Adjustment tables are provided in Appendix A.

CHAPTER 4 DATA COLLECTION

4.1 Introduction

Field data were collected in multiple steps as listed in Table 4.1. Pavement performance data were collected by both WisDOT and the research team. In addition, the reserach team collected field FWD and permeability test data. Traffic control was contracted and provided by the highway departments from Brown, Columbia, and Iowa Counties. All traffic control charges were included in the research budget. The 511 System for lane closures was implemented according the WisDOT policy. There were multiple types of data collected and assembled for data analysis, with following sections detailing features of each procedure.

Table 4.1 Field Test Procedures and Remarks

Test Sequence	Remarks
1. IRI	• Measured with WisDOT performance van. • Scheduled in advance of other field testing.
2. PDI	• Initially raw data were collected and measured with WisDOT performance van, but was omitted due to resource constraints and duplication of effort. • Manual field measurements were conducted by UW-Platteville faculty and engineering students.
3. FWD	• WisDOT Kuab 2-m Falling Weight Deflectometer. • Nine test locations per section. • Three sites per test section (joint, mid-panel, and corner).
4. Core	• One full-depth, 4-inch diameter core in each OGBC test section. • Top, mid-depth, and bottom temperature recorded to compensate for warping.
5. Drainage	• Measured using core hole and flow meter. • Flow rate and time from core hole to outlet recorded.
6. Patch	• Patched by county crews.

4.2 IRI and Pavement Condition Survey

Both automated and manual pavement condition surveys were conducted for each test section. First, a semi-automated electronic survey and IRI measurements were collected using the WisDOT performance van prior to FWD and permeability testing. This was intended to anticipate actual performance measures, provide a pilot data analysis, and make any necessary adjustments to field testing. Due to resource constraints and duplicate effort, it was decided that only IRI and electronic faulting data be furnished by WisDOT to the research team. PDI data would have required additional time and effort, and since a manual condition survey was planned by the research team, the semi-automated PDI data were omitted. Rutting and faulting measurements were retained.

Pavement condition was manually measured for PDI. Trained UW-Platteville civil engineering students collected the manual data under faculty supervision. Prior to project data collection, the students completed a course in pavement design and pavement rehabilitation at UW-Platteville, and conducted practice measurements on actual concrete segments. Figures 4.1 and 4.2 illustrate pavement condition measurement during the study.

Figure 4.1 Transverse Joint Fault Measurement on USH 18/151

Figure 4.2 Longitudinal Fault Measurement on USH 18/151

4.3 Falling Weight Deflectometer

Falling Weight Deflectometer (FWD) testing was performed using WisDOT's Kuab 2-m FWD. FWD testing was utilized to provided fundamental measures of transverse joint load transfer capacity, subgrade strength and uniformity of slab support. A traditional two-layer analysis procedure, using FWD deflections measured at various load levels, was used to determine the in-place stiffness of the constructed pavement (all layers combined) and the dynamic *k* value of the subgrade. Effective PCC slab thicknesses were estimated based on assumed pavement modulii values based on the PCC pavement deflection analysis procedure outlined in the 1998 supplement to the AASHTO guide. Namely, an AREA-based solution was employed to estimate the pavement's radius of relative stiffness, with corrections applied to account for in-place slab dimensions (length and width). Figure 4.3 illustrates FWD testing on the USH 18/151 project.

Figure 4.3 FWD Testing on USH 18/151 at Barneveld Interchange

Longitudinal test locations varied by project, and were at least several hundred feet from transitions in base type or PCC cross-section. Readings were recorded at the joint, mid-slab, and at the corner. A benefit of repeating this testing on the same segments was to measure the relative change in subgrade reaction and modulus over time; however, based upon a review of data from previous studies, little change was expected. It is commonly believed that load transfer values calculated from deflections measured when the FWD load plate is on the leave side of the joint tend to be lower than load transfer values calculated from deflections measured when the FWD load plate is on the approach side of the joint (Khazanovich and Gotlif 2003). The rationale for this belief is that support under the slab on the leave side of the joint is expected to be weaker, according to the classical description of the mechanism of pumping at transverse joints.

A key factor in back-calculating subgrade reaction with the FWD is pavement temperature. It has been well documented that curling and warping occur due to changes in PCC pavement temperature, thus, temperature was measured at the surface, at or near the middepth, and near the bottom of the PCC layer. Then, the readings were factored during the data analysis to screen their relative effect. Fortunately, the weather conditions were ideal for FWD testing on each of the three projects with morning overcast and little temperature change.

4.4 Permeability and Drainage

One of the key features of permeable PCC pavement base layers is their ability to quickly drain moisture away from the structural section. The ease with which water flows through the permeable base, commonly measured as the permeability or hydraulic conductivity, *k*, is a function of the gradation and density of the drainage layer. In-place permeability may be estimated by transfer functions or quantified by direct measurements of infiltration capacity. For this study, direct field measurements were made to establish the infiltration capacity of the drainage layers. This value was then used to estimate the in-place permeability value, *k*, using direct measures of flow gradients and an assumed geometry of the subsurface flow (see Figures 4.4 and 4.5). Because the calculated *k* can be changed significantly with variations in the assumed width of the flow plume (Figure 4.5), this approach is better used as a relative indicator of the subsurface drainage capacity, i.e., as the in-place permeability increases the infiltration capacity should also increase. The overall quality of the drainage system may also be assessed by visual observations of outflow waters exiting the transverse drains. During steady-state infiltrations, a rapid and sustained outflow confirms the hydraulic continuity of a well-functioning drainage system. When no inflow or outflow occurs, on the other hand, this indicates a potential malfunctioning of the subdrainage system due to a clogged drainage layer and/or clogged longitudinal/transverse pipe systems.

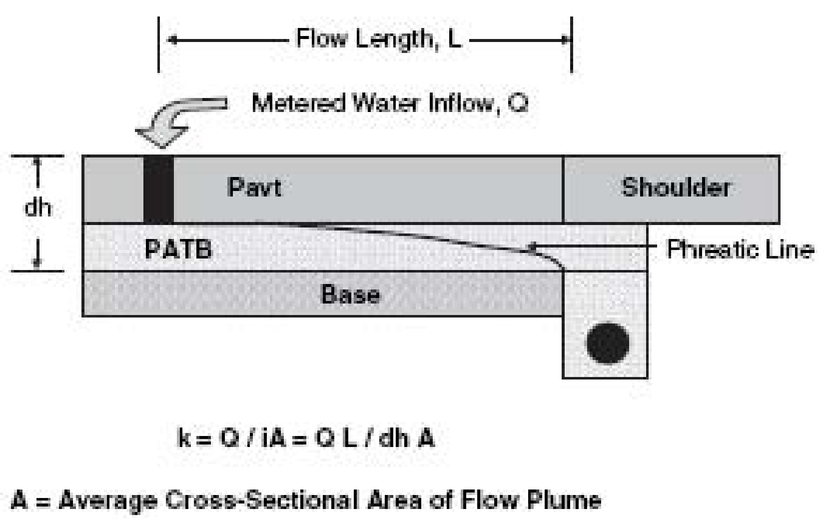

Figure 4.4 Measurement of In-Place Permeability

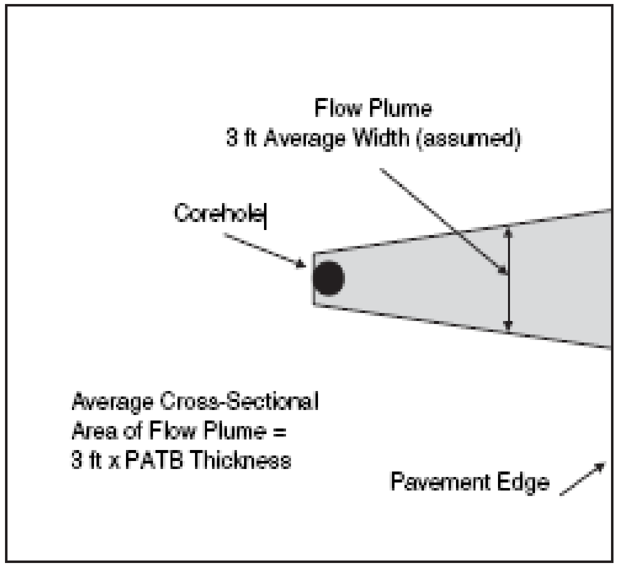

Figure 4.5 Illustration of plume of water from core hole to pavement edge

Permeability and drainage capacity were measured within each of the OGBC test sections. Full-depth cores were cut through the PCC pavement, then water from a county highway truck was pumped into the drainage layer to allow subsurface flow towards the longitudinal collector edgedrain pipe and ultimately out the transverse edgedrain outlet. (FWD testing preceded water permeability testing so that the flow of water in the pavement base did not alter the FWD reading). A flow meter measured and displayed the rate of water inflow, in gallons per minute, and the total volume of infiltrated water, in gallons. Inflow rates ranged from 0 to 8 gallons per minute. The permeability of the drainage layer, in feet per day, was then estimated using standard permeability and flow calculations. Target permeability from the FDM is recommended at 1,000 feet per day (WisDOT 2008). In several test sections, no water flow was observed after 100 gallons of water were added to the corehole. If water was observed at the outlet, tracer dye was added at the corehole to then measure the time to flow through the drainage system. Figures 4.6 through 4.12 capture permeability testing.

Figure 4.6 Drilling Corehole and Recording Elevations (USH 18/151)

Figure 4.7 Water Inflow at Corehole and Recording Rate and Total Volume (USH 18/151)

Figure 4.8 Water Flow Meter (USH 18/151)

Figure 4.9 Recording Elevation at Outflow Drainpipe (USH 18/151)

Figure 4.10 Flow Meter measuring 97.94 Gallons of Water into Corehole (STH 29)

Figure 4.11 Water flowing out Edgedrain Pipe and Apron Endwall (STH 29)

Figure 4.12 Water with Tracer Dye flowing out Edgedrain Pipe and Apron Endwall (STH 29)

CHAPTER 5 PERFORMANCE ANALYSIS

5.1 Introduction

A comprehensive analysis was conducted on the collected data to understand relationships among key variables in PCC pavement performance and to provide guidelines for modifications to pavement design inputs. The data analysis focused on understanding responses in pavement performance to design variables.

5.2 Methodology

Traditionally, field survey data are used to compute the WisDOT Pavement Distress Index (PDI). Combined indices such as the PDI and Pavement Condition Index (PCI) have been widely used by state highway agencies to characterize pavement performance. There are, however, major concerns associated with the use of such combined indices to indicate performance. These problems have been outlined by Paterson (1987) and include:

a) Different types of maintenance are appropriate for different levels of each distress type.

b) The relative seriousness of different defects varies with the pavement type, environment, the rate of deterioration and the maintenance program in place.

c) Each distress type evolves at different rates in different pavement types and under different traffic and environmental conditions.

The problems outlined by Paterson (1987) suggest that modeling the performance of PCC pavements using a combined index, such as the PDI, requires determining the average amount of distress effects from the many different combinations of distresses encountered. This method has the potential to yield results that have wide variances that, in turn, may suppress the very effects of interest. Thus, the analysis approach adopted in this study considered both the combined index (PDI) approach and a more versatile approach that evaluated major distress modes to better explain the relationship between distress progression and its influential factors. International Roughness Index (IRI) for the segment was evaluated as a single performance indicator.

5.3 Individual Distress Measures

The measurement of performance data included separate extent and severity values for each of the individual distresses observed. The extent provides information on the frequency of occurrence while the severity indicates the seriousness of the distress. Established WisDOT measures pertaining to PCC pavement distress include slab breakup, distressed joints and cracks, joint crack filling, patching, surface distresses, longitudinal joint distress and distortion, and transverse faulting. The length of evaluation segment for each test section was 0.1 mile (528 feet). Pavement area or location for the measured distresses, along with extent and severity values, are listed in Table 5.1.

Table 5.1 Distress Measures for PCC Pavement

Index (1)	Pavement Distress Indicator (2)	Pavement Area Measured (3)	Extent Levels (4)	Severity Levels (5)
1	Slab Breakup	Total pavement area.	0, 1, 2, 3, 4	0, 1, 2, 3
2	Joint Crack Filling	None defined.	None	None
3	Distress Joints/Cracks	Within 2 feet on either side of a joint or crack.	0, 1-2, 3-4, 5+	0, 1, 2,3
4	Patching	Total pavement area.	0, 1, 2, 3, 4	0, 1, 2, 3
5	Surface Distress	Total pavement area.	0, 1, 2	0, 1, 2
6	Longitudinal Joint Distress	Distress within 2 feet on either side of longitudinal joint.	0, 1	0, 1, 2, 3
7	Transverse Faulting	2 to 3 feet from both the outside and inside pavement edge.	0, 1, 2, 3	0, 1, 2, 3

Measured performance data from the three highway segments are shown in Tables 5.2 through 5.4. The third TIC section on USH 18/151 was omitted from the analysis since patching and distressed joints/cracks appeared to be the result of a removed right-side longitudinal pipe when a new eastbound entry ramp was constructed in 2004. Figure 5.1 illustrates the extent of patching in this TIC section adjacent to the new on-ramp.

Figure 5.1 Patching in TIC Section Adjacent to New On-Ramp

Table 5.2 Measured Distresses for Slab and Crack Filling

Highway (1)	Section (2)	Dowels (3)	Drainage (4)	Base (5)	Joints Sealed (6)	Slab Extent (7)	Slab Severity (8)	Crack Filling (9)
18/151	1	No	Drained	OGBC	Yes	3	1	0
18/151	2	No	Drained	OGBC	No	3	1	2
18/151	3	No	Drained	CSOG	Yes	1	1	0
18/151	4	No	Drained	CSOG	No	1	1	2
18/151	5	No	Drained	ASOG	Yes	0	0	0
18/151	6	No	Drained	ASOG	No	0	0	2
18/151	7A	No	Drained	TIC	No	0	0	2
18/151	7B	No	Drained	TIC	No	0	0	2
18/151	7C	No	Drained	TIC	No	1	1	2
18/151	8	No	Undrained	DGBC	Yes	1	1	0
18/151	9	No	Undrained	DGBC	No	1	1	2
18/151	10A	Yes	Drained	TIC	No	1	1	2
18/151	10B	Yes	Drained	TIC	No	2	1	2
18/151	11	Yes	Drained	CSOG	No	1	1	2
18/151	12	Yes	Drained	ASOG	No	0	0	2
18/151	13	Yes	Drained	OGBC	No	2	1	2
18/151	14	Yes	Undrained	DGBC	No	1	1	2
18/151	15	Yes	Undrained	DGBC	Yes	1	1	0
29/32	1	No	Drained	OGBC	No	4	1	2
29/32	2	No	Drained	OGBC	Yes	4	1	0
29/32	3	Yes	Drained	OGBC	Yes	1	1	0
29/32	4	Yes	Drained	OGBC	No	2	1	2
151	1	Yes	Drained	CSOG	No	0	0	2
151	2	Yes	Drained	OGBC	No	0	0	2
151	3	Yes	Drained	ASOG	No	0	0	2
151	4	Yes	Drained	OGBC	No	1	1	2
						1=10% 2=20% 3=30% 4=40%	0=none 1= 2 to 3 blocks	0=filled 1=need more 2=none

Table 5.3 Measured Distresses for Joints/Cracks, Patching, and Surface

Highway (1)	Section (2)	Distressed Joint/Crack Extent (3)	Distressed Joint/Crack Severity (4)	Patch Extent (5)	Patch Severity (6)	Surface Distress Extent (7)	Surface Distress Severity (8)	Long. Jt. Distress Extent (9)	Long. Jt. Distress Severity (10)
18/151	1	3	2	0	0	0	0	0	0
18/151	2	3	2	0	0	1	1	1	2
18/151	3	1	1	0	0	1	1	0	0
18/151	4	3	2	0	0	0	0	0	0
18/151	5	3	2	0	0	1	1	0	0
18/151	6	3	2	0	0	0	0	1	1
18/151	7A	3	1	0	0	1	1	0	0
18/151	7B	3	2	0	0	1	1	0	0
18/151	7C	3	3	3	2	1	1	0	0
18/151	8	3	2	0	0	0	0	0	0
18/151	9	3	2	0	0	0	0	0	0
18/151	10A	3	1	0	0	0	0	1	1
18/151	10B	3	1	0	0	0	0	1	1
18/151	11	3	1	1	1	1	1	1	1
18/151	12	3	2	0	0	0	0	0	0
18/151	13	3	1	0	0	1	1	1	2
18/151	14	3	1	1	1	1	1	1	1
18/151	15	3	1	0	0	1	1	1	1
29/32	1	3	1	0	0	1	1	1	1
29/32	2	3	2	0	0	1	1	1	1
29/32	3	3	1	0	0	0	0	1	1
29/32	4	3	1	0	0	0	0	1	1
151	1	3	1	0	0	0	0	0	0
151	2	3	1	0	0	0	0	0	0
151	3	3	1	0	0	0	0	0	0
151	4	3	1	0	0	1	1	0	0
		0=none 1=1 to 2 2=3 to 4 3=5+	0=none 1=slight 2=moderate 3=severe	0=none 1=1to3 2=4to6 3=7to9 4=10+	0=none 1=good 2=fair 3=poor	0=none 1=LT 10% 2=GT 10%	0=none 1=LT 1 in. 2=GT 1 in.	0=none 1=yes	0=none 1=slight 2=moderate 3=severe

Table 5.4 Measured Distresses for Faulting, PDI, and IRI

Highway (1)	Section (2)	Transverse Fault Extent (3)	Transverse Fault Severity (4)	PDI (5)	Faulting inches (6)	IRI Left WhP inch/mile (7)	IRI Right WhP inch/mile (8)	IRI Average inch/mile (9)
18/151	1	3	1	34	0.10	156	154	155
18/151	2	3	2	51	0.10	151	153	152
18/151	3	3	2	25	0.06	129	145	137
18/151	4	3	2	40	0.12	152	165	158
18/151	5	3	1	33	0.02	94	103	99
18/151	6	3	1	35	0.03	103	103	103
18/151	7A	3	1	28	0.08	100	119	109
18/151	7B	3	1	33	0.08	100	119	109
18/151	7C	3	2	76	0.08	100	119	109
18/151	8	3	2	40	0.10	115	136	125
18/151	9	3	2	40	0.14	119	140	130
18/151	10A	3	1	30	0.01	88	103	95
18/151	10B	3	1	30	0.01	88	103	95
18/151	11	3	1	36	0.00	91	103	97
18/151	12	3	1	32	0.01	84	120	102
18/151	13	3	1	39	0.01	100	100	100
18/151	14	3	1	36	0.01	118	152	135
18/151	15	3	1	32	0.01	110	128	119
29/32	1	3	1	39	0.15	135	158	147
29/32	2	3	1	55	0.16	137	180	159
29/32	3	2	1	32	0.02	84	104	94
29/32	4	2	1	33	0.02	107	122	114
151	1	0	0	23	0.01	112	106	109
151	2	0	0	23	0.00	88	92	90
151	3	1	1	26	0.01	134	135	134
151	4	0	0	26	0.01	124	114	119
		0=none 1=LT 1 per sta 2=1 to 2 per sta 3=GT 3 per sta	0=none 1= LT 1/4in 2= 1/4 - 1/2in 3= GT 1/2in					

5.4 Performance Analysis Plots

A series of plots were prepared to illustrate the relationship between pavement design features and resulting performance. The following sections break down the effect of key design features, including base type, dowels, and sealant upon pavement performance as measured by the composite PDI, individual distresses, and ride performance.

5.4.1 Base Type and Performance

USH 18/151 and USH 151 provided important data to compare the different base types and their effect on performance. STH 29 had the same base type throughout, an untreated open graded base course, thus it was unable to directly evaluate a monolithic base type.

The plot for doweled, unsealed JPCP in Figure 5.2 illustrates nearly the same performance among the different bases for doweled concrete pavement. Except for skewed transverse joints, the dense-graded base section (#14) is considered the standard, having a 6-inch thick dense-graded crushed aggregate base, and unsealed doweled transverse joints. Distresses common to all segments included slight to moderate distressed joints/cracks and slight transverse faulting. PDI scores were highest for open-graded base and lowest for the asphalt-stabilized open-graded (ASOG) base. ASOG had no slab breakup or surface distresses, however it measured a greater severity of distressed joints and cracks (moderate rating compared to others with a slight rating).

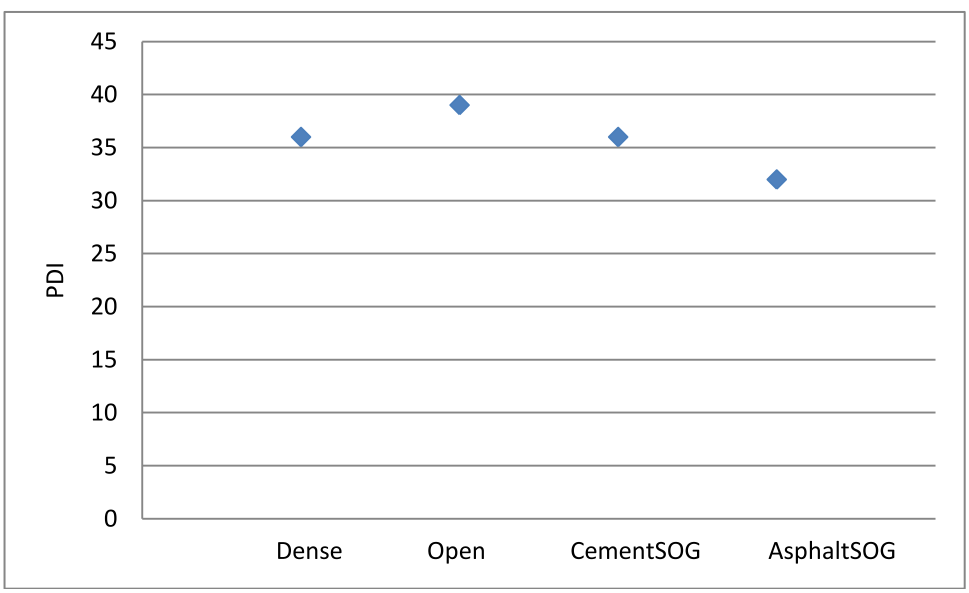

Figure 5.2 PDI variation for Unsealed Doweled Transverse Joints on USH 18/151

Figure 5.3 compares the IRI (inches per mile) with the same four sections, where values ranged from 97 to 135 inches per mile. Overall, this is considered good ride performance and within the normal range of what is expected for a rural PCC pavement nearing its first maintenance cycle. The dense-graded base section (#14) had the roughest ride when compared to all open-graded doweled sections. There was little difference among the open-graded sections, with a measured IRI approximately 100 inches per mile. Thus, based on the analysis of these two plots, asphalt-stabilized open graded base had the lowest measured distresses, while the open-graded bases had a lower surface roughness but higher measured distresses.

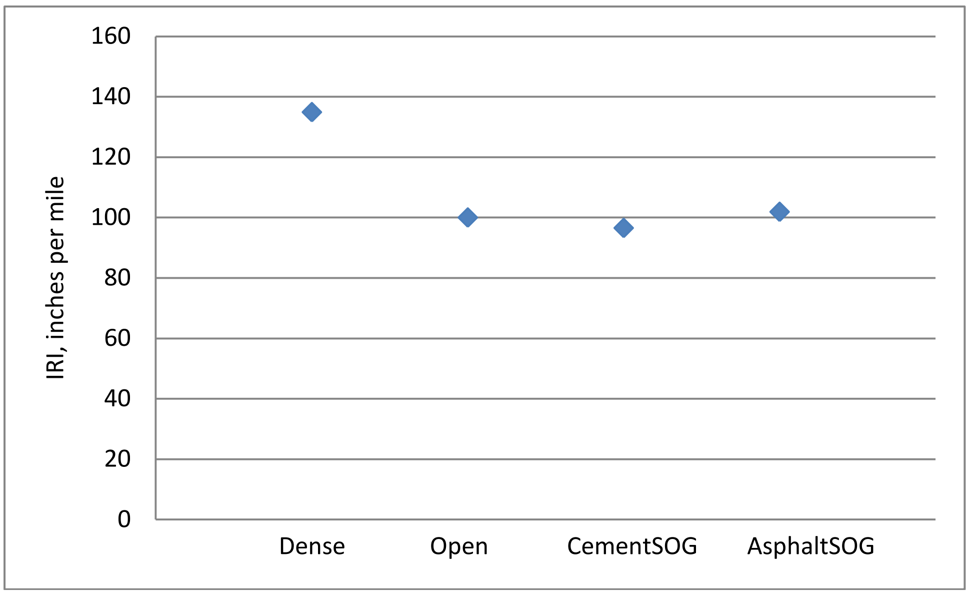

Figure 5.3 IRI variation for Unsealed Doweled Transverse Joints on USH 18/151

Similar plots were prepared for the test sections on USH 151, where all test sections were constructed with permeable base; no mainline dense-graded control sections were constructed for comparative purposes. Sections were doweled 10-inch thick PCC, unsealed skewed transverse joints, paved over a 4-inch upper permeable base and 4-inch lower dense base. Drainage pipe was 6-inch diameter, unlike USH 18/151 with 4-inch pipe diameter. Figure 5.4 illustrates nearly the same performance among the different bases, where PDI scores were very similar for the four sections, ranging from 23 to 26. Slight distressed joints/cracks were common to all segments. The higher PDI scores were attributed to the untreated OGBC base having 10% of slab area with slab breakup and surface distresses, and asphalt-stabilized OGBC having slight transverse faulting. However, faulting data from the WisDOT performance van measured 0.01 inches for all sections except New Jersey OGBC with a recorded value of zero.

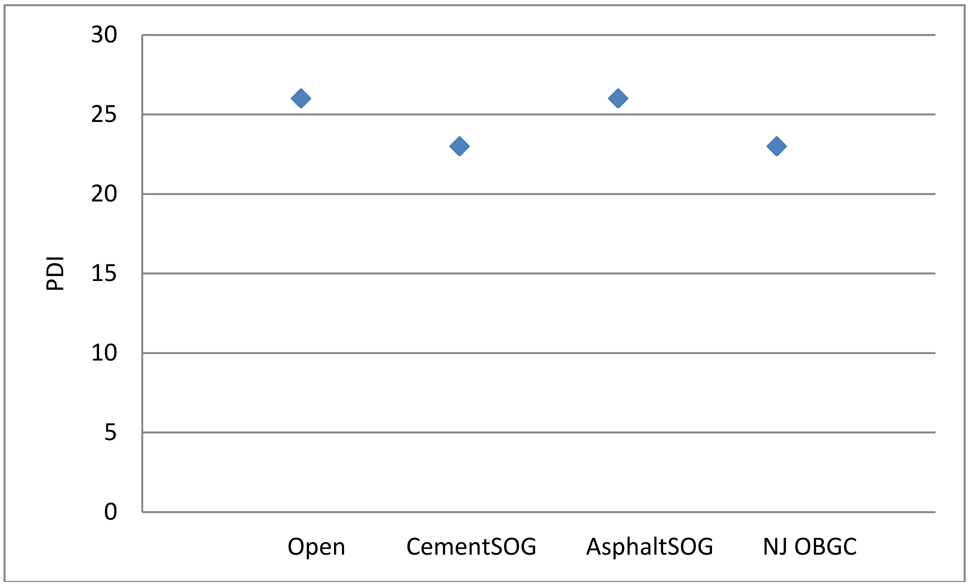

Figure 5.4 PDI variation for Unsealed Doweled Transverse Joints on USH 151

IRI was plotted for the same four sections in Figure 5.5. The New Jersey open-graded base had the smoothest ride when compared to other open-graded sections. ASOG base had the roughest ride, and unstabilized OGBC and CSOG bases had intermediate values. Overall, the finer-graded New Jersey base had less composite distresses and a smoother ride. The more coarse OGBC Gradation #1 is no longer specified for drainable PCC pavement bases.

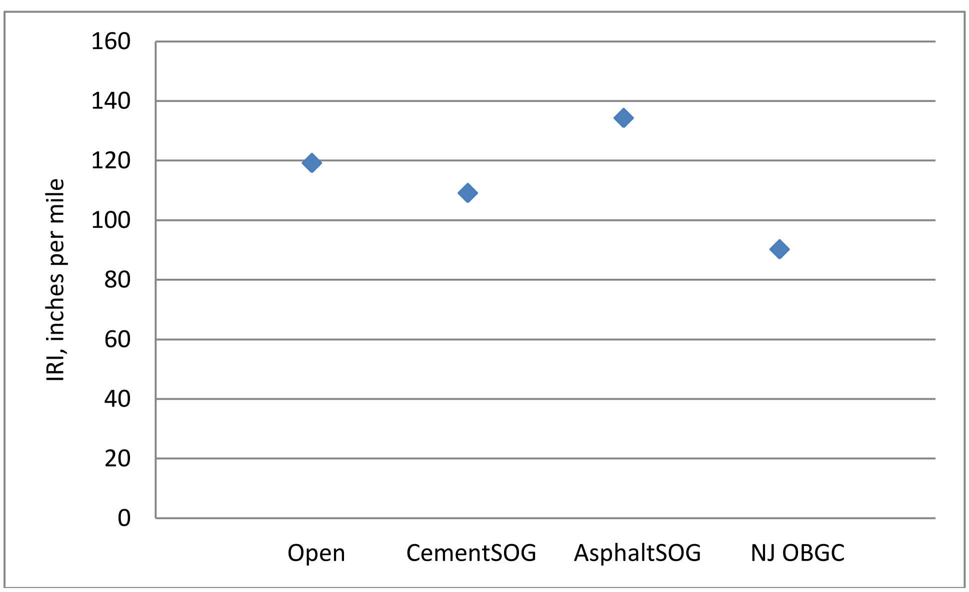

Figure 5.5 IRI variation for Unsealed Doweled Transverse Joints on USH 151

Of key interest in this study is the effect of base type on distress-based performance and ride quality. Removing doweled sections from the data set allowed a more direct evaluation of base type. Only USH 18/151 data were used in the evaluation since STH 29 had a monolithic base and all sections of USH 151 sections were doweled. Sealed and unsealed joints were stratified in the analysis since that is the only remaining design variable.

Figure 5.6 plots the PDI against base type on non-doweled USH 18/151 sections. When using the median value for the sealed and unsealed PDI scores, it can be concluded that cement-stabilized, asphalt-stabilized, and TIC drains had the least amount of distress. Dense-graded and unstabilized OGBC had the highest composite measure of pavement distress. Sealed joints produced a better performing pavement than unsealed joints. All TIC sections were unsealed.

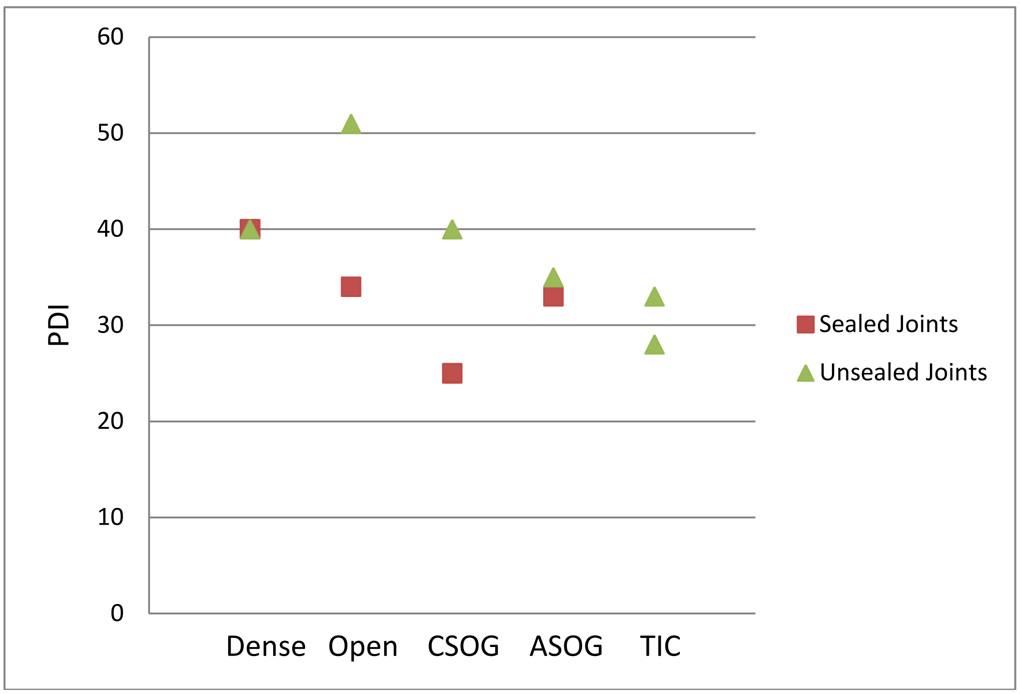

Figure 5.6 PDI variation for Base Type on Non-doweled Sections on USH 18/151

Similarly, ride was plotted against the same sections (see Figure 5.7). Asphalt-stabilized open base and TIC drains had the smoothest ride, while untreated and CSOG had the rougher ride. Sealant did not appear to have a consistent effect on ride. Based on this analysis of non-doweled sections, ASOG base and TIC drains had better distress-based performance and ride than the other non-doweled sections.

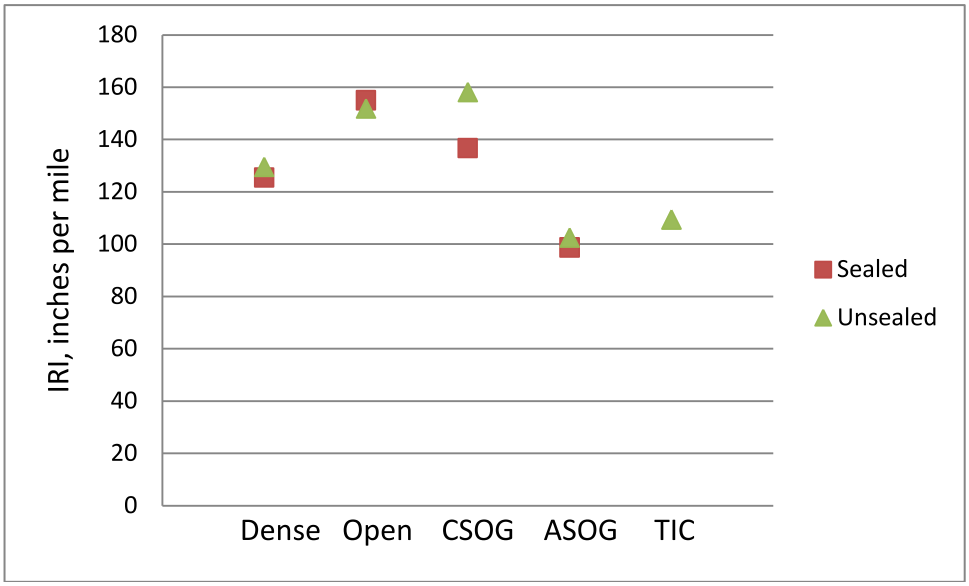

Figure 5.7 IRI variation for effect of Base Type on Non-doweled Sections on USH 18/151

5.4.2 Transverse Dowels and Performance

Since 1988, doweled JPCP pavements have been exclusively specified as a WisDOT PCC pavement standard. This study presented a valuable opportunity to understand their effect on performance and ride quality. Figure 5.8 combines the data from the three projects and plots the PDI against doweled and non-doweled pavement. This plot suggests that non-doweled pavement generally has a higher distress level than doweled; however, with the two highest non-doweled values removed, the difference is less pronounced.

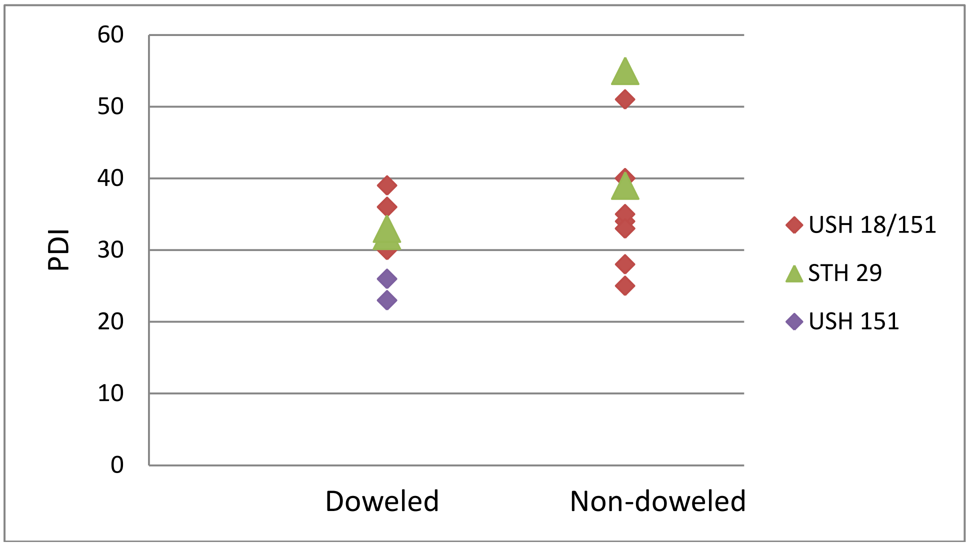

Figure 5.8 PDI variation with Transverse Joint Dowel Treament (all segments)

Since the composite PDI inherently limits the ability to assess individual distresses, they were reviewed individually to assess their contribution. As would be expected, transverse faulting impacted the PDI scores. The extent of transverse faulting was equal among all test sections; however, the severity was higher for non-doweled joints with about half of those sections rated a level 2 (¼ to ½ inch). Figure 5.9 illustrates this relationship using actual data from the WisDOT performance Van. Clearly, faulting was higher on the non-doweled sections, yielding values greater than or equal to 0.02 inches. All doweled sections were either at or less than 0.02 inches.

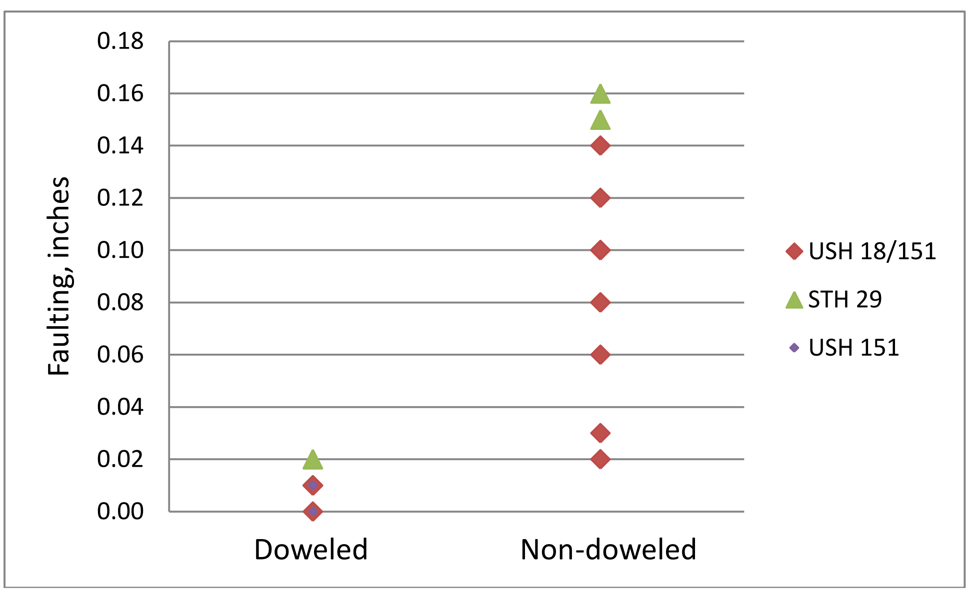

Figure 5.9 Faulting variation with Transverse Joint Dowel Treament (all segments)

Transverse faulting affects ride quality, and Figure 5.10 confirms this relationship where IRI was generally higher on non-doweled pavements, but many doweled sections had an equal roughness to non-doweled sections.

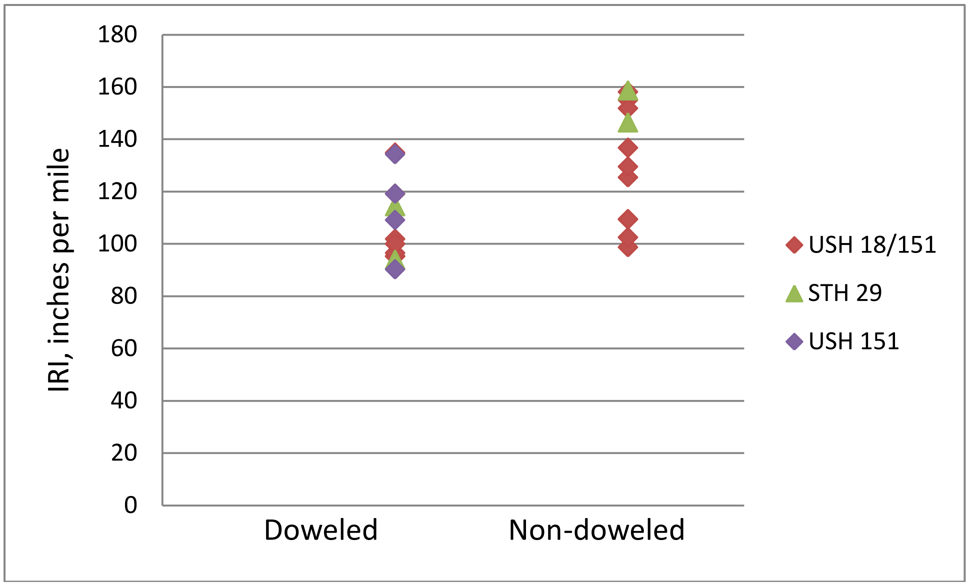

Figure 5.10 IRI variation with Transverse Joint Dowel Treament (all segments)

5.4.3 Sealant and Performance

The earlier plot in Figure 5.6 of PDI against base type on non-doweled USH 18/151 sections suggested that sealed joints produced a better performing pavement than unsealed joints. However, in Figure 5.7, sealant did not appear to have a consistent effect on ride across non-doweled sections.

The effect of sealant was further investigated by comparing adjacent sealed and unsealed sections on doweled USH 18/151 dense graded sections, and a combination of doweled and non-doweled sections on STH 29. First, the two sections on USH 18/151 were analyzed. These adjacent sections both had doweled JPCP with 6-inch thick dense graded base, with the only difference being Section #14 without sealant (WisDOT standard) and Section #15 with sealant. Traffic levels were assumed equal, however the 528-foot PDI segment was just beyond an exit ramp for STH 78 on the west side of Mount Horeb (Figure 5.11).

Figure 5.11 Location of USH 18/151 Test Section east of STH 78 Exit Ramp

Figure 5.12 illustrates the PDI for the two dense-graded doweled JPCP segments on USH 18/151 where the sealed section slightly outperformed the unsealed section. Both sections had identical extent and severity levels for slab breakup, distressed joints/cracks, surface distress, longitudinal distress, and transverse faulting; all were lowest severity rating, and all except distressed joints/cracks were lowest extent. The distress that produced the higher PDI for the unsealed section was patching, where 1 to 3 patches (extent=1) in good condition (severity=1) increased the PDI from 32 to 36. In Figure 5.13, the sealed section had a lower roughness than the unsealed section. Based on these two plots, sealed doweled joints yielded a better performing pavement than unsealed doweled joints.

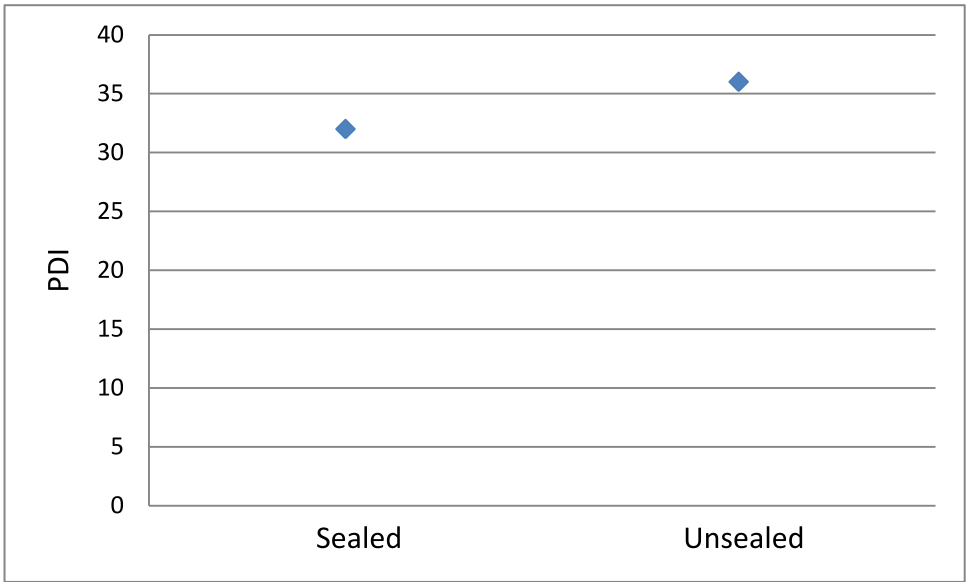

Figure 5.12 PDI for Sealed/Unsealed Doweled Joints on Dense Base (USH 18/151)

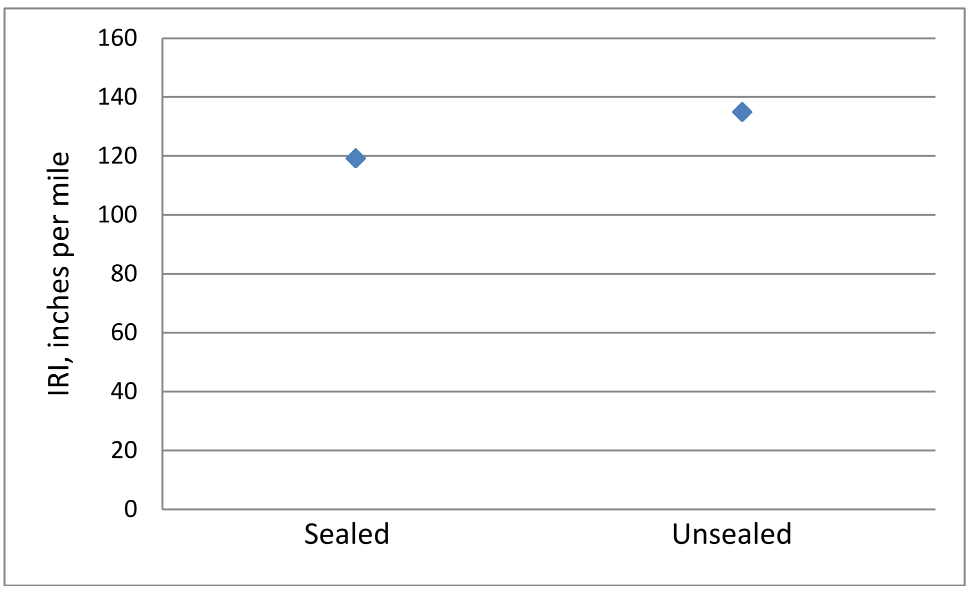

Figure 5.13 IRI for Sealed/Unsealed Doweled Joints on Dense Base (USH 18/151)

STH 29 had 10-inch thick PCC with a combination of both doweled/non-doweled joints and sealed/unsealed joints, and a uniform unstabilized open-graded base course. Figure 5.14 shows that the unsealed sections for doweled/non-doweled joints performed better than the median PDI for the sealed sections. This finding is contrary to USH 18/151. The sealed doweled pavement did perform a little better than the non-doweled section, but the opposite occurred on the non-doweled sections. Figure 5.15 for IRI performance found that sealed doweled joints had a smoother ride than the other combinations. Sealed/non-doweled joints produced the roughest ride, and as expected, non-doweled joints, whether sealed or unsealed, had the highest IRI values.

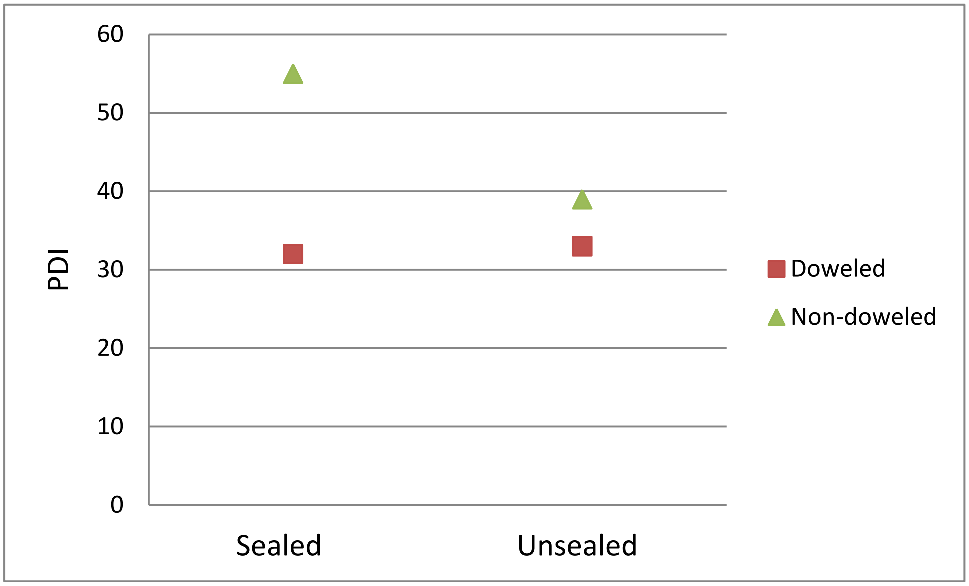

Figure 5.14 PDI for Sealed/Unsealed Doweled/Non-Doweled Joints on STH 29

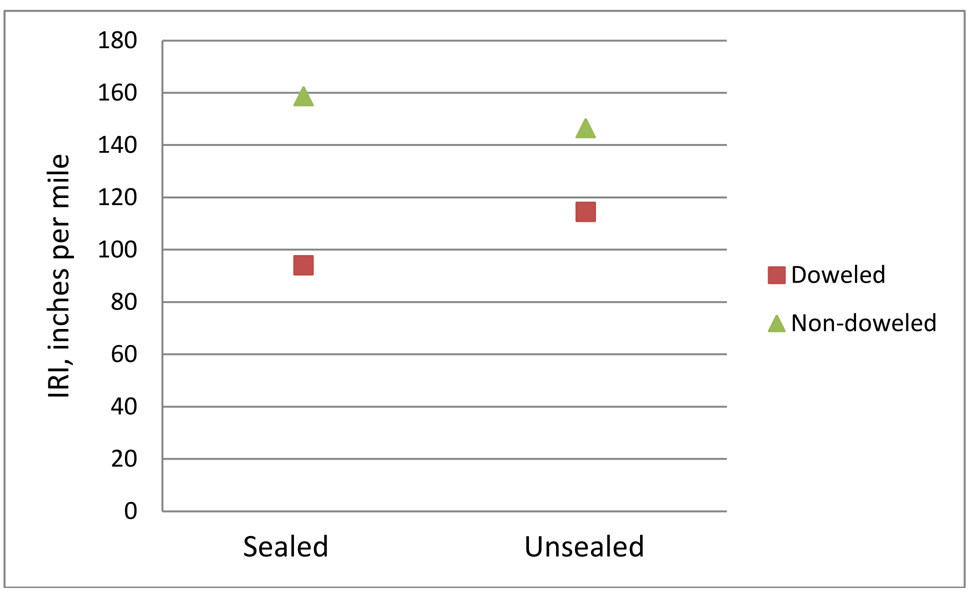

Figure 5.15 IRI for Sealed/Unsealed Doweled/Non-Doweled Joints on STH 29

5.5 Statistical Analysis

The statistical analysis consisted of two phases: a preliminary phase and a model-building phase. The former phase used basic statistics and correlations to identify key design input variables (base type, drainage type, etc.) having an effect on the extent, severity, and PDI of the major distresses predominantly observed. From a designer's point of view, the influential factors for the extent and severity can provide a basis for design modifications. For example, if a high frequency or severity level of distressed cracks is related to base type, an investigation will be warranted and proper design and materials recommended. The combination of the severity and extent is also needed for determining the type and level of maintenance work to be performed, and consequently, aid in the life-cycle cost analysis associated with specific maintenance alternatives. The visual analysis in the previous section were very useful; however, they may partially confound the relationships of primary variables.

5.5.1 Basic Statistics

Table 5.5 summarizes the mean, standard deviation, minimum and maximum for each rural JPCP variable. A total of 26 unique sections were tested, but only sample sizes of 25 are shown since the TIC section (#7C) adjacent to the new on ramp at Barneveld was dropped from the analysis. Excessive patching occurred where it appeared that longitudinal drains were removed during ramp construction.

Variables were coded for the statistical analysis to improve computing time with these designations:

- Dowel1Y0N: Dowels present, 1 = Yes, 0 = No.
- B1D2O3C4A5T: Base type, 1 = Dense, 2 = OGBC, 3 = CSOG, 4 = ASOG, 5 = TIC.
- Seal1Y0N: Sealant, 1 = Yes, 0 = No.
- Slbext through TransFaultsev: Performance distress measures for extent and severity designated with numeric condition code.
- PDI: Pavement Distress Index.
- IRIavg: average of IRI readings from left and right wheel path.
- Faulting: measured in one hundredths of an inch.

Table 5.5 Basic Summary Statistics

Variable	N	Mean	Std Dev	Minimum	Maximum
Dowel1Y0N	25	0.5200000	0.5099020	0	1.0000
B1D2O3C4A5T	25	2.8000000	1.3540064	1.0000000	5.0000
Seal1Y0N	25	0.2800000	0.4582576	0	1.0000
slbext	25	1.2000000	1.2247449	0	4.0000
slbsev	25	0.6800000	0.4760952	0	1.0000
crkfill	25	1.4400000	0.9165151	0	2.0000
distjtckext	25	2.9200000	0.4000000	1.0000000	3.0000
distjtcksev	25	1.4000000	0.5000000	1.0000000	2.0000
patchext	25	0.0800000	0.2768875	0	1.0000
patchsev	25	0.0800000	0.2768875	0	1.0000
surfdistext	25	0.4800000	0.5099020	0	1.0000
surfdistsev	25	0.4800000	0.5099020	0	1.0000
LJDistext	25	0.4800000	0.5099020	0	1.0000
LJDistsev	25	0.5600000	0.6506407	0	2.0000
TranFaultext	25	2.4800000	1.0456258	0	3.0000
TranFaultsev	25	1.0800000	0.5715476	0	2.0000
PDI	25	34.0400000	7.7216147	23.0000000	55.0000
IRIavg	25	119.4400000	22.3421873	90.0000000	159.0000
Faulting	25	0.0508000	0.0528299	0	0.1600

A correlation matrix was prepared for each combination of variables, with full output in Appendix C. Significant correlations are summarized in Figure 5.16. The correlation coefficient is a numerical measure that quantifies the strength of linear relationships, where coefficients at or near 1.000 indicate a strong relationship. Doweled joints affected distressed joints/cracks severity, transverse faulting extent/severity, PDI, IRI, and Faulting. This validated earlier findings from the plots, and as expected, there was a strong correlation with distressed joints, IRI, and faulting. Base type affected slab breakup extent/severity, PDI, and IRI. Sealed or unsealed joints had no effect on performance, which confirmed prior analysis where sealant had contradictory results on USH 18/151 and STH 29.

Pearson Correlation Coefficients, N = 25
Prob > |r| under H0: Rho=0

	Dowels	Base	Sealant
Slbext	-0.24019	-0.40202	0.19302
	0.2475	0.0464	0.3553
Slbsev	0.02746	-0.55587	0.23681
	0.8963	0.0039	0.2544
Crkfill	0.29244	0.30890	-1.00000
	0.1560	0.1330	<.0001
Distjtcksev	-0.68641	0.00000	0.21822
	0.0002	1.0000	0.2947
TranFaultext	-0.48765	0.10006	0.22957
	0.0134	0.6341	0.2696
TranFaultsev	-0.57761	-0.13999	0.22908
	0.0025	0.5045	0.2707
PDI	-0.47114	-0.35788	0.14978
	0.0174	0.0790	0.4748
IRI	-0.54759	-0.40880	0.21129
	0.0046	0.0425	0.3106
Faulting	-0.82040	-0.23649	0.19689
	<.0001	0.2551	0.3455

Figure 5.16 Significant Correlations between Design Variables and Performance

5.5.2 Statistical Models

Statistical models were developed to quantify the key relationship between design variables and performance. The primary modeling technique was Analysis of Variance (ANOVA). The ANOVA procedure first finds the mean of the data, then the function. The key objective was to understand what design variables provide a change in the mean PDI, individual numeric distresses, and IRI; the ANOVA output naturally provides this mean in the function. Equation 5.1 provides a general framework for the full model of variables.

Performance = Design + Construction + Traffic + Environment + Interactive Effects + Unexplained Variability or Error (5.1)

Performance was treated as the dependent variable, and measured by the IRI, composite PDI, and extent/severity of each pavement distress category. Since the sections have the same level of traffic and exposure to the environment, these factors were blocked and removed from the model.

Construction is a key factor in the performance of any pavement; however, construction records for these sections were not readily available. For practical purposes, the as-built properties were assumed homogenous across a highway segment and this variable was dropped from the model. This is not preferred since there may be some unique features of construction that may have an effect on performance, such as flexural strength and gradation. Equation 5.1 was reduced by dropping construction, traffic and environment effects to yield Equation 5.2:

$$\text{Performance} = \text{Design (Base, Dowels, Sealant)} + \text{Interactive Effects} + \text{Unexplained Variability or Error} \qquad (5.2)$$

The ANOVA procedure has the ability to test the significance of a variable when entered last into the model using Type III Sum of Squares, while regression computes the Sum of Squares in the specified model order using Type II Sum of Squares. Two standard statistics were calculated and used to determine significance: (1) F-value and (2) p-value. The F-value was calculated from the ratio of variances, then the probability level of significance, or p-value, was calculated. Equation 5.3 shows how the F-value for each distress indicator was calculated from the ratio of variability in each design variable (base type, dowels, sealant) to the unexplained variability (error). A p-value cutoff of 0.10 was specified since the "noise" in field data may inadvertently drop a marginally significant variable when the p-value is just beyond the traditional 0.05 cutoff value. In addition, small sample sizes have a more pronounced effect on significance tests.

$$F_{\text{Ext, Sev, or PDI}} = \frac{MS(\text{Input Variable})}{MS(\text{Error})} \qquad (5.3)$$

A full model of variables was initially tried, then only significant variables were retained. The TIC system was removed since it has not been adopted as a design standard, and a reduction from 5 to 4 base levels allowed an increase in the degrees of freedom and enhanced sensitivity in parameter significance. A total of 21 test sections were used to develop the models; USH 18/151 n=13 (15 sections minus 2 TIC), STH 29 n=4, and USH 151 n=4. Additionally, this allowed a pooling of a two-level project data set having 4 base types each (DGBC, OGBC, CSOG, and ASOG) from the USH 18/151 and USH 151 projects. STH 29 had only one level of base, untreated OGBC, and it was decided to retain this variable to pool with the OGBC sections on USH 18/151 and USH 151.

Table 5.6 summarizes the significant model relationships from initial investigation. The models largely reflect the significant linear correlation relationships. A notable observation is that sealant did not have significant relationship with any performance distress measure or the IRI, confirming earlier plots where there was disagreement between USH 18/151 (sealant has an effect) and STH 29 (sealant does not have an effect). It is clear that the transverse faulting (severity and extent) was affected by the presence of dowel bars.

Table 5.6 Significant Model Parameters from Initial Models

Performance Measure (1)	Base (2)	Dowels (3)	Sealant (4)	Interactions (5)
PDI	X			
Slab extent	X	X		Base*Dowel
Slab severity	X			
Dist. joints/cracks severity		X		
Transverse faulting extent		X		
Transverse faulting severity		X		
IRI		X		Base*Dowel

Based on the preliminary findings, final revised models were developed by dropping the insignificant variables. The final models had greater degrees of freedom and resulting model strength, and estimated parameters to quantify the effect on the dependent performance variable. Table 5.7 presents the final models, along with model accuracy as measured by the R-squared statistic. Key model parameters were dowels affecting PDI, slab extent and severity, distressed joints and cracks, transverse faulting extent and severity, and IRI. There were insufficient degrees of freedom to include interaction terms in parameter estimation.

The final set of models focused on doweled-only pavements, consistent with current WisDOT design standards and practice. A total of 11 test sections were used in model development; USH 18/151 n=5, STH 29 n=2, and USH 151 n=4. The dowel and sealant variables were dropped from the models to enhance the degrees of freedom as a tradeoff to a reduced sample size. Table 5.8 presents the models for the estimate of base type on performance. At 20 years of age, more slab breakup is expected with DGBC, OGBC, and CSOG.

Table 5.7 Model Parameter Estimates from Initial Models

Performance Measure (1)	Parameter Estimates (2)	R-squared (3)
PDI	30.727 + 8.473 (1 non-doweled, 0 otherwise) Base only was not significant. Base and dowels in the model determined that only dowels were significant.	27.8%
Slab extent	-0.537 + 1.074 (1 non-doweled, 0 otherwise) +1.000 (1 DGBC, 0 otherwise) +2.282 (1 OGBC, 0 otherwise) +0.750 (1 CSOG, 0 otherwise) +0.000 (1 ASOG, 0 otherwise)	68.2%
	0.0 + 1.000 (1 DGBC, 0 otherwise) + 2.222 (1 OGBC, 0 otherwise) + 0.750 (1 CSOG, 0 otherwise) + 0.000 (1 ASOG, 0 otherwise)	50.0%
	0.818 + 0.982 (1 non-doweled, 0 otherwise)	15.6%
Slab severity	0.0 + 1.000 (1 DGBC, 0 otherwise) + 0.889 (1 OGBC, 0 otherwise) + 0.750 (1 CSOG, 0 otherwise) + 0.000 (1 ASOG, 0 otherwise)	61.7%
Distressed joints cracks severity	1.091 + 0.709 (1 non-doweled, 0 otherwise)	51.2%
Transverse faulting extent	1.818 + 1.182 (1 non-doweled, 0 otherwise)	29.3%
Transverse faulting severity	0.727 + 0.773 (1 non-doweled, 0 otherwise)	40.0%
IRI(in/mile)	96.213 + 26.574 (1 non-doweled, 0 otherwise) + 17.750 (1 DGBC, 0 otherwise) + 17.532 (1 OGBC, 0 otherwise) + 15.750 (1 CSOG, 0 otherwise) + 0.000 (1 ASOG, 0 otherwise)	44.2%
	110.273 + 26.227 (1 non-doweled, 0 otherwise) Base only was not significant.	34.9%

Table 5.8 Doweled-only Model Parameter Estimates for Base Type

Performance Measure (1)	Parameter Estimates (2)	R-squared (3)
PDI	Base only was not significant.	---
Slab extent	0.0 + 1.000 (1 DGBC, 0 otherwise) + 1.200 (1 OGBC, 0 otherwise) + 0.500 (1 CSOG, 0 otherwise) + 0.000 (1 ASOG, 0 otherwise)	41.5%
Slab severity	0.0 + 1.000 (1 DGBC, 0 otherwise) + 0.800 (1 OGBC, 0 otherwise) + 0.500 (1 CSOG, 0 otherwise) + 0.000 (1 ASOG, 0 otherwise)	48.9%
Dist. joints/cracks severity	1.5 - 0.500 (1 DGBC, 0 otherwise) - 0.500 (1 OGBC, 0 otherwise) - 0.500 (1 CSOG, 0 otherwise) + 0.000 (1 ASOG, 0 otherwise)	45.0%
Transverse faulting extent	Base only was not significant.	---
Transverse faulting severity	Base only was not significant.	---
IRI	Base only was not significant.	---

There were only three significant models for doweled JPCP, those for slab breakup extent and severity, and the extent of distressed joints and cracks. Model parameters for PDI, transverse faulting, and IRI were insignificant. Important estimates for slab breakup extent compute an increase of 10% area for DGBC, increase of 12% area for OGBC, and an increase of 5% area for CSOG. Asphalt-stabilized open-graded base is the best choice for reducing the extent of slab breakup. With respect to severity, asphalt-stabilized base is again the ideal choice, and increases in slab breakup severity are expected by 1 for DGBC, 0.8 for OGBC, and 0.5 for CSOG. To limit the severity of distressed joints and cracks, three bases performed well including DGBC, OGBC, and CSOG. An increase of 1 (from slight to moderate) is estimated with ASOG. These are important model estimates since they can predict timing of maintenance and rehabilitation treatments in the life-cycle cost analysis.

5.6 Summary of Performance Analysis

5.6.1 Base Type

USH 18/151 and USH 151 had multiple base types, while STH 29 had one type, an untreated open graded base course. On USH 18/151, there was similar PDI-based performance for doweled unsealed pavement on dense and permeable base. Distresses common to all segments included slight to moderate distressed joints/cracks and slight transverse faulting. ASOG had no slab breakup or surface distresses, however it measured a greater severity of distressed joints and cracks. Statistical models estimate an increase in severity of distressed joints/cracks from slight to moderate with doweled ASOG. The DGBC section had the roughest ride when compared to all open-graded doweled sections. There was little difference in ride among the open-graded sections. In summary, doweled pavement on DGBC and ASOG bases had the lowest measured composite distresses, while the open-graded bases generally had a lower surface roughness.

For non-doweled sections on USH 18/151, the CSOG, ASOG, and TIC drains had the least amount of distress. DGBC and untreated OGBC had the highest composite measure of pavement distress. ASOG base and TIC drains had the smoothest ride, while untreated and CSOG had the rougher ride. In summary, non-doweled sections having ASOG permeable base and TIC drains had better performance and ride than the other non-doweled sections.

USH 151 had doweled 10-inch thick PCC, unsealed skewed transverse joints, paved over a 4-inch top permeable base (untreated with two gradations, CSOG, and ASOG) and 4-inch bottom dense base. All permeable base types had nearly the same performance among the different bases with slight distressed joints/cracks. Slight differences were untreated aggregate with 10% of slab area with slab breakup and surface distresses, and ASOG having slight transverse faulting. The finer New Jersey open-graded base had the smoothest ride when compared to other open-graded sections. ASOG had a rougher ride when compared to unstabilized OGBC and CSOG bases, which had intermediate values. In summary, the much finer-graded New Jersey base had less composite distresses and a smoother ride.

There were only three significant models for doweled JPCP, those for slab breakup extent and severity, and the extent of distressed joints and cracks. Model parameters for PDI, transverse faulting, and IRI were insignificant. Important estimates for slab breakup extent compute an increase of 10% area for DGBC, increase of 12% area for OGBC, and an increase of 5% area for CSOG. Asphalt-stabilized open-graded base is the best choice for reducing the extent of slab breakup. With respect to severity, asphalt-stabilized base is again the ideal choice, and increases in slab breakup severity are expected by 1 for DGBC, 0.8 for OGBC, and 0.5 for CSOG. To limit the severity of distressed joints and cracks, three bases performed well including DGBC, OGBC, and CSOG. An increase of 1 (from slight to moderate) is estimated with ASOG.

5.6.2 Transverse Dowels

Combined data from the three projects found that non-doweled pavement generally has a higher distress level than doweled; however, when two non-doweled outliers are removed, the difference is less pronounced. The extent of transverse faulting was equal among all test sections; however, the severity was higher for non-doweled joints with about half of those sections rated a level 2 (¼ to ½ inch). All doweled sections were either at or less than 0.02 inches. IRI was generally higher on non-doweled pavements, but many doweled sections had an equal roughness to non-doweled sections.

5.6.3 Sealant

USH 18/151 sealed non-doweled joints produced a better performing pavement than unsealed joints, however, sealant did not appear to have a consistent effect on ride. On two doweled dense-graded sections, sealant slightly outperformed the unsealed section, with minor patching as the prominent distress for the unsealed section. Both sections had identical extent and severity levels for slab breakup, distressed joints/cracks, surface distress, longitudinal distress, and transverse faulting.

STH 29 unsealed sections for doweled/non-doweled joints performed better than the median PDI for the sealed sections. The sealed doweled pavement did perform a little better than the non-doweled section, but the opposite occurred on the non-doweled sections. Sealed doweled joints had a smoother ride than the other combinations. Sealed/non-doweled joints produced the roughest ride, and as expected, non-doweled joints, whether sealed or unsealed, had the highest IRI values. Statistical models determined that sealant did not have an effect on overall performance and ride of doweled or non-doweled sections.

CHAPTER 6 STRUCTURAL ANALYSIS

6.1 Water Flow Field Testing Procedure

The procedure and the equipment for testing the rate of flow of water through the subsurface drainage system were developed under NCHRP Project 1-34D. The testing procedure required approximately two hours per test section, including pavement coring, elevation measurements, flow testing, and corehole patching.

Locating the transverse outlets: For each of the drained test sections, a typical transverse drainage outlet was located and marked for testing.

Measuring longitudinal grade: Locations for coring were selected based on the local longitudinal gradient of the pavement. The longitudinal grade was measured using a 2-foot carpenter's level with a digital display. The corehole location was then selected to ensure that subsurface flow though the permeable base layer would enter the longitudinal pipe system upstream of the located transverse outlet.

Coring: A 4-inch core was cut through the concrete surface down to the top of the permeable base layer. Coring was terminated as soon as the wash water was seen to be draining through the base layer.

Other measurements: A variety of distance and elevation measurements were collected for later use in calculating the length of the flow path and the hydraulic conductivity of the drainage layer. A measuring wheel was used to measure the transverse distance from the core hole to the edge of the pavement, the longitudinal distance from the core hole to the drainage outlet, and the transverse distance from the edge of the pavement to the transverse drainage outlet. A rod and laser level were used to obtain elevation readings at the surface of the pavement next to the corehole, the surface of the drainage layer within the corehole, the surface of the pavement at the pavement edge at the corehole station, the surface of the pavement at the edge adjacent to the drainage outlet station, and the inside bottom edge of the drainage outlet pipe. (Note: elevation readings increase as the measuring rod lowers).

Measuring inflow and outflow: Water was run from a water truck provided by the county highway department, through a hose to a water pump, then through a flow meter, and then into the corehole. The flow meter's screen can display either the total volume of water used, in gallons, or the rate of water flow, in gallons per minute. The water pump is powered by connection to a 12V car battery. Normally the tests were conducted by first adjusting the flow rate to the maximum that the drainage layer could accommodate without water spilling over the top of the corehole. The maximum inflow rate was recorded, and then the flow rate was reduced to a steady-state rate of 8 gallons per minute. The steady-state elevation of the free water level within the core hole was recorded. If the maximum inflow capacity of the drainage layer was less than 8 gallons per minute, the inflow rate was set to a value which maintained the water level in the core hole just below the pavement surface.

Clear water was allowed to flow into the base until it was observed flowing out of the downstream outlet, as illustrated in Figure 6.1. Once free-flow through the drainage system was established, a tracer dye was added to the inflowing water. A stopwatch was used to measure the time to when outflow was first observed, the time to when tracer dye outflow was observed, and the time when inflow was stopped. In many cases no water was observed flowing out of the downstream outlet after 20 minutes. In these cases, drainage testing was terminated and no tracer dye was introduced.

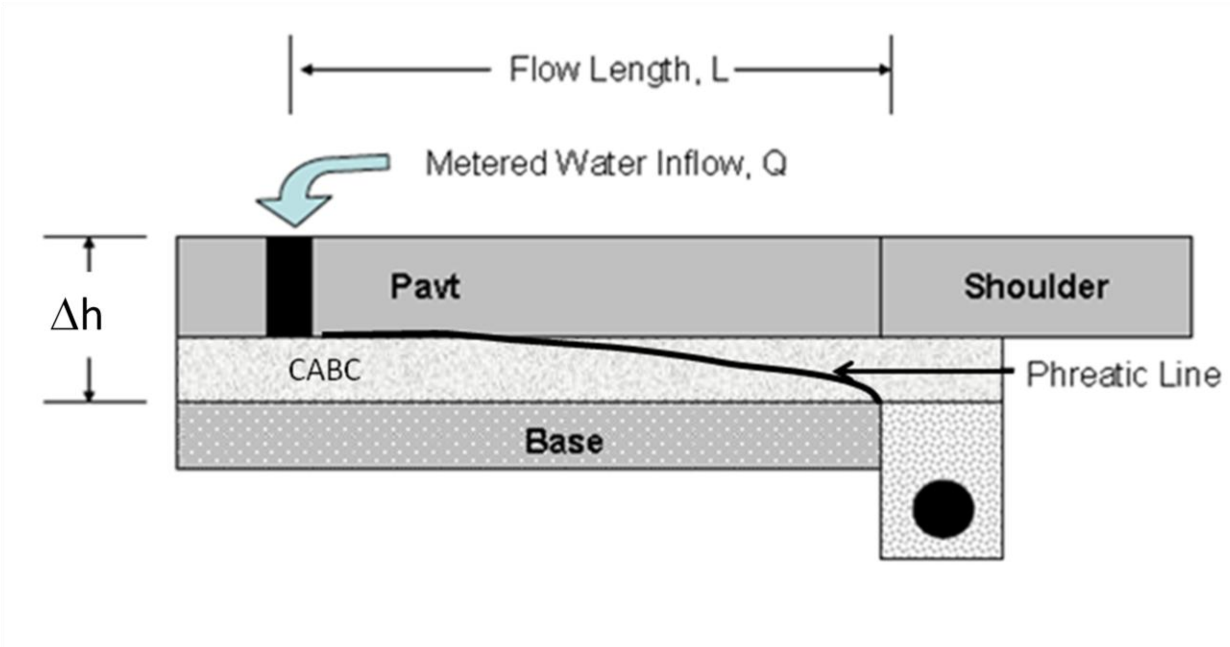

Figure 6.1 Flow Conditions During Permeability Testing

6.2 Drainage Flow Calculations

The following general equation is used to determine the rate of flow through a porous medium:

$$Q = K\, i\, A \qquad (6.1)$$

Where:

Q = rate of flow through cross sectional area, L^3 / t
K = hydraulic conductivity of medium, L / t
i = hydraulic gradient, L / L
A = cross sectional area, L^2

Equation 6.1 can be rearranged to solve for the hydraulic conductivity, K, of the open graded permeable base (OGPB) as a function of a known flow rate, hydraulic gradient, and cross-sectional area of flow:

$$K = 192.5\, Q / i\, A \qquad (6.2)$$

Where:

K = hydraulic conductivity of OGPB, feet/day
Q = maximum inflow rate measured during field tests (gal/min)
1 gal/min = 192.5 ft^3/day
i = hydraulic gradient measured in field = Δh / L
Δh = elevation head difference measured in field = (1 - 2) + 3 + 4 - 5, each defined below

1= elevation measure at top of pavement at edge, ft
2= elevation measure at top of pavement at corehole, ft
3= pavement thickness above OGPB, ft
4= thickness of OGPB, ft (assumed = 0.33 ft)
5= depth to top of free water surface, ft

L = flow length, ft = distance measured from corehole to pavement edge
A = cross-sectional area of flow, ft^2
= thickness of OGPB (ft) x assumed width of flow plume through OGPB (= 3ft)

Equations 6.1 and 6.2 are based on transverse flow (i.e., for a longitudinal grade of 0 percent). As the longitudinal grade increases above 0 percent, both the hydraulic gradient and the flow length increase. However, the proportional increase for both is the same, and thus Equation 6.2 can be considered valid for any longitudinal gradient provided the flow remains laminar. If turbulence is introduced due to an increased longitudinal gradient, the computed hydraulic conductivity can be considered as an equivalent *K*. In general, a minimum hydraulic conductivity of 1,000 fpd is required to provide adequate drainage. Target permeability from the FDM is recommended at 1,000 fpd (WisDOT 2008). Significantly higher values are indicative of highly permeable systems which can be considered as excellently drained but may lack stability unless adequately confined and/or stabilized.

An example of the hydraulic conductivity calculation sequence is detailed in Figure 6.2, using field measurements from the sealed, non-doweled, cement-stabilized open-graded test section within the USH 18/151 project location.

Date: 06/16/09
Site ID: USH18/151-3 CSOG
GPS Coordinates: N 43° 00.307', W 89° 58.200'
Cross Slope (%): 4.2
Longitudinal Grade (%): 1.1

Distance Measures, ft
Core to Edge: 4.6
Core to Outlet: 20.0
Edge to Outlet: 16.5

Elevation Readings, ft
Top of Pavement at Core: 2.52
Top of OGBC after Coring: 3.31
Top of Pavement at Edge: 2.69
Top of Pavement at Outlet: 2.79
Outlet: 5.81

Infiltration Measures
Maximum Inflow Rate (gal/min): 12
Steady State Infiltration Rate (gal/min): 8
Depth to Upstream Head (ft): 0.79
Time to First Outflow (min:sec): 14:42
Time to Tracer Input (min:sec): 14:42
Time to Tracer Outflow (min:sec) 20:28
Water Inflow Stopped: Stop @ 150 gal

Calculations from Measurements
Cross Slope (%): 3.7 [=(2.69-2.52)/4.6]
Longitudinal Grade (%): 0.5 [=(2.79-2.69)/20.0]
Thickness of pavement above OGPB (ft) 0.79 [=3.31-2.52]
Maximum Inflow, Q (ft^3/day): 2,310 [=192.5*12]
Head Difference, Δh (ft) 0.50 [=2.69-2.52+0.79+0.33-0.79]
Flow Length, L (ft): 4.6
Hydraulic Gradient, i (ft/ft): 0.11 [=0.50/1.6]
Cross Sectional Flow Area (ft^2): 0.99 [=0.33*3]
Hydraulic Conductivity, K (fpd): 21,212 [=2,310/(0.11*0.99)]

Figure 6.2 Hydraulic Conductivity Calculation Sequence

The result obtained appears reasonable since the hydraulic conductivity falls within the expected range for an OGPB; however, it should be noted that there is at least one limitation to this approach for calculating the in-place hydraulic conductivity of the OGPB. The actual value obtained for the hydraulic conductivity, *K*, is a function of the assumed width of the flow plume. For these calculations, a flow plume width of 3 feet is assumed for all of the outlets tested in this study. Because there is no way of knowing what the true flow plume width was for any particular core hole test, the calculated k values are best used as comparators for the different types (unstabilized, cement stabilized, asphalt stabilized) and gradations (open-graded No. 1, New Jersey) of permeable base materials tested. In other words, the computed *K* values are more meaningful as relative indicators of the capacity and functioning of the subdrainage system. When no inflow or outflow occurs, on the other hand, this indicates a malfunctioning of the subdrainage system, due to a clogged base layer, longitudinal pipe, and/or transverse outlet.

6.3 Field Permeability Testing Results

The hydraulic conductivity values calculated from all field measurements are summarized in Table 6.1. As shown, all calculated hydraulic conductivities computed for the USH 18/151 test section in Iowa-Dane Counties are in excess of 5,000 fpd, indicating very good to excellent drainage characteristics. The average hydraulic conductivity for the unstabilized permeable base (OGPB) is 17,481 fpd and there appears little variation due to doweling or joint sealant. The average hydraulic conductivity for the cement-stabilized permeable base (CSOG) is 15,129 fpd and there is a substantial variation due to joint sealant, with the sealed section having a hydraulic conductivity of 21,212 fpd and the unsealed sections averaging 12,087 fpd. The average hydraulic conductivity for the asphalt-stabilized permeable base (ASOG) is 8,471 fpd which is significantly lower than the OGPB and CSOG sections. There appears to be a slight variation due to doweling with the doweled section having a hydraulic conductivity of 5,920 fpd and the undoweled sections averaging 9,747 fpd.

The results provided for STH 29 Brown County indicate adequate drainage capacity in all sections. The data indicates a significant variation due to doweling but little variation due to joint sealant. The average hydraulic conductivity for the unstabilized permeable base (OGPB) sections without dowels is 2,817 fpd and 13,637 fpd for the doweled test sections.

The results provided for USH 151 Columbia-Dane Counties indicate adequate drainage capacity in only the cement stabilized permeable base section (CSOG), with a calculated hydraulic conductivity of 10,697 fpd. The base layers in the remaining three test sections would not accept water, indicating a complete blockage of the layer. The exact reason for this condition is unknown, however, the source aggregate for this project has a history of degradation concerns that may have caused an increase in fine particles and reduction in the permeability rate.

Table 6.1 Summary of Calculated Hydraulic Conductivities

Hwy No.	County Name	Test Section	Dowels	Base Type	Joints Sealed	Hydraulic Conductivity fpd
USH 18/151	IOWA	1	N	OGPB	Y	17,949
USH 18/151	IOWA	2	N	OGPB	N	16,667
USH 18/151	IOWA	3	N	CSOG	Y	21,212
USH 18/151	IOWA	4	N	CSOG	N	10,802
USH 18/151	IOWA	5	N	ASOG	Y	10,234
USH 18/151	IOWA	6	N	ASOG	N	9,259
USH 18/151	DANE	11	Y	CSOG	N	13,371
USH 18/151	DANE	12	Y	ASOG	N	5,920
USH 18/151	DANE	13	Y	OGPB	N	17,828
STH29	BROWN	1	N	OGPB	N	3,241
STH29	BROWN	2	N	OGPB	Y	2,393
STH29	BROWN	3	Y	OGPB	Y	11,438
STH29	BROWN	4	Y	OGPB	N	15,837
USH 151	DANE	1	Y	CSOG	n/a	10,697
USH 151	COLUMBIA/DANE	2	Y	NJOG	n/a	n/a
USH 151	COLUMBIA	3	Y	ASOG	n/a	n/a
USH 151	COLUMBIA	4	Y	OGPB	n/a	n/a
Note: n.a. indicates water flow into base layer could not be initiated.						

6.4 Falling Weight Deflectometer (FWD) Analysis

Nondestructive deflection testing (NDT) using the WisDOT 2m-FWD was conducted to provide a measure of the structural response of the pavement systems to loads similar in magnitude and duration to moving truck loadings. Deflection testing at slab centers and transverse joints was conducted prior to drainage testing to provide an indication of the interior support and deflection load transfer. Deflection testing at wheelpath edge and corner locations was performed during drainage testing to provide an indication of the uniformity of slab support and deflection load transfer. Loads of approximately 9,000 and 20,000 lbf were used at all test locations.

6.4.1 Analysis of Interior Slab Deflections

The foundation k-value and slab properties were backcalculated from center slab and mid-slab transverse joint deflections using the following 7-step process which is applicable to highway pavements:

Step 1: The deflection basin AREA was computed from center slab deflections using the equation:

$$\text{AREA} = (6 / \delta_0)\,(\delta_0 + 2\delta_{12} + 2\delta_{24} + \delta_{36}) \qquad (6.3)$$

where: AREA = deflection basin AREA, in

δ_i = surface deflection measure at i inches from the load

Step 2: A first estimate of the dense-liquid radius of relative stiffness of the pavement system, $l_{k\text{-}est}$ is backcalculated using the equation:

$$l_{k\text{-}est} = \{\ln[(36\text{-AREA}) / 1812.279133] / -2.55934\}^{4.387009} \qquad (6.4)$$

The dense-liquid radius of relative stiffness is a combined term which incorporates slab and subgrade properties and is defined as:

$$l_k = [\ (E_c\ H_c^3) / (12\ (1\text{-}\mu_c^2)\ k)\ ]^{\ 0.25} \qquad (6.5)$$

where: E_c = elastic modulus of concrete slab, psi
H_c = thickness of concrete slab, in
μ_c = Poisson=s ratio of concrete slab (assumed = 0.15)
k = subgrade k-value, psi/in

Step 3: The effective dimensions of the test slab are computed as:

$$L_{eff} = L_{act} + \Sigma\ (\ L_{adj} * LT_\delta^{\ 2}\ ) \qquad (6.6)$$

$$W_{eff} = W_{act} + \Sigma\ (\ W_{adj} * LT_\delta^{\ 2}\ ) \qquad (6.7)$$

where: L_{eff}, W_{eff} = effective slab length or width, in
L_{act}, W_{act} = actual slab length or width, in
L_{adj}, W_{adj} = adjacent slab length or width, in
LT_δ = deflection load transfer across adjacent slab joint(s), decimal form
$LT_\delta = \delta_u / \delta_l$
δ_u = deflection of unloaded slab at 12 inches from the load plate, mils
δ_l = deflection of the loaded slab at the center of loading, mils

Step 4: Slab size correction factors are computed as:

$$CF_{lk\text{-}est} = 1 - 0.89434 \exp[\ -0.61662\ (L_{eff} / l_{k\text{-}est})^{\ 1.04831}\ ] \qquad (6.8)$$

$$CF_{\delta i} = 1 - 1.15085 \exp[\ -0.71878\ (W_{eff} / l_{k\text{-}est})^{\ 0.80151}\ ] \qquad (6.9)$$

where: $CF_{lk\text{-}est}$ = correction factor for estimated dense-liquid radius of relative stiffness
$CF_{\delta i}$ = correction factor for maximum center slab deflection

Step 5: Compute adjusted l_k and δ_i values by:

$$l_{k\text{-}adj} = l_{k\text{-}est} * CF_{lk\text{-}est} \qquad (6.10)$$

$$\delta_{i\text{-}adj} = \delta_i * CF_{di} \qquad (6.11)$$

Step 6: The interior slab dynamic k-value is backcalculated using the equation:

$$k_i = [1000\ P / (\delta_{i\text{-}adj}\ l_{k\text{-}adj}^2)]\ [0.1253 - 0.008\ a / l_{k\text{-}adj} - 0.028\ (a/l_{k\text{-}adj})^2] \qquad (6.12)$$

where: k_i = interior subgrade dynamic k-value, psi/in
P = applied load, lb
$\delta_{i\text{-}adj}$ = maximum adjusted center slab deflection, mils
$l_{k\text{-}adj}$ = adjusted dense-liquid radius of relative stiffness, in
a = radius of load, in

Step 7: The effective thickness of the concrete slab was estimated from previously backcalculated l_k and k values by a rearrangement of Eq. 6.5 as follows:

$$H_c = [\ 11.73\ l_{k\text{-}adj}^{4}\ k_i\ /\ E_c\ ]^{1/3} \qquad (6.13)$$

where: E_c = known or assumed PCC modulus, psi

Incremental analysis of deflection response was also conducted to provide a means of differentiating slab curling from poor foundation support. For those cases where the slab temperature gradient (top temperature - bottom temperature) is excessively positive and foundation support stiffness is high, the center of the slab may be lifted off the foundation. In these cases, the maximum deflection and the deflection basin AREA term increase, resulting in a reduced backcalculated foundation k-value. If, however, at least two of the load levels used during testing were sufficient to create maximum surface deflections exceeding the depth of curling-induced voids, incremental analysis should indicate an increased dynamic foundation k-value and a decreased effective slab thickness as compared to values backcalculated from individual load/deflection pairs.

For the purposes of this incremental analysis of interior deflections, the incremental maximum interior deflection and loading were computed as:

$$\delta_{inc} = \frac{\delta_{P1} - \delta_{P2}}{P2 - P1} \qquad (6.14)$$

$$P_{inc} = P2 - P1 \qquad (6.15)$$

where: δ_{inc} = incremental maximum interior deflection, inches
P_{inc} = incremental load, lb
δ_{P2} = maximum interior deflection at highest load level, inches
δ_{P1} = maximum interior deflection at second highest load level, inches
P2 = maximum load level, lb (approximately 17,000 lb)
P1 = second highest load level, lb (approximately 12,000 lb)

The incremental maximum interior deflection and loading were then used to compute the incremental dynamic K-value and the slab bending stiffness modulus using the following equations:

$$k_{i\text{-}inc} = (P_{inc}\ /\ \delta_{inc}\ l_{k\text{-}adj}^{2})\ (0.1253 - 0.008(a/l_{k\text{-}adj}) - 0.028\ (a/l_{k\text{-}adj})^{2}) \qquad (6.16)$$

$$D_k = l_{k\text{-}adj}^{4} * k_{i\text{-}inc} \qquad (6.17)$$

where: $k_{i\text{-}inc}$ = incremental interior dynamic K-value, psi/in
D_k = slab bending stiffness modulus, lb-in

The above processes were used to estimate slab and foundation properties for all sections included in the FWD testing program. Table 6.2 provides a summary of these results.

Table 6.2 Summary of Backcalculated Values from Interior Deflections

Hwy No	County	Test Sect	Dowels	Base Type	Joints Sealed	$k_{i\text{-}inc}$ pci	H_{eff} in
USH 18/151	IOWA	1	N	OGPB	Y	278	9.5
USH 18/151	IOWA	2	N	OGPB	N	328	9.4
USH 18/151	IOWA	3	N	CSOG	Y	335	10.2
USH 18/151	IOWA	4	N	CSOG	N	370	10.8
USH 18/151	IOWA	5	N	ASOG	Y	270	10.4
USH 18/151	IOWA	6	N	ASOG	N	461	10.3
USH 18/151	IOWA	7a	N	CABC-TIC	N	688	10.8
USH 18/151	IOWA	7b	N	CABC-TIC	N	359	10.4
USH 18/151	IOWA	7c	N	CABC-TIC	N	547	9.6
USH 18/151	IOWA	8	N	CABC	Y	772	9.7
USH 18/151	IOWA	9	N	CABC	N	284	10.8
USH 18/151	IOWA	10a	Y	CABC-TIC	N	742	9.9
USH 18/151	IOWA	10b	Y	CABC-TIC	N	385	11.3
USH 18/151	DANE	11	Y	CSOG	N	667	11.9
USH 18/151	DANE	12	Y	ASOG	N	372	11.2
USH 18/151	DANE	13	Y	OGPB	N	380	10.5
USH 18/151	DANE	14	Y	CABC	N	913	9.6
USH 18/151	DANE	15	Y	CABC	Y	367	10.3
STH29	BROWN	1	N	OGPB	N	245	10.0
STH29	BROWN	2	N	OGPB	Y	228	11.6
STH29	BROWN	3	Y	OGPB	Y	212	10.6
STH29	BROWN	4	Y	OGPB	N	201	10.7
USH 151	DANE	1	Y	CSOG	N	270	13.8
USH 151	DANE	2	Y	NJOG	N	428	13.1
USH 151	COLUMBIA	3	Y	ASOG	N	378	10.8
USH 151	COLUMBIA	4	Y	OGPB	N	536	11.1

6.4.2 Analysis of Transverse Joint Deflections

The transverse joint deflections obtained at mid-panel and wheel path (WhP) locations were used to determine the normalized total joint deflection, dynamic edge and corner foundation support, and transverse edge and corner slab support ratios. The normalized total edge/WhP deflection is computed as the simple addition of unloaded and loaded slab deflections, normalized to a common load level of 9,000 lb, using the equation:

$$DT = \frac{9000 * (\delta_U + \delta_L)}{P} \qquad (6.18)$$

where: DT = total deflection, mils, normalized to 9-kip load
δ_U = unloaded slab deflection, mils (12 inches from the load center)
δ_L = loaded slab deflection, mils (at the center of loading)
P = applied load, lb

The normalized total edge/WhP deflection should remain relatively constant regardless of available deflection load transfer, provided that slab thickness, elastic modulus, and foundation support remain constant. The total edge/WhP deflection can be used as a relative indicator of the overall edge structural capacity of a test section as well as an input for the backcalculation of edge foundation support.

The edge foundation support was backcalculated based on the assumption that each test slab is of uniform thickness and elastic modulus, using the following equation:

$$k_e = \frac{D_K}{\left[0.82\, a + \sqrt{\dfrac{2.32\, DT_e\, D_K}{1000\, P}} \right]^4} \qquad (6.19)$$

where: k_e = transverse edge foundation k-value, psi/in
D_K = slab bending stiffness modulus, lb-in
a = radius of load plate, inches (= 5.9055 in)
DT_e = normalized total edge deflection, mils
P = normalized load value (= 9,000 lb)

Incremental analysis of transverse edge deflection response was conducted to provide a means of differentiating slab curling from poor foundation support. For the purposes of this incremental analysis of edge deflections, the incremental total edge deflection, normalized to a 9,000 lb load level, was computed as:

$$DT_{e\text{-}inc} = \frac{9000 * \left(DT_{e\text{-}P2} - DT_{e\text{-}P1}\right)}{P2 - P1} \quad (6.20)$$

where: $DT_{e\text{-}inc}$ = incremental normalized total edge deflection, mils
$DT_{e\text{-}P2}$ = total transverse edge deflection at maximum load level, mils
$DT_{e\text{-}P1}$ = total transverse edge deflection at second highest load level, mils
P2 = highest load level, lb (approximately 17,000 lb)
P1 = second-highest load level, lb (approximately 12,000 lb)

The incremental normalized total edge deflection was then used to compute the incremental transverse edge slab support using the equation:

$$k_{e-inc} = \frac{D_K}{\left[0.82\,a + \sqrt{\frac{2.32\ DT_{e-inc}\,D_K}{1000\,P}}\right]^4} \quad (6.21)$$

where: $k_{e\text{-}inc}$ = transverse edge incremental foundation k-value, psi/in
D_K = slab bending stiffness modulus, lb-in
a = radius of load plate, inches (= 5.9055 in)
$DT_{e\text{-}inc}$ = incremental normalized total edge deflection, mils
P = normalized load value (= 9,000 lb)

In those cases where temperature curling alone was responsible for poor support, incremental slab support should increase over that computed based on individual load levels, provided at least two load levels produced sufficient total edge deflection to close any curl-induced voids.

The uniformity of support under the transverse edge, termed the transverse edge slab support ratio, is computed as the ratio of backcalculated incremental edge to interior dynamic foundation k-values using the equation:

$$SSR_{et} = \frac{k_{e-inc}}{k_{i-inc}} \quad (6.22)$$

where: SSR_{et} = incremental transverse edge slab support ratio
$k_{e\text{-}inc}$ = incremental transverse edge foundation k-value, psi/in
$k_{i\text{-}inc}$ = incremental interior foundation k-value of the same test slab, psi/in

A similar approach was used to compute the corner support ratios based on the deflections obtained at the wheelpath locations. In general, incremental edge/corner slab support ratios less than approximately 0.75 are indicative of slabs with poor edge/corner support due to foundation densification/pumping and/or temperature curling. Tables 6.3 and

6.4 provide a summary of the results obtained from the edge and wheelpath locations, respectively, for the test sections included in the FWD program.

Table 6.3 Summary of Results from Transverse Edge Joint Deflections

Hwy No	Test Sect	Dowels	Base Type	Joints Sealed	LT_{δ} %	$\delta_{e\text{-}inc}$ mils@9k	$k_{e\text{-}inc}$ pci	SSRe
USH 18/151	1	N	OGPB	Y	35.7	8.13	317	1.16
USH 18/151	2	N	OGPB	N	39.3	6.59	469	1.45
USH 18/151	3	N	CSOG	Y	50.5	7.38	372	1.07
USH 18/151	4	N	CSOG	N	52.2	6.50	388	1.06
USH 18/151	5	N	ASOG	Y	65.0	6.89	368	1.42
USH 18/151	6	N	ASOG	N	29.2	6.43	565	1.17
USH 18/151	7a	N	CABC-TIC	N	10.4	5.84	594	0.86
USH 18/151	7b	N	CABC-TIC	N	32.8	7.14	338	0.99
USH 18/151	7c	N	CABC-TIC	N	67.9	5.94	546	1.07
USH 18/151	8	N	CABC	Y	29.2	4.89	760	0.97
USH 18/151	9	N	CABC	N	37.7	6.68	376	1.42
USH 18/151	10a	Y	CABC-TIC	N	94.6	6.62	437	0.58
USH 18/151	10b	Y	CABC-TIC	N	93.9	6.39	403	1.06
USH 18/151	11	Y	CSOG	N	92.2	4.78	587	0.83
USH 18/151	12	Y	ASOG	N	100.2	5.61	461	1.21
USH 18/151	13	Y	OGPB	N	94.7	10.29	203	0.54
USH 18/151	14	Y	CABC	N	92.4	6.80	462	0.67
USH 18/151	15	Y	CABC	Y	96.0	6.44	462	1.17
STH29	1	N	OGPB	N	17.1	11.36	181	0.81
STH29	2	N	OGPB	Y	18.6	10.37	159	0.73
STH29	3	Y	OGPB	Y	91.8	11.40	144	0.69
STH29	4	Y	OGPB	N	94.2	10.45	173	0.83
USH 151	1	Y	CSOG	N	98.2	7.33	173	0.66
USH 151	2	Y	NJOG	N	100.1	7.99	182	0.42
USH 151	3	Y	ASOG	N	96.1	10.30	178	0.45
USH 151	4	Y	OGPB	N	98.7	10.62	291	0.49

Table 6.4 Summary of Results from Transverse Wheel Path Joint Deflections

Hwy No	Test Sect	Dowels	Base Type	Joints Sealed	$\delta_{c\text{-}inc}$ mils@9k	$k_{c\text{-}inc}$ pci	SSRc
USH 18/151	1	N	OGPB	Y	16.59	484	1.75
USH 18/151	2	N	OGPB	N	18.33	436	1.43
USH 18/151	3	N	CSOG	Y	19.64	360	1.07
USH 18/151	4	N	CSOG	N	14.93	473	1.34
USH 18/151	5	N	ASOG	Y	21.15	328	1.25
USH 18/151	6	N	ASOG	N	14.83	528	1.13
USH 18/151	7a	N	CABC-TIC	N	n/a	n/a	n/a
USH 18/151	7b	N	CABC-TIC	N	n/a	n/a	n/a
USH 18/151	7c	N	CABC-TIC	N	n/a	n/a	n/a
USH 18/151	8	N	CABC	Y	n/a	n/a	n/a
USH 18/151	9	N	CABC	N	n/a	n/a	n/a
USH 18/151	10a	Y	CABC-TIC	N	n/a	n/a	n/a
USH 18/151	10b	Y	CABC-TIC	N	n/a	n/a	n/a
USH 18/151	11	Y	CSOG	N	n/a	n/a	n/a
USH 18/151	12	Y	ASOG	N	n/a	n/a	n/a
USH 18/151	13	Y	OGPB	N	n/a	n/a	n/a
USH 18/151	14	Y	CABC	N	n/a	n/a	n/a
USH 18/151	15	Y	CABC	Y	n/a	n/a	n/a
STH29	1	N	OGPB	N	12.82	213	0.88
STH29	2	N	OGPB	Y	11.54	225	1.09
STH29	3	Y	OGPB	Y	16.28	134	0.63
STH29	4	Y	OGPB	N	12.12	225	1.08
USH 151	1	Y	CSOG	N	15.42	92	0.32
USH 151	2	Y	NJOG	N	18.17	66	0.16
USH 151	3	Y	ASOG	N	18.87	90	0.25
USH 151	4	Y	OGPB	N	20.05	126	0.22

The deflection load transfer results provided in Table 6.3 indicate expected high average values for the doweled sections and fair to poor values for the non-doweled sections. For USH 18/151, the overall average load transfer values for the doweled and undoweled sections were 94.8% and 40.9%, respectively. For the non-doweled sections, the overall average load transfer values for the sealed and unsealed sections were 45.1% and 38.5%, respectively. For the doweled sections, the overall average load transfer values for the sealed and unsealed sections were 96.0% and 94.7%, respectively. For STH 29, the overall average load transfer values for the doweled and non-doweled sections were 93.0% and 17.9%, respectively. Little variation was noted for the sealed and unsealed sections. For USH 151, the overall average load transfer value for the doweled sections was 98.3%.

The slab support ratios provided in Tables 6.3 and 6.4 indicate variable results based on base type, joint reinforcement and joint sealant. For USH 18/151 Iowa-Dane Counties, all corner support ratios indicate full support is maintained. The edge support ratios generally indicate full support is maintained with the exception of three doweled and unsealed sections; namely sections 10a (SSRe=0.58), 13 (SSRe=0.54) and 14 (SSRe=0.67). These reduced values (< 0.75) suggest support problems due to densification of the base layers which is not normally expected for doweled sections. For the STH 29 sections, reduced edge support is noted for undoweled section 2 (SSRe=0.73) and doweled section 3 (SSRe=0.69) and reduced corner support is noted for doweled section 3 (SSRc=0.63). While these values are near the trigger value of 0.75, indicating only minor loss of support, it is interesting to note that these are the sealed sections. The results from USH 151 Columbia-Dane Counties indicates support problems under all edges and corners, with SSR values ranging from a low of 0.16 to a high of 0.66.

6.5 Summary

The results of the permeability and FWD tests may provide insight into the performance of the various test sections. For the USH 18/151 Iowa/Dane County test sections, while all bases can be considered adequately drained (K > 1,000 fpd), there appears to be a substantial reduction in the flow capacity for the ASOG base (#12) when compared to the other permeable bases. This section, however, is performing well in comparison to others in terms of PDI and IRI values. The poor load transfer evident in all un-doweled sections has led to increased faulting in all but the ASOG sections 5 and 6 and increased roughness in all but the ASOG sections 5 and 6 and the TIC sections 7a – 7c. Poor slab support ratios in the doweled sections 10a, 13 and 14 has only led to increased roughness in the DGBC section (#14). The PDI for all sections is generally comparable with the exception of increased PDI values in TIC section 7a and OGBC section 2. As a whole, these results indicate the ASOG is providing the best overall performance.

The results from the STH 29 Brown County sections indicate reduced drainage capacity for the non-doweled OGBC base sections 1 and 2. These sections also exhibit comparably poor load transfer, increased faulting and increased roughness. The doweled and sealed section 3 exhibits reduced edge and corner support and increased PDI.

The results from USH 151 Columbia/Dane County sections indicate poor drainage capacity for all but the CSOG section and poor edge and corner support for all sections. However, the PDI values are similar for all sections and only the ASOG section 3 has increased roughness.

CHAPTER 7 ECONOMIC ANALYSIS

7.1 Introduction

Existing WisDOT design practice, as well as recommended changes to current practice, must be made in the context of costs. For that reason, an economic analysis was performed to (1) quantify costs of comparable sections for the various base types, (2) identify the stage or time in pavement life when maintenance and rehabilitation activities are performed, and (3) quantify a life-cycle cost analysis (LCCA) for comparable sections. To conduct this analysis, several tools were used to yield proposed design guidelines and maintenance policies. The WisDOT LCCA methodology was used as the analysis tool, and recent construction and maintenance cost data were collected and applied.

The WisPave Pavement Design and LCCA computer program is considered department policy (WisDOT 2008). The FDM also states that, in addition to the pavement surface type, pavement structures are classified as "drained" and "un-drained. The selection of a drained system should be based on need, and in an LCCA, a drained pavement structure should not be compared to an un-drained pavement structure (WisDOT 2009). Despite that policy, a comparison was necessary to quantify the costs associated with each system. A manual approach illustrated the transformation of all cost data over time using fundamental engineering economic methods.

In addition to these standard tools, the developed performance relationships from prior analyses were interfaced with the LCCA to reflect input levels of construction and maintenance activities with the observed performance level. In essence, the relationships were able to predict the age of a certain maintenance treatment based on the distress level. The following sections describe the LCCA process.

7.2 Roadway Cross Section

Cross-sections for the project test sections are shown in Appendix B. Since there were discrepancies in cross-section designs, standardized cross-sections for dense-graded and open-graded base courses were created using the primary features from the three projects, as illustrated in Figure 7.1.

Primary differences among projects included USH 18/151 having a 9-inch thick PCC pavement, while both STH 29 and USH 151 having a 10-inch thick PCC pavement. Based upon an investigation in the MEDPG software calibration project, a majority of drainable PCC pavements in the state are 10 inches thick. Thus, a 10-inch thick pavement was selected for LCCA; however, any pavement thickness must be designed for traffic loading and base support. PCC pavement width was specified at 26 feet, with two 12-foot driving lanes and an integral 2-foot shoulder. The composite shoulder consists of a 2-foot extension of the PCC plus 8-foot wide asphaltic concrete (AC) surface, where the outer slab of the roadway is 14 feet with the striped pavement edge marked 12 feet from the centerline to indicate the limits of the travel lane.

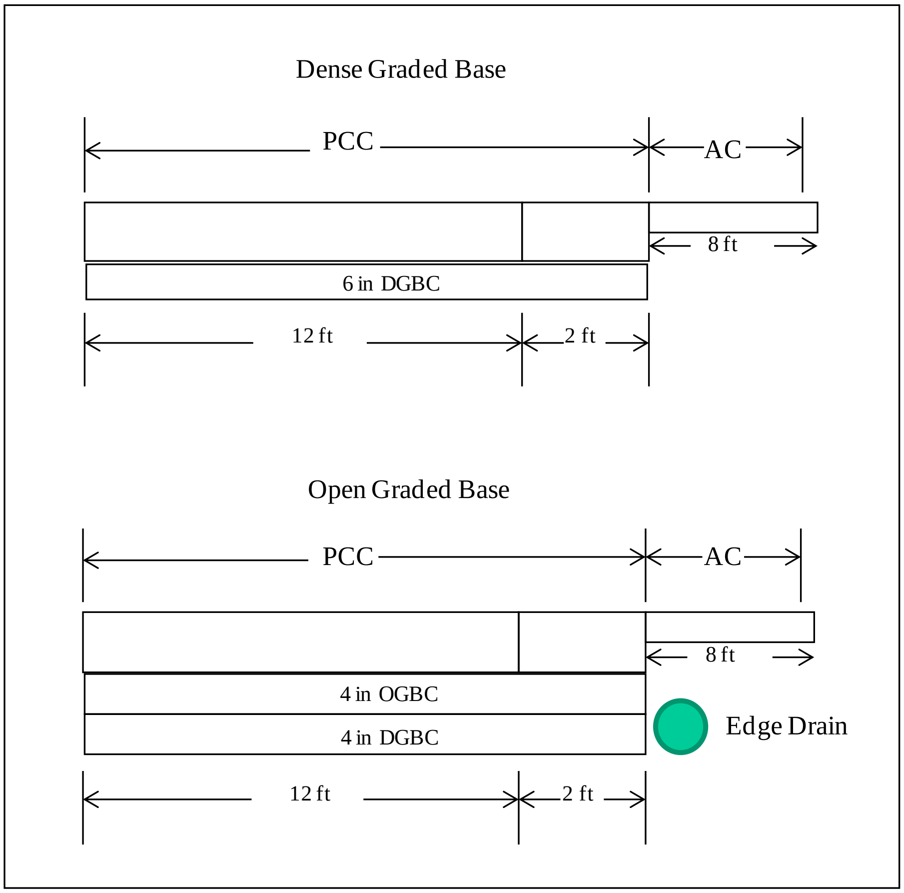

Figure 7.1 Pavement Cross Sections for LCCA

Drainage pipe diameter varied with 2-inch TIC and 4-inch perforated pipe underdrain installed on USH 18/151, and 6-inch perforated pipe on both STH 29 and USH 151. Since it was not possible to directly analyze the effectiveness of 4-inch and 6-inch pipe, and current design guidelines in the FDM specify a 6-inch diameter drainage pipe, a 6-inch size was adopted for the LCCA. TIC drainage with 2-inch diameter transverse pipe, that has not been adopted as a design standard, was not considered. Outfall pipe spacing varied among the three projects, from 100 feet to 400 feet. On the upper side of super-elevated curves, drainage pipe was omitted. Ramps and at-grade interchanges may create a change in spacing. The current FDM specifies a maximum 250-foot spacing, and since the spacing on the projects could not be directly evaluated, the 250-foot spacing was used in the LCCA. A tangent rural section having longitudinal perforated pipe on both edges was adopted.

Aggregate base designs vary for dense-graded and open-graded particle distribution. Current policy states that minimum thickness of the Base Aggregate Open Graded (BAOG) layer, when placed directly on subgrade, be 8 inches regardless of pavement type (WisDOT 2009). The particle size of the soil and BAOG must meet three filter criteria. If the filter criteria are not met, 6 inches of crushed aggregate base is required to protect the BAOG layer from contamination. A minimum thickness of 4 inches is required for the BAOG layer.

Based upon these requirements, and to provide a comparable section with dense and open graded bases, the following base aggregate sections were used in the LCCA: (1) dense graded base with 6 inches of crushed aggregate, and (2) open graded base with 4-inch lower layer of dense-graded crushed aggregate and 4-inch upper layer of open-graded crushed aggregate.

Details of the two alternatives are summarized in Table 7.1 for one direction of the 4-lane freeway. Material in the shoulders, including base aggregate and asphalt concrete, were included in the cost analysis since a substantial rehabilitation in the PCC pavement structure (i.e., partial depth patching, full depth patching, diamond grinding, etc.), may directly impact the shoulder. Shoulder base aggregate is not considered in the maintenance or rehabilitation. A minimum 6-inch base thickness was designed for the entire 38-foot paved roadway width, with shoulder base thickness of 12 to 13 inches to account for the 6 and 7 inch difference in PCC pavement (10 inches) and both the inner and outer AC shoulders (3 and 4 inches), respectively.

Table 7.1 Cross-Section Details for each Alternative

Cross-Section Element (1)	Alternative 1 JPCP with Dense Base (2)	Alternative 2 JPCP with Drainable Base (3)
Paved Roadway Width	38 ft (4ft + 12ft + 14ft + 8 ft)	38 ft (4ft + 12ft + 14ft + 8 ft)
Pavement Structure	26-ft wide Type-8 PCC, 10-inch thick	26-ft wide Type-8 PCC, 10-inch thick
Left Shoulder	4-ft wide AC, 3-inch thick 4-ft wide dense aggregate base, 13-in thick	4-ft wide AC, 3-inch thick 4-ft wide dense aggregate base, 13-in thick
Right Shoulder	2-ft Type-8 PCC 10-in thick (incl.) 8-ft wide AC, 4-in thick 8-ft wide dense aggregate base, 12-in thick	2-ft Type-8 PCC 10-in thick (incl.) 8-ft wide AC, 4-in thick 8-ft wide dense aggregate base, 12-in thick
Drainage Pipe	None	6-in perforated longitudinal pipe on both edges of slab. 6-in transverse pipe, every 250 feet, 14 feet long, both sides of slab. Apron end wall every 250 feet, both sides of slab.

7.2 Cost Data

The most recent construction and maintenance cost data were collected from WisDOT Average Unit Prices reported at the end of each fiscal year (WisDOT 2009). To achieve the objectives of the LCCA, the unit costs of all major pay items associated with PCC pavements over a life-cycle had to be collected. These bid prices include direct construction costs from the material, labor, and equipment, plus indirect costs from job overhead (temporary facilities, supervision, etc.), general and administrative expenses of the company (main office expenses, legal, etc.), bonds, and profit. Table 7.2 provides the relevant bid prices for the three most recent fiscal years.

Table 7.2 Input Cost Values for Life-Cycle Cost Analysis (Source: WisDOT 2009)

ITEM (1)	DESCRIPTION (2)	UNIT (3)	FY2007 (4)	FY2008 (5)	FY2009 (6)
305.011	BASE AGGREGATE DENSE 3/4-INCH	TON	$9.78	$10.89	$11.05
305.0115	BASE AGGREGATE DENSE 3/4-INCH	CY	$17.33	$18.96	$24.50
310.011	BASE AGGREGATE OPEN GRADED	TON	$10.28	$11.86	$13.73
310.0115	BASE AGGREGATE OPEN GRADED	CY	$43.78	$20.00	$36.05
315.01	ASPHALTIC BASE	TON	$26.90	$70.00	$47.52
415.006	CONCRETE PAVEMENT 6-INCH	SY	$25.45	$39.34	$29.68
415.007	CONCRETE PAVEMENT 7-INCH	SY	$19.05	$22.58	$26.06
415.0075	CONCRETE PAVEMENT 7 1/2-INCH	SY	$21.17	$22.50	$25.70
415.008	CONCRETE PAVEMENT 8-INCH	SY	$19.88	$25.64	$25.04
415.0085	CONCRETE PAVEMENT 8 1/2-INCH	SY	$23.78	$26.34	$28.01
415.009	CONCRETE PAVEMENT 9-INCH	SY	$21.55	$21.40	$29.96
415.0095	CONCRETE PAVEMENT 9 1/2-INCH	SY	$24.17	$30.24	$34.28
415.01	CONCRETE PAVEMENT 10-INCH	SY	$20.88	$24.45	$27.46
415.0105	CONCRETE PAVEMENT 10 1/2-INCH	SY	$23.25	$47.00	$26.29
415.012	CONCRETE PAVEMENT 12-INCH	SY	$35.70	$37.61	$39.99
416.071	CONCRETE PAVEMENT REPAIR	CY	$186.78	$200.95	$202.00
416.0905	CONCRETE PAVEMENT CONT. DIAMOND GRINDING	SY	$2.59	$3.05	$2.40
455.0105	ASPHALTIC MATERIAL PG 58-28	TON	$307.12	$386.93	$295.37
460.11	HMA PAVEMENT TYPE E-0.3	TON	$26.35	$36.82	$37.12
460.1103	HMA PAVEMENT TYPE E-3	TON	$25.64	$29.97	$39.43
490.01	SALVAGED ASPHALTIC PAVEMENT	SY	$1.29	$1.07	$1.00
490.0105	SALVAGED ASPHALTIC PAVEMENT	TON	$3.93	$4.26	$5.17
612.0104	PIPE UNDERDRAIN 4-INCH	LF	$8.88	$8.47	$7.10
612.0106	PIPE UNDERDRAIN 6-INCH	LF	$1.90	$4.79	$4.18
612.0204	PIPE UNDERDRAIN UNPERFORATED 4-INCH	LF	$11.10	$11.94	$8.69
612.0206	PIPE UNDERDRAIN UNPERFORATED 6-INCH	LF	$8.45	$8.56	$9.40
612.0404	PIPE UNDERDRAIN WRAPPED 4-INCH	LF	$10.00	$7.85	$5.52
612.0406	PIPE UNDERDRAIN WRAPPED 6-INCH	LF	$3.32	$6.38	$3.86
612.0806	APRON ENDWALLS FOR UNDERDRAIN REINF. CONC.	EACH	$110.27	$140.95	$132.99

An important stipulation in the LCCA is that the costs should take into account the quantity of materials, as well as the location and type of project being analyzed (WisDOT 2009). Given the limited test sections in this project, average unit prices were used in the analysis. A downside of using an average is not accounting for the effect of disproportionate units prices from small and large volume projects, and from the location of the project itself. Due to economy of scale, larger volume projects tend to have a lower unit price, while smaller volume projects have a higher unit price. Possibilities of unbalanced bids and different margins are also a factor.

In some cases, a lesser product size has a higher unit price because of material price variation, and effects on labor and equipment productivity. For example, a 7-inch concrete pavement may have a higher unit price than thicker mainline concrete pavements because of potential effects from reduced productivity placing concrete in a more confined space, and labor and equipment costs spread across a smaller volume and area of work. Additional factors are the number of projects used on an annual basis, the project size, raw material price fluctuations, cash or futures market pricing of commodities, special conditions in the contract, and regional cost differentials from labor agreements and material suppliers. Thus, average unit prices were more closely scrutinized by evaluating recent historic trends of average contract unit prices for aggregate base, PCC pavement, and drainage pipe for years 2007 through 2009. Unit cost variability has a direct impact on the results of an LCCA, thus, warranting an investigation.

Figure 7.2 illustrates the trends in aggregate bases (dense, open, and asphaltic) from 2007 to 2009. All untreated aggregate bases had a consistent upward trend, and it was reasonable to adopt the most recent 2009 price for LCCA. The 3/4–inch NMAS was adopted for dense-graded aggregate base consistent with current practice, and a smaller NMAS provides improved grade control for the pavement base, a feature critical for high-quality ride PCC pavement. Open-graded base specified in Section 310 is used on standard permeable base concrete pavements. The percentage passing the #4 sieve ranges from 15 to 45%. Wisconsin Gradation #1 was constructed on the USH 151 test sections in this study, where the percentage passing the #4 sieve was limited to 0 to 10%, yielding a theoretical permeability of 10,000 feet per day (Rutkowski 1998). USH 151 had an actual Gradation #1 passing the #4 sieve of 3%. One section constructed on USH 151 had New Jersey OGBC with a passing #4 of 49% (specification range of 40 to 55%) and theoretical permeability of 3,000 feet per day. Neither the Gradation #1 or New Jersey OGBC are currently specified, thus, the most reasonable approach for LCCA is to use the current Section 310 prices. Conversions used, consistent with WisDOT policy, were 2 tons/CY for dense-graded base and 1.75 tons/CY of open-graded base.

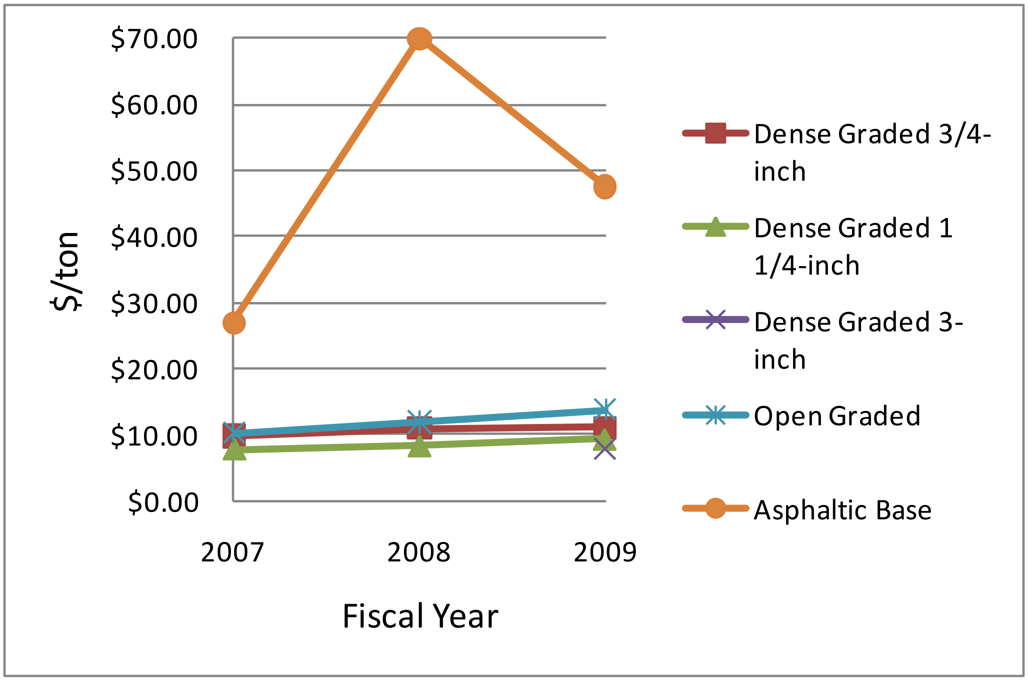

Figure 7.2 Unit Prices for Aggregate Base

Asphaltic base had a spike of $70/ton in 2008 followed by a drop to about $48/ton. The asphaltic base cost item includes furnishing and installing both the aggregate and asphalt cement, unlike the standard Section 460 E-series mainline or shoulder mixtures that pay separately for the furnished and installed pavement (minus the asphalt cement) plus the separate tonnage of asphalt cement used in the mixture. The purpose of the Section 315 asphaltic base is to provide the base support for overlaying with new pavement. The original test sections limited the passing #4 sieve to 10%, allowing water permeability. One-inch NMAS aggregate is specified and to be placed in 4-inch or thinner compacted layers. An E-3 Section 460 mixture (minus asphalt cement) averaged $39.42/ton in 2009, and with 4.6% asphalt cement at $300/ton (non-polymer modified) added to a typical coarse-graded mixture, the total furnished and installed price would be approximately $52/ton (95.6% x $40/ton + 4.6% x $300/ton). (E-3 was chosen since it is one of the higher tonnage asphaltic materials and would reduce the effect of quantity pricing). The E-3 price closely approximates the $47.52/ton asphaltic base price.

Cement-treated open-graded base, similar to that constructed on the test sections in this study, is not an active WisDOT bid item. Currently, Section 320 concrete is specified to provide a support base for pavement. This concrete base is constructed using Section 415 specifications, with modifications, and does not include an aggregate gradation and permeability thresholds consistent with the objectives of a permeable base. The cement-treated aggregate structure in the test sections was open graded allowing the flow of water. The cement-treated bases in this study can be considered “lean concrete”, where the test sections on USH 18/151 and USH 151 specified per cubic yard of mixture were: (1) 250 lbs of Type-I Portland cement equating to about a 3-bag mixture, and (2) 11 gallons of water (water-cement ratio of 0.36). The mixture was placed and roller compacted, then asphalt emulsion sprayed across the surface at a rate of 0.1 to 0.2 gallons/S.Y. There is not a single bid item available to assign to the cement-treated base, and a combination of costs would be necessary to create a prototype bid item. Based on this price uncertainty, it was decided to omit this base type for the LCCA, and focus the analysis on dense-graded aggregate base, untreated open graded base, and asphaltic-treated aggregate base.

A plot of $/SY for PCC pavement thickness by year was prepared to illustrate the cost trends (Figure 7.3) to assess whether thickness and year had a relative impact. Upward cost trends were observed for 7, 8½, 9, 9½, 10, and 12-inch thick PCC pavements, while 6, 8, and 10½-inch had a general decrease from 2008 to 2009. Overall, pavement costs have generally increased the past few years. As noted earlier, the 10-inch thick pavement was selected for the LCCA.

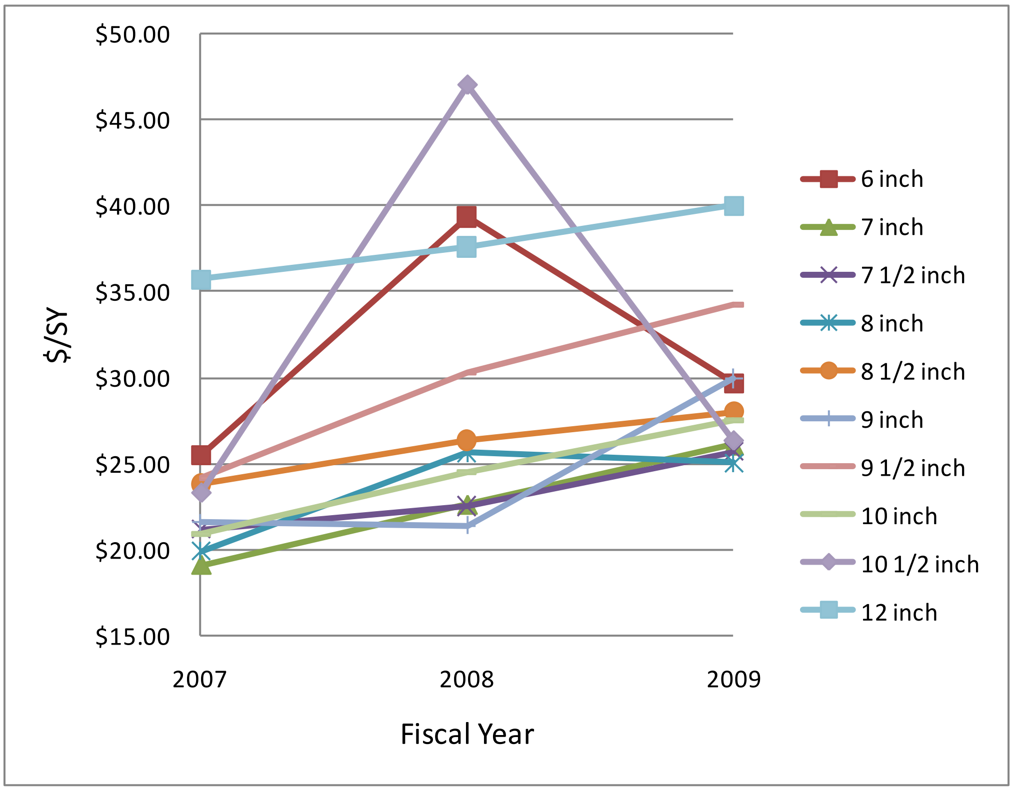

Figure 7.3 Unit Prices for Concrete Pavement Thickness

Drainage pipe unit prices were then analyzed by plotting the data in Figure 7.4. Both the 4-inch and 6-inch prices were plotted since a 4-inch perforated pipe underdrain was installed on USH 18/151, and 6-inch perforated pipe on STH 29 and USH 151. As mentioned earlier, the 6-inch pipe has been selected for LCCA. The prices show a general spike in 2008, for 4-inch unperforated and 6-inch wrapped, consistent with the spike in oil prices, since polyvinyl chloride drainage pipe (AASHTO M 278) is a petroleum-based product. The 6-inch perforated had an increasing trend, while 4-inch wrapped had a decreasing trend.

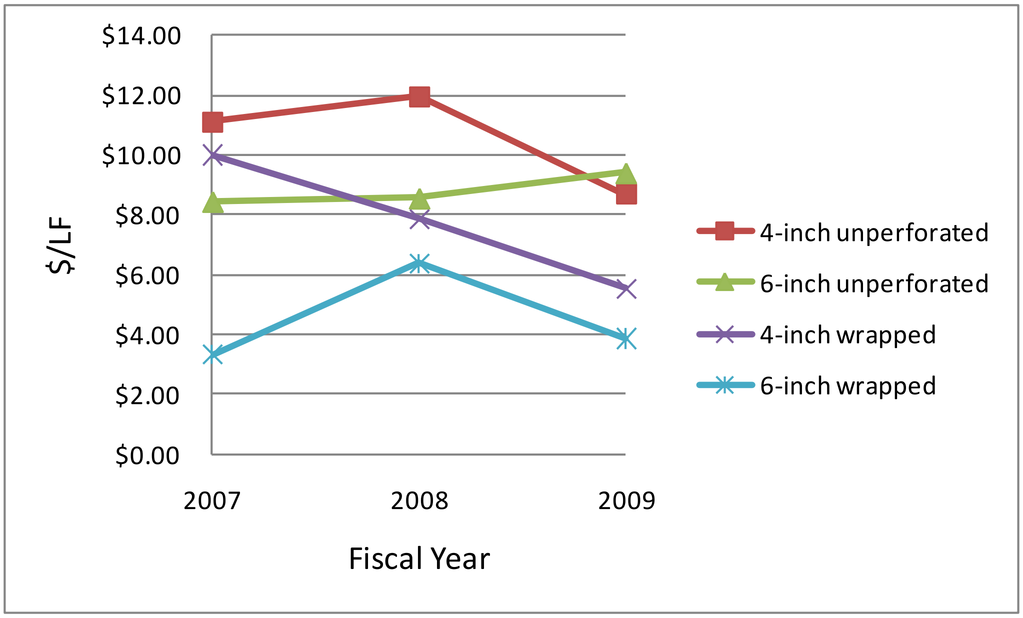

Figure 7.4 Unit Prices for Underdrain Pipe

Unit bid prices were then compared to U.S. Consumer Price Index (inflation rates), Producer Price Index, and the Building Construction Cost Index (BCCI) computed by ENR, to determine if the unit price trends were consistent with broader index values. The BCCI is computed with 68.38 hours of skilled labor at the 20-city average of bricklayers, carpenters and structural ironworkers rates, plus 25 cwt of standard structural steel shapes at the mill price prior to 1996 and the fabricated 20-city price from 1996, plus 1.128 tons of portland cement at the 20-city price, plus 1,088 board-ft of 2 x 4 lumber at the 20-city price (ENR 2010). Table 7.3 compares the change in prices for the CPI, PPI, BCCI, and three primary unit price items. Changes in dense-graded base more closely reflected inflation and the BCCI than 10-inch PCC pavement and OGBC. This may be explained by dense-graded base used across a greater number of project applications than PCC and OGBC, thus, aligning more closely than PCC pavement and OGBC. Based on this review, it was decided to use the most recent FY 2009 unit prices in the analysis.

Table 7.3 Unit Price Change Comparison

Year (1)	U.S. inflation rate (CPI), % (2)	Producer Price Index, % (3)	BCCI, index (4)	BCCI, % change (5)	PCC 10-inch, $/SY (6)	PCC 10-inch, % change (7)	Dense graded ¾-inch, $/ton (8)	Dense graded ¾-inch, % change (9)	Open graded, $/ton (10)	Open graded, % change (11)
2007	2.8	3.9	4554	2.5	20.88	--	9.78	--	10.28	--
2008	3.8	6.3	4796	5.3	24.45	17.1	10.89	11.3	11.86	15.4
2009	-0.3	-2.6	4795	0	27.46	12.3	11.05	1.5	13.73	15.8

7.3 Timing of Maintenance and Rehabilitation Treatments

An important step in an LCCA is to determine the timing of maintenance and rehabilitation treatments. A combination of WisDOT policies and findings from the doweled sections on USH 18/151, STH 29, and USH 151 were used to determine the treatment schedule. Only doweled sections were considered because of the 1988 design policy change.

The initial service life from the WisDOT FDM is shown in Table 7.4. According to the FDM, an undrained PCC pavement has a service life of 25 years, while a drained pavement adds 6 years of service life to yield 31 years. Rehabilitation service lives are shown in Table 7.5. The FDM further states that the service lives for drained pavement structures are estimates that add 25 percent more life onto like undrained pavement structures. Service lives of pavement rehabilitations over drained bases are considered the same as like pavement rehabilitations over undrained bases (WisDOT 2008).

Table 7.4 Initial Service Life (WisDOT 2008)

Initial Construction (1)	Service Life, years (2)
HMA – Traditional or Deep-Strength	18
HMA (drained) – Traditional or Deep-Strength	22
HMA – Perpetual	16
HMA over Pulverized HMA	18
HMA over Rubblized Concrete	22
Concrete	25
Concrete (drained)	31
Concrete over Rubblized Concrete	31

To establish the most probable sequence of rehabilitations, a standard sequence has been developed to maximize initial cost expenditures (WisDOT 2008). Table 7.6 shows typical rehabilitation scenarios and standard sequences that should be used as guidance.

Table 7.5 Rehabilitation Service Life (WisDOT 2008)

Rehabilitation (1)	Service Life, years (2)
HMA Overlay over Traditional HMA Pavement	12
HMA Overlay over CRCP	8
HMA Overlay over JRCP	8
HMA Overlay over JPCP	15
Mill and HMA Overlay over Deep-Strength HMA Pavement	12
Mill and HMA Overlay over Perpetual HMA Pavement	16
Concrete Grind	15[a]
Concrete Pavement Repair and Grind	15[a]
[a] New service life values to be published in 2010.	

Table 7.6 Pavement Life Cycle for Doweled JPCP (WisDOT 2008)

Scenario (1)	Rehabilitation Options[1] (2)
Initial Construction (Concrete Pavement over granular base)	
First Rehabilitation (Functional Repair)	Concrete Pavement Repair and Grind or Concrete Partial Depth Repair or Concrete Pavement Repair and HMA Overlay
Second Rehabilitation (Functional or Structural Repair)	Concrete Pavement Repair and Grind or Concrete Pavement Repair and HMA Overlay
Third Rehabilitation (Functional or Structural Repair)	Concrete Pavement Repair and HMA Overlay or HMA Mill, Concrete Pavement Repair and HMA Overlay or Concrete Pavement Repair and Concrete Overlay
Reconstruction	Pavement Removal and Pavement Reconstruction or Concrete Rubblization and Pavement Reconstruction
[1] See Table 7.5 for service lives.	

To supplement the treatment alternatives and policy timelines, the observed distresses and the developed models from analysis in Chapter 5 were used to estimate the timing of the maintenance and rehabilitation treatments. Observed distresses were reported in Tables 5.2 through 5.4, and statistical performance models were developed in Table 5.8. There were only three significant distress models for doweled PCC for base type: (1) slab breakup extent, (2) slab breakup severity, and (3) the extent of distressed joints and cracks. Estimates for slab breakup extent computed a 10% area for DGBC and 12% area for OGBC. ASOG was observed at 0% area. For slab breakup severity, estimated values are 1 for DGBC, 0.8 for OGBC, and 0 for ASOG. Distressed joints and cracks were the only remaining significant distress. DGBC and OGBC were estimated to be slight severity, while ASOG was estimated at 1.5 (between slight and moderate). Additionally, patching (extent=1, severity=1) was observed on the DGBC section. Table 7.7 summarizes distress modes by base type.

Table 7.7 Distress Modes at 20-year Life for Doweled JPCP

Distress Mode (1)	DGBC (2)	OGBC (3)	ASOG (4)
Slab breakup extent	10% area	12% area	0% area
Slab breakup severity	1 (1 to 3 blocks)	0.8 (1 to 3 blocks)	0 (none)
Distressed joints/cracks severity	1 (slight)	1 (slight)	1.5 (slight to moderate)
Patching	Extent = 1 Severity = 1	none	none
IRI	119 in/mile; 134 in/mile	100 in/mile (#1); 119 in/mile (#1); 90 in/mile (#2)	102 in/mile; 134 in/mile

The distress modes were then coupled with best practices and recommendations for rehabilitating in-service concrete pavement. In a WHRP report by Titus-Glover and Darter (2008), guidelines for full-depth concrete repair have been set forth. They are a composition of existing WisDOT guidelines, Wisconsin research, and published national and regional literature pertaining to full-depth repair projects in Wisconsin. Full-depth repair depends on several factors, such as the extent and severity of the distress and rate of deterioration. As a general rule, when 10 to 20 percent of the slabs in the outer traffic lane are cracked, a full-depth repair is needed. The slab breakup and cracking is typically caused by repeated heavy truck loads and loss of support from beneath the slab (Titus-Glover and Darter 2008). Based on the observed conditions and developed models, both the DGBC and OGBC are at or approaching full-depth repair warrants. In fact, the DGBC section #14 on USH 18/151 has received full-depth repair (see Figure 7.5) at least a year prior to the 2009 dowel-bar retrofit and diamond grind project.

The severity of the OGBC has not reached a full “1” rating, and is about 20% less than DGBC with an “0.8” estimated rating. Since the DGBC section on USH 18/151 has already received patching at 20 years, and none has been recorded on the OGBC but is about 80% of the DGBC severity, a reasonable estimate for full-depth repair would be DGBC at 20 years and OGBC at 24 years (20 years plus linear 20% to achieve severity=1 rating). A single patch observed in a 528-foot segment would imply at least 10 patches in a one-mile segment. OGBC extent has been measured at 12% slab area, so 11 patches would be constructed in a one-mile segment. Width of full-depth repair can range from 6 to 15 feet; to standardize the analysis, a 6-foot wide segment a length of 26 feet was estimated.

Figure 7.5 Patching on Doweled Dense-Graded Test Section #14 on USH 18/151

For distressed joints and cracks, or general joint spalling, full-depth repair is recommended when 50% of the joints in the outer lane have medium- or high-severity joint deterioration, and/or when there are about 75 or more medium- or high-severity transverse cracks per mile in the outer traffic lane (Titus-Glover and Darter 2008). Low-severity cracks are part of the design and are not structural distresses. The ASOG sections have reached a slight to moderate condition (USH 151 is slight, USH 18/151 is moderate) at 20 years of age. It can be assumed that the full moderate condition is reached for the test sections in about 5 years at a non-linear rate; 1.5 reached at 20 years and 2 reached at 25 years. Using an assumed single patch observed in a 528-foot segment would result in 10 patches in a one mile segment. Width of full-depth repair was set at 6 feet and length set at 26 feet.

Three rehabilitations at 15-year service lives were planned over a 65-year service life, including a 20-year initial life, 15-year life for full-depth patching, 15-year life for patching and diamond grinding, and 15-year life for asphalt overlay. Since the OGBC and ASOG initial service lives were 24 and 25 years, respectively, the remaining service life of the asphalt overlay was credited back to the cost.

In the summer and fall of 2009, eastbound lanes of USH 18/151 in Iowa and Dane Counties underwent their first rehabilitation. The rehabilitation included dowel-bar retrofit in the non-doweled section, dowel-bar installation in the doweled sections where transverse faulting was noted, full-depth patching, and diamond grinding of the entire surface. A field investigation of USH 18/151 after construction documented the rehabilitation activities listed in Table 7.8. The extent of full-depth patching per mile closely approximated the estimated quantity in life-cycle cost analysis. It must be noted that the location of the dowel-bar retrofit for the doweled sections (12, 13, 14, and 15) was mid-panel to remedy slab breakup.

Table 7.8 Rehabilitation Activities of USH 18/151 Eastbound Lanes in 2009

Test Section (1)	Base Type (2)	Doweled Transverse Joints (3)	Sealed Transverse Joints (4)	Dowel Bar Retrofit (5)	Full Depth Patches (6)
1	OGBC	No	Yes	All	12
2	OGBC	No	No	All	10
3	CSOG	No	Yes	All	8
4	CSOG	No	No	All	12
5	ASOG	No	Yes	All	11
6	ASOG	No	No	All	6
7	DGBC/TIC	No	No	None	3
8	DGBC	No	Yes	All	15
9	DGBC	No	No	All	9
10	DGBC/TIC	Yes	No	None	3
11	CSOG	Yes	No	23 joints	2
12	ASOG	Yes	No	None	5
13	OGBC	Yes	No	33 joints	5
14	DGBC	Yes	No	17 joints	8
15	DGBC	Yes	Yes	15 joints	5

7.4 LCCA Computations

Life-cycle costing process illustrated the interface of standard WisDOT practice and recent bid prices with the results of the performance analysis in this study. The general inputs considered in the analysis included initial construction cost, maintenance and rehabilitation costs, analysis period, and interest rate. A 65-year analysis period and a discount rate of 5%, as defined by WisDOT policy, were used in the analysis.

After all the LCCA parameters were identified, the LCCA calculation was performed using standard engineering economic analysis procedures for computing present worth costs. For alternatives that have rehabilitation cycles that extend beyond 65 years, a “Rehabilitation Salvage Value” was calculated and credited back into the alternative’s “Total Facility Cost.” The “Rehabilitation Salvage Value” calculation consists of discounting the linearly prorated rehabilitation cost (WisDOT 2008). The cost to place a base and pave a 38-foot wide roadway section for a distance of 1 mile is calculated in Table 7.9.

Table 7.9 New Construction Cost for 38-foot wide 1-mile length PCC Roadway

Base Type (1)	Work Item (2)	Quantity (3)	Unit (4)	Unit Price (5)	Total Cost (6)
Dense Graded Base	PCC Pavement, 10-inch	15253	SY	27.46	$418,857
	Pavement dense graded base, 6-inch thick	2542	CY	24.50	$62,284
	AC inside shoulder, E-0.3	387	Ton	37.12	$14,373
	AC inside shoulder, PG 58-28, 5.5%	21	Ton	295.37	$6,290
	AC outside shoulder, E-0.3	1033	Ton	37.12	$38,328
	AC outside shoulder, PG 58-28, 5.5%	57	Ton	295.37	$16,774
	AC inside shld DGBC, 13-inch thick	636	CY	24.50	$15,571
	AC outside shld DGBC, 12-inch thick	1564	CY	24.50	$38,329
				Total =	$610,806
Open Graded Base	PCC Pavement, 10-inch	15253	SY	27.46	$418,857
	Pavement open graded base, 4-inch thick	1695	CY	36.05	$61,098
	Pavement dense graded base, 4-inch thick	1695	CY	24.50	$41,523
	AC inside shoulder, E-0.3	387	Ton	37.12	$14,373
	AC inside shoulder, PG 58-28, 5.5%	21	Ton	295.37	$6,290
	AC outside shoulder, E-0.3	1033	Ton	37.12	$38,328
	AC outside shoulder, PG 58-28, 5.5%	57	Ton	295.37	$16,774
	AC inside shld DGBC, 13-inch thick	636	CY	24.50	$15,571
	AC outside shld DGBC, 12-inch thick	1564	CY	24.50	$38,329
	Long. Perf. Wrapped Pipe, 6-inch dia	10560	LF	3.86	$40,762
	Trans. Pipe, 6-inch dia.	486	LF	9.40	$4,566
	Apron endwalls	44	Each	132.99	$5,883
				Total =	$702,353
Asphalt Stabilized Base	PCC Pavement, 10-inch	15253	SY	27.46	$418,857
	Asphaltic pavement base, 4-inch thick	3356	Ton	47.52	$159,464
	Pavement dense graded base, 4-inch thick	1695	CY	24.50	$41,523
	AC inside shoulder, E-0.3	387	Ton	37.12	$14,373
	AC inside shoulder, PG 58-28, 5.5%	21	Ton	295.37	$6,290
	AC outside shoulder, E-0.3	1033	Ton	37.12	$38,328
	AC outside shoulder, PG 58-28, 5.5%	57	Ton	295.37	$16,774
	AC inside shld DGBC, 13-inch thick	636	CY	24.50	$15,571
	AC outside shld DGBC, 12-inch thick	1564	CY	24.50	$38,329
	Long. Perf. Wrapped Pipe, 6-inch dia	10560	LF	3.86	$40,762
	Trans. Pipe, 6-inch dia.	486	LF	9.40	$4,566
	Apron endwalls	44	Each	132.99	$5,883
				Total =	$800,720

An additional two inches of subbase for a dense-graded base was omitted since it assumed that the roadway profile would be adjusted for grade difference. Since project time frame and number of rehabilitations were similar, these items were omitted:

- Preliminary engineering costs
- Engineering and contingencies
- Mobilization
- Traffic control costs
- Sales tax (included in material price and overall unit price)
- Labor overhead (included in bid unit price)
- Rumble strips since design varies (integral to PCC, or AC shoulder).
- Pavement markings

The dense-graded section has the lowest initial construction cost of $610,806. It costs an additional $91,547 to construct an untreated OGBC system, an increase of 14%. This cost increase is from approximately $40,000 for permeable stone base and $50,000 for the gravity drained pipe system. (Note, this percentage would change if asphalt shoulders and base were removed from the cost). The ASOG system costs more than both the DGBC and OGBC systems, with relative cost increases of 31% and 14%.

The rehabilitation costs for all three alternatives include full-depth repair for a 6-foot x 26-foot area and depth of 10 inches. Total volume of concrete per repair is 4.8 CY. Bid price for full-depth concrete repair is $202.00/CY, or a total cost of $970 each. Crack filling has not been clearly defined and was omitted from the LCCA, and would have a negligible effect if the timing is similar among alternatives.

The costs of the relevant activities were computed for each year with data using the line-item construction and treatment estimates. Present worth costs were computed for all future rehabilitations using standard time-based Equation 7.1.

$$\$P = \$F / (1 + i)^{n} \qquad (7.1)$$

Where,

$P = Current year (2009) cost;
$F = Future year cost adjusted for inflation/discount rate;
i = Interest rate, 0.05; and
n = Number of years between base year and rehabilitation treatment.

Tables 7.10 through 7.12 compute the 65-year life-cycle costs for the three base alternatives. A summary of costs is provided in Table 7.13. Table 7.14 combines the cost analysis with primary performance characteristics identified earlier.

Table 7.10 Dense Graded LCCA Computations for 1-mile length of Roadway

Year (1)	Treatment (2)	Description (3)	Quantity (4)	Unit Price (5)	Total Cost (6)	Present Worth Cost (7)
0	New construction	10-inch PCC Pavement	---	---	$610,806	$610,806
20	Rehabilitation	Full depth repair	48 CY	$202.00/CY	$9,696	$3,654
		10 panels per mile				
35	Rehabilitation	Full depth repair	48 CY	$202.00/CY	$9,696	$1,758
		Concrete diamond grinding	15,253 SY	$2.40/SY	$36,607	$6,637
		Salvaged asph. pavt. inside	408 Tons	$5.17/ton	$2,109	$382
		Salvaged asph. pavt. outside	1090 Tons	$5.17/ton	$5,635	$1,022
		AC inside shoulder, E-0.3	387 Tons	$37.12/ton	$14,373	$2,606
		AC inside shoulder, PG 58-28	21 Tons	$295.37/ton	$6,290	$1,140
		AC outside shoulder, E-0.3	1033 Tons	$37.12/ton	$38,328	$6,948
		AC outside shoulder, PG 58-28	57 Tons	$295.37/ton	$16,774	$3,041
50	Rehabilitation	AC overlay, E-10	4905 Tons	$47.19/Ton	$231,467	$20,185
	AC Overlay	AC cement, PG 58-28	270 Tons	$295.37/Ton	$79,750	$6,954
65	Remaining Service Life		0	0	$0	$0
				Totals	**$1,061,531**	**$665,133**

Table 7.11 Open Graded LCCA Computations for 1-mile length of Roadway

Year (1)	Treatment (2)	Description (3)	Quantity (4)	Unit Price (5)	Total Cost (6)	Present Worth Cost (7)
0	New construction	10-inch PCC Pavement	---	---	$702,353	$702,353
24	Rehabilitation	Full depth repair	53 CY	$202.00/CY	$10,706	$3,320
		11 panels per mile				
39	Rehabilitation	Full depth repair	53 CY	$202.00/CY	$10,706	$1,597
		Concrete diamond grinding	15,253 SY	$2.40/SY	$36,607	$5,460
		Salvaged asph. pavt. inside	408 Tons	$5.17/ton	$2,109	$315
		Salvaged asph. pavt. outside	1090 Tons	$5.17/ton	$5,635	$840
		AC inside shoulder, E-0.3	387 Tons	$37.12/ton	$14,373	$2,144
		AC inside shoulder, PG 58-28	21 Tons	$295.37/ton	$6,290	$938
		AC outside shoulder, E-0.3	1033 Tons	$37.12/ton	$38,328	$5,716
		AC outside shoulder, PG 58-28	57 Tons	$295.37/ton	$16,774	$2,502
54	Rehabilitation	AC overlay, E-10	4905 Tons	$47.19/Ton	$231,467	$20,185
	AC Overlay	AC cement, PG 58-28	270 Tons	$295.37/Ton	$79,750	$6,954
65	Remaining Service Life		0	0	-$82,991	-$3,481
				Totals	**$1,072,107**	**$748,843**

Table 7.12 Asphalt Stabilized OGBC LCCA Computations for 1-mile length of Roadway

Year (1)	Treatment (2)	Description (3)	Quantity (4)	Unit Price (5)	Total Cost (6)	Present Worth Cost (7)
0	New construction	10-inch PCC Pavement	---	---	$800,720	$800,720
25	Rehabilitation	Full depth repair	48 CY	$202.00/CY	$9,696	$2,863
		10 panels per mile				
40	Rehabilitation	Full depth repair	48 CY	$202.00/CY	$9,696	$1,377
		Concrete diamond grinding	15,253 SY	$2.40/SY	$36,607	$5,200
		Salvaged asph. pavt. inside	408 Tons	$5.17/ton	$2,109	$300
		Salvaged asph. pavt. outside	1090 Tons	$5.17/ton	$5,635	$800
		AC inside shoulder, E-0.3	387 Tons	$37.12/ton	$14,373	$2,042
		AC inside shoulder, PG 58-28	21 Tons	$295.37/ton	$6,290	$893
		AC outside shoulder, E-0.3	1033 Tons	$37.12/ton	$38,328	$5,444
		AC outside shoulder, PG 58-28	57 Tons	$295.37/ton	$16,774	$2,383
55	Rehabilitation	AC overlay, E-10	4905 Tons	$47.19/Ton	$231,467	$20,185
	AC Overlay	AC cement, PG 58-28	270 Tons	$295.37/Ton	$79,750	$6,954
65	Remaining Service Life		0	0	-$103,739	-$4,351
				Totals	**$1,147,706**	**$844,810**

Table 7.13 Categorical Cost Comparison of Base Alternatives

Base Type (1)	Net Present Worth ($/roadway-mile)			
	Initial Construction (2)	Rehabilitation (3)	Salvage (4)	Total (5)
DGBC	610,806	54,327	0	665,133
OGBC	702,353	49,971	-3,481	748,843
ASOG	800,720	48,441	-4,351	844,810

Dense-graded base was the least cost among all base alternatives, with a total estimated present-worth cost of $665,133 per roadway mile. Open-graded permeable bases were more expensive, with the estimated cost of untreated open-graded base at $748,843 and asphalt-stabilized open-graded base at $844,810. These costs translate to increases of 13% for untreated open-grade base and 27% for asphalt-stabilized open-graded base. When only cost is considered, the dense-graded base is the recommended choice. Rehabilitation cost for dense-graded base was $4,000 to $6,000 more than the permeable base, but first construction cost was the primary determinant. Another factor in choosing dense-graded base over open-graded base is the drainage conditions on the project as set forth in the FDM guidelines. Also, ride performance is another factor, where, on USH 18/151, the dense-graded base sections had an IRI =135 ipm (unsealed) and IRI =119 ipm (sealed), while drained sections were approximately IRI=100 ipm.

Table 7.14 Cost and Performance Comparison of Doweled PCC Base Alternatives

Base Type (1)	Net Present Worth, $/roadway-mile (2)	20-year Pavement Distresses (3)	20-year Surface Roughness (4)
Dense Graded Base Course	$665,133	• 10% slab break up • Slight distressed joints and cracks	• Rougher ride • IRI ≥ 119 in/mile
Open Graded Base Course	$748,843	• 12% slab break up • Slight distressed joints and cracks	• Smoother ride • IRI = 90 in/mile, Gradation #2 • IRI = 100 and 119 in/mile, Gradation #1
Asphalt Stabilized Open Graded Base Course	$844,810	• No slab break up • Moderate distressed joints and cracks • Lower composite distresses	• Smooth to rough ride • IRI = 102 and 134 in/mile

CHAPTER 8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

This research examined the performance of 20-year old doweled and non-doweled PCC pavement sections constructed over a variety of base course materials. The test sections examined were located along USH 18/151 (17 test sections), STH 29 in Brown County (4 test sections), and USH 151 (4 test sections). Data were collected for the Pavement Distress Index (PDI), International Roughness Index (IRI), Falling Weight Deflectometer (FWD), and water drainage to evaluate pavement performance, support conditions, and water permeability through the base course. Both automated and manual pavement condition surveys were conducted for each test section. First, a semi-automated electronic survey were collected for transverse faulting and ride quality with IRI measurements in both wheel paths. Pavement condition was manually measured for traditional PCC pavement distresses, including slab breakup, distressed joints and cracks, joint crack filling, patching, surface distress, longitudinal joint distress and distortion, and transverse faulting. In addition, a life cycle cost analysis was conducted to determine the most cost effective base course material for consideration in PCC pavement design. On the basis of data analyses, several conclusions that relate to the pavement characteristics are presented in the following sub-sections.

8.1.1 Base Type

For doweled pavements along USH 18/151, dense-graded base sections exhibited poor ride quality compared to open-graded base sections. There was no significant difference in ride quality among open-graded base sections. Asphalt-stabilized open-graded bases exhibited the least composite distresses when compared to a dense-graded section on USH 18/151.

For non-doweled sections on USH 18/151, the cement-stabilized, asphalt-stabilized, and TIC drains had the least amount of distress. Dense-graded and untreated OGBC had the highest composite measure of pavement distress. Asphalt-stabilized open base and TIC drains had the smoothest ride, while untreated and cement-stabilized OGBC had the rougher ride.

USH 151 had doweled 10-thick PCC, unsealed skewed transverse joints, paved over a 4-inch top permeable base (untreated with two gradations, cement-stabilized, and asphalt-stabilized) and 4-inch bottom dense base. All permeable base types had nearly the same performance among the different bases with slight distressed joints/cracks. Slight differences were untreated aggregate with 10% of slab area with slab breakup and surface distresses, and asphalt-stabilized OGBC having slight transverse faulting. The finer New Jersey open-graded base had the smoothest ride when compared to other open-graded sections. Asphalt-stabilized open-graded base had the roughest ride, and un-stabilized and cement-stabilized open-graded bases had intermediate values. In summary, the much finer-graded New Jersey base had less composite distresses and a smoother ride.

8.1.2 Transverse Dowels

Combined data from the three projects found that non-doweled pavement generally has a higher distress level than doweled, however, when two non-doweled outliers are removed, the difference is less pronounced. The extent of transverse faulting was equal among all test sections, however, the severity was higher for non-doweled joints with about half of those sections rated a level 2 (¼ to ½ inch). All doweled sections were either at or less than 0.02 inches. IRI was generally higher on non-doweled pavements, but many doweled sections had an equal roughness to non-doweled sections.

8.1.3 Sealant

USH 18/151 sealed non-doweled joints produced a better performing pavement than unsealed joints, however, sealant did not appear to have a consistent effect on ride. On two doweled dense-graded sections, sealant slightly outperformed the unsealed section, with minor patching as the prominent distress for the unsealed section. Both sections had identical extent and severity levels for slab breakup, distressed joints/cracks, surface distress, longitudinal distress, and transverse faulting.

STH 29 unsealed sections for doweled/non-doweled joints performed better than the median PDI for the sealed sections. The sealed doweled pavement did perform a little better than the non-doweled section, but the opposite occurred on the non-doweled sections. Sealed doweled joints had a smoother ride than the other combinations. Sealed/non-doweled joints produced the roughest ride, and as expected, non-doweled joints, whether sealed or unsealed, had the highest IRI values.

8.1.4 Drainage

The average hydraulic conductivity for the unstabilized permeable base (OGPB) is 17,481 fpd and there appears little variation due to doweling or joint sealant. The average hydraulic conductivity for the cement-stabilized permeable base (CSOG) is 15,129 fpd and there is a substantial variation due to joint sealant, with the sealed section having a hydraulic conductivity of 21,212 fpd and the unsealed sections averaging 12,087 fpd. The average hydraulic conductivity for the asphalt-stabilized permeable base (ASOG) is 8,471 fpd which is significantly lower than the OGPB and CSOG sections. There appears to be a slight variation due to doweling with the doweled section having a hydraulic conductivity of 5,920 fpd and the non-doweled sections averaging 9,747 fpd.

The results provided for STH 29 Brown County indicate adequate drainage capacity in all sections. The data indicates a significant variation due to doweling but little variation due to joint sealant. The average hydraulic conductivity for the unstabilized permeable base (OGPB) sections without dowels is 2,817 fpd and 13,637 fpd for the doweled test sections.

The results provided for USH 151 Columbia-Dane Counties indicate adequate drainage capacity in only the cement stabilized permeable base section (CSOG), with a calculated hydraulic conductivity of 10,697 fpd. The base layers in the remaining three test sections would not accept water, indicating a complete blockage of the layer. The reason for this condition is unknown.

Overall, for the USH 18/151 Iowa/Dane County test sections, while all bases can be considered adequately drained (k > 1,000 fpd), there appears to be a substantial reduction in the flow capacity for the ASOG base (#12) when compared to the other permeable bases. This section, however, is performing well in comparison to others in terms of PDI and IRI values. STH 29 Brown County sections indicate reduced drainage capacity for the non-doweled OGBC base sections 1 and 2. USH 151 Columbia/Dane County sections indicate poor drainage capacity for all but the CSOG section.

8.1.5 Structural Capacity

The deflection load transfer results indicate expected high average values for the doweled sections and fair to poor values for the non-doweled sections. For USH 18/151, the overall average load transfer values for the doweled and non-doweled sections were 94.8% and 40.9%, respectively. For the non-doweled sections, the overall average load transfer values for the sealed and unsealed sections were 45.1% and 38.5%, respectively. For the doweled sections, the overall average load transfer values for the sealed and unsealed sections were 96.0% and 94.7%, respectively. For STH 29, the overall average load transfer values for the doweled and non-doweled sections were 93.0% and 17.9%, respectively. Little variation was noted for the sealed and unsealed sections. For USH 151, the overall average load transfer value for the doweled sections was 98.3%.

The slab support ratios indicate variable results based on base type, joint reinforcement and joint sealant. For USH 18/151 Iowa-Dane Counties, all corner support ratios indicate full support is maintained. The edge support ratios generally indicate full support is maintained with the exception of three doweled and unsealed sections; namely sections 10a (SSRe=0.58), 13 (SSRe=0.54) and 14 (SSRe=0.67). These reduced values (< 0.75) suggest support problems due to densification of the base layers which is not normally expected for doweled sections. For the STH 29 sections, reduced edge support is noted for undoweled section 2 (SSRe=0.73) and doweled section 3 (SSRe=0.69) and reduced corner support is noted for doweled section 3 (SSRc=0.63). While these values are near the trigger value of 0.75, indicating only minor loss of support, it is interesting to note that these are the sealed sections. The results from USH 151 Columbia-Dane Counties indicates support problems under all edges and corners, with SSR values ranging from a low of 0.16 to a high of 0.66.

The results of the permeability and FWD tests may provide insight into the performance of the various test sections. For the USH 18/151 Iowa/Dane County test sections, poor load transfer evident in all non-doweled sections has led to increased faulting in all but the ASOG sections 5 and 6 and increased roughness in all but the ASOG sections 5 and 6 and the TIC sections 7a – 7c. Poor slab support ratios in the doweled sections 10a, 13 and 14 has only led to increased roughness in the DGBC section (#14). The PDI for all sections is generally comparable with the exception of increased PDI values in TIC section 7a and OGBC section 2. As a whole, these results indicate the ASOG is providing the best overall performance.

The results from the STH 29 Brown County non-doweled OGBC base sections 1 and 2 indicate comparably poor load transfer, increased faulting and increased roughness. The doweled and sealed section 3 exhibits reduced edge and corner support and increased PDI.

The results from USH 151 Columbia/Dane County sections indicate poor edge and corner support for all sections. However, the PDI values are similar for all sections and only the ASOG section 3 has increased roughness.

8.1.6 Life-Cycle Cost Analysis

A life-cycle cost analysis found that dense-graded base was the least cost among all base alternatives, with a total estimated present-worth life-cycle cost of $665,133 per roadway mile. Open-graded permeable bases were more expensive, with the estimated cost of untreated open-graded base at $748,843 and asphalt-stabilized open-graded base at $844,810. These costs translate to increases of 13% for untreated open-grade base and 27% for asphalt-stabilized open-graded base. Rehabilitation cost for dense-graded base was $4,000 to $6,000 more than the permeable base, but first construction cost was the primary determinant.

8.2 Recommendations

- An investigation is warranted as to why permeable bases (with the exception of CSOG) on USH 151 in Columbia-Dane counties failed to drain during field testing. This suggests a complete blockage of the base layers or drainage system. The blockage may further explain the poor slab edge and corner support problems as revealed in this research for the USH 151 sections. The trapped water can potentially affect the foundation K-value and cause pumping in addition.

- This research suggests that the selection of base material for PCC pavements should be based on factors including life-cycle cost, drainage characteristics, pavement distresses, and overall expected performance as dictated by ride quality, which is measured in terms of the International Roughness Index (IRI). If cost is the only consideration, then dense-graded base is the recommended choice. If ride quality is the preferred criterion, then permeable bases should be considered.

- Testing and analysis of additional test sections in the state will augment the data and findings in this report. A simplified analysis would involve collection and synthesis of WisDOT PDI and IRI performance data traditionally collected on a biennial basis. An advanced analysis would include physical field testing similar to that conducted in this study. Projects from the Rutkowski report (1998) include STH 50, STH 164, IH

43 in both Ozaukee and Walworth Counties. There was an additional IH 90 segment in Rock County not included in the Rutkowski report.

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Appendix A - Adjustment of Data Sets

Tables A.3 through A.7 provides the locations of the RP and Sequence Numbers for the three projects, along with a comparison of the Sequence Number length with as-built construction stationing. The purpose of this comparison was to verify that the traditional Sequence Number was compatible with the construction plans during the overlay process.

The beginning and ending limits for STH 29 and USH 151 projects did not extend to the limits of the Sequence Number, thus, a verification was not possible. It was possible to compare all Sequence Number locations with as-built stationing on USH 18/151, and the comparison proved that the lengths were within 1/100th of a mile. This is important, since it confirms that the stationing and RP overlay are compatible.

Table A.1 STH 29/32 Data Set Locations of Pathway Profiler Van

Index (1)	Distance to PDI Survey Segment (2)	Reference Point Number (3)	Road Name (4)	Reference Point From Feature (5)	Reference Point To Feature (6)	PIF Section Length (7)	Lane (8)
29				SITE	DELIMITER		
30	1.0	284K 0.0	029E	CTH U	TEST 1	3229.0	Driving Lane
31	1.0	001 0.0	029E	TEST 1	TEST 2	2500.0	Driving Lane
32	1.0	002 0.0	029E	TEST 2	TEST 3	2500.0	Driving Lane
33	1.0	003 0.0	029E	TEST 3	CONTROL	2500.0	Driving Lane
34	1.0	004 0.0	029E	CONTROL	END	2500.0	Driving Lane
35				SITE	DELIMITER		

Table A.2 USH 151 Data Set Locations of Pathway Profiler Van

Index (1)	Distance to PDI Survey Segment (2)	Reference Point Number (3)	Road Name (4)	Reference Point From Feature (5)	Reference Point To Feature (6)	PIF Section Length (7)	Lane (8)
35				SITE	DELIMITER		
36	1.0	95K 0.0	151S	STH 73 OH (LEAD IN)	TEST 1	8052.0	Driving Lane
37	1.0	001 0.0	151S	TEST 1	SKIP	528.0	Driving Lane
38	1.0	002 0.0	151S	SKIP	TEST 2	3473.0	Driving Lane
39	1.0	003 0.0	151S	TEST 2	SKIP	528.0	Driving Lane
40	1.0	004 0.0	151S	SKIP	TEST 3	1972.0	Driving Lane
41	1.0	005 0.0	151S	TEST 3	SKIP	528.0	Driving Lane
42	1.0	006 0.0	151S	SKIP	TEST 4	1353.0	Driving Lane
43	1.0	95K 0.9	151S	TEST 4	END	528.0	Driving Lane
44				DIRECTION	DELIMITER		
45	1.0	95K 0.0	151S	STH 73 OH (LEAD IN)	TEST 1	8052.0	Passing Lane
46	1.0	001 0.0	151S	TEST 1	SKIP	528.0	Passing Lane
47	1.0	002 0.0	151S	SKIP	TEST 2	3473.0	Passing Lane
48	1.0	003 0.0	151S	TEST 2	SKIP	528.0	Passing Lane
49	1.0	004 0.0	151S	SKIP	TEST 3	1972.0	Passing Lane
50	1.0	005 0.0	151S	TEST 3	SKIP	528.0	Passing Lane
51	1.0	006 0.0	151S	SKIP	TEST 4	1353.0	Passing Lane
52	1.0	95K 0.9	151S	TEST 4	END	528.0	Passing Lane
				SITE	DELIMITER		

Table A.3 USH 18/151 Data Set Locations of Pathway Profiler Van

Index (1)	Distance to PDI Survey Segment (2)	Reference Point Number (3)	Road Name (4)	Reference Point From Feature (5)	Reference Point To Feature (6)	PIF Section Length (7)	Lane (8)
1				SITE	DELIMITER		
2	6125.0	108G 0.0	18 W	CTH HHH INT	BEGIN TEST 3 (CTH HHH)	8237.0	Driving Lane
3	5702.0	106K 0.0	18 W	TEST 3 (CTH HHH)	TEST 2 (CTH BB)	7814.0	Driving Lane
4	5227.0	103G 0.0	18 W	TEST 2 (CTH BB)	TEST 1 (CTH Y)	7339.0	Driving Lane
5	4699.0	101K 0.0	18 W	TEST 1 (CTH Y)	END TEST 1 (Dodge BR Stream STR)	6811.0	Driving Lane
6	5333.0	101K 1.2	18 W	END TEST 1 (Dodge BR Stream STR)	USH 151S INT	7973.0	Driving Lane
7				SITE	DELIMITER		
8	1584.0	101K 0.0	18 E	CTH Y	BEGIN TEST 1 (CTH BB)	7339.0	Driving Lane
9	1584.0	103G 0.0	18 E	TEST 1 (CTH BB)	TEST 2 (CTH HHH)	7814.0	Driving Lane
10	1584.0	106K 0.0	18 E	TEST 2 (CTH HHH)	TEST 3 (CTH HHH)	8237.0	Driving Lane
11	1584.0	108G 0.0	18 E	TEST 3 (CTH HHH)	TEST 4 (W. Brigham Rd)	5650.0	Driving Lane
12	1584.0	110M 0.0	18 E	TEST 4 (W. Brigham Rd)	TEST 5 (Thompson Rd)	5122.0	Driving Lane
13	1584.0	111K 0.0	18 E	TEST 5 (Thompson Rd)	TEST 6 (CTH T)	2640.0	Driving Lane
14	1584.0	112D 0.0	18 E	TEST 6 (CTH T)	TEST 7 (CTH ID OH)	6758.0	Driving Lane
15	1584.0	113M 0.0	18 E	TEST 7 (CTH ID OH)	TEST 8 (CTH K)	6019.0	Driving Lane
16	1584.0	115G 0.0	18 E	TEST 8 (CTH K)	TEST 9 (Mound View Rd)	4488.0	Driving Lane
17	1584.0	117M 0.0	18 E	TEST 9 (Mound View Rd)	TEST 10 (Co Line)	6706.0	Driving Lane
18	1584.0	117M 0.0	18 E	TEST 10 (Co Line)	End Test 10	6706.0	Driving Lane
19				SITE	DELIMITER		
20	879.0	119K 0.2	18 E	DANE CO LN (TEST 1)	CTH F	3749.0	Driving Lane
21	2139.0	121K 0.0	18 E	CTH F (TEST 2)	CAVE OF MOUNDS RD	3960.0	Driving Lane
22	5404.0	122G 0.0	18 E	CAVE/MOUNDS RD(T #3)	CTH E	11035.0	Driving Lane
23	417.0	126D 0.0	18 E	CTH E (TEST 4)	STH 78 OH	3062.0	Driving Lane
24	542.0	127K 0.0	18 E	STH 78 OH (CONTROL)	SANDROCK RD STR	4805.0	Driving Lane
25	0.0	128M 0.0	18 E	SANDROCK RD STR(SKIP	CTH "JG" OH	4171.0	Driving Lane
26	680.0	129G 0.0	18 E	CTH "JG" OH (TEST 5)	STH 92 STR	3062.0	Driving Lane
27	0.0	130D 0.0	18 E	STH 92 STR (SKIP)	CTH "ID" OH	5386.0	Driving Lane
28	2246.0	131K 0.0	18 E	CTH "ID" OH (TEST 6)	TOWN HALL RD OH	3010.0	Driving Lane
29				SITE	DELIMITER		

Table A.4 STH 29/32 Sequence Number Locations

Section Name (1)	Start Description (2)	End Description (3)	Length, feet (4)	Length, mile (5)	Station, mile (6)	As-built Start Sta. (7)	As-built End Sta. (8)
Seq 36180	Center CTH 'U	Center 'VV'	7181	1.36	out limits	out limits	559+50
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	out limits	out limits
Seq 36190	Center CTH 'VV'	Center Sunlite Dr	7210	1.39	1.37	559+50	631+60
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	575+34	580+62

Table A.5 USH 151 Sequence Number Locations

Section Name (1)	Start Description (2)	End Description (3)	Length, feet (4)	Length, mile (5)	Station, mile (6)	As-built Start Sta. (7)	As-built End Sta. (8)
Seq 126370	STH 73 OH	STH 73 STR	11563	2.19	---	out limits	out limits
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	---	out limits	out limits
Seq 126350	STH 73 STR	Columbia / Dane Co. Line	4649	0.92	0.88	1202+98	1156+49
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	---	---	---
Seq 126360	Columbia / Dane Co. Line	CTH 'V' OH	8659	1.64	---	1156+49	---
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	---	---	---

Table A.6 USH 18/151 Iowa County EB Sequence Number Locations

Section Name (1)	Start Description (2)	End Description (3)	Length, feet (4)	Length, mile (5)	Station, mile (6)	As-built Start Sta. (7)	As-built End Sta. (8)
Seq 21000	Center CTH 'Y'	Center CTH 'BB'	7266	1.39	1.38	358+65	521+01
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	374+49	379+77
Seq 21010	Center CTH 'BB'	Center HHH to Ridgeway	7733	1.48	1.46	521+01	599+42
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	536+85	542+13
Seq 21020	Center HHH to Ridgeway	Center HHH to Ridgeway	8769	1.56	1.58	599+42	687+11
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	615+26	620+54
Seq 21030	Center HHH to Ridgeway	Center West Brigham Rd	5614	1.07	1.06	687+11	743+25
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	702+95	708+23
Seq 21040	Center West Brigham Rd	Center Thompson Dr.	5279	0.97	1.00	743+25	796+04
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	759+09	764+37
Seq 21050	Center Thompson Dr.	Center CTH 'T'	2651	0.50	0.50	796+04	822+55
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	811+88	817+16
Seq 21060	Center CTH 'T'	Center CTH 'ID' Overpass	9571	1.28	1.28	822+55	818+42
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	838+39	843+67
Seq 21070	Center CTH 'ID' Overpass	Center CTH 'K'	3108	1.14	1.12	818+42	849+50
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	834+26	839+54
Seq 21080	Center CTH 'K'	Center Mounds View Rd	4500	0.85	0.85	849+50	894+50
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	865+34	870+62
Seq 21090	Center Mounds View Rd	Dane / Iowa Co. Line	6672	1.27	1.26	894+50	961+22
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	910+34	915+62

Table A.7 USH 18/151 Dane County EB Sequence Number Locations

Section Name (1)	Start Description (2)	End Description (3)	Length, feet (4)	Length, mile (5)	Station, mile (6)	As-built Start Sta. (7)	As-built End Sta. (8)
Seq 21100	Dane / Iowa Co. Line	Center CTH 'F'	3739	0.71	0.71	961+22	998+61
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	977+06	982+34
Seq 21110	Center CTH 'F'	Center Cave of the Mounds R	3941	0.75	0.75	998+61	1038+02
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	1014+45	1019+73
Seq 21120	Center Cave of the Mounds R	Center Erbe Road	6964	1.33	1.32	1038+02	1220+61
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	1053+86	1059+14
Seq 21130	Center Erbe Road	Center CTH 'E'	4022	0.76	0.76	1220+61	1260+83
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	1236+45	1241+73
Seq 21140	Center CTH 'E'	Center STH '78'	3075	0.58	0.58	1260+83	1291+58
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	1276+67	1281+95
Seq 21150	Center STH '78'	Center Sand Rock Road	4712	0.91	0.89	1291+58	1221+70
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	1307+42	1312+70

Table A.8 USH 18/151 Iowa County WB Sequence Number Locations

Section Name (1)	Start Description (2)	End Description (3)	Length, feet (4)	Length, mile (5)	Station, mile (6)	As-built Start Sta. (7)	As-built End Sta. (8)
Seq 22140	Center HHH to Ridgeway	Center HHH to Ridgeway	8769	1.56	1.58	687+11	599+42
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	702+95	697+67
Seq 22130	Center HHH to Ridgeway	Center CTH 'BB'	7733	1.48	1.46	599+42	521+01
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	615+26	609+98
Seq 22120	Center CTH 'BB'	Center CTH 'Y'	7267	1.39	1.38	521+01	358+65
WISDOT PDI	0.3 mile past intersection	0.4 mile past intersection	528	0.10	0.10	536+85	531+57

Table A.9 STH 29/32 Research PDI Location Relative to RP

Section Name (1)	Start Description (2)	End Description (3)	Start Sta (4)	Distance to Butt Joint, mile (5)	Length, feet (6)	Length, mile (7)	End Sta. (8)
1	E. of CTH 'U'	---	520+00	0.61	2500	0.47	545+00
Research PDI	E. of CTH 'U'	---	540+00	0.99	500	0.09	545+00
2	E. of CTH 'U'	---	545+00	1.08	2500	0.47	570+00
Research PDI	E. of CTH 'U'	---	545+00	1.08	500	0.09	550+00
3	E. of CTH 'VV'	---	570+00	0.20	2500	0.47	595+00
Research PDI	E. of CTH 'VV'	---	590+00	0.58	500	0.09	595+00

Table A.10 USH 151 Research PDI Location Relative to RP

Section Name (1)	Start Description (2)	End Description (3)	Start Sta (4)	Distance to Butt Joint, mile (5)	Length, feet (6)	Length, mile (7)	End Sta. (8)
4	W. of '73'	---	1250+00	1.33	3500	0.66	1215+00
Research PDI	W. of '73'	---	1240+00	1.52	500	0.09	1235+00
3	W. of '73'	---	1215+00	1.99	2000	0.38	1195+00
Research PDI	W. of '73'	mi W. of W. end Deansville Rd Ov	1205+00	2.18	500	0.09	1200+00
2	W. of W. end of Dean. Rd Overpass	---	1195+00	0.15	2800	0.53	1167+00
Research PDI	W. of W. end of Dean. Rd Overpass	---	1175+00	0.53	500	0.09	1170+00
1	W. of W. end of Dean. Rd Overpass	---	1167+00	0.68	2800	0.53	1139+00
Research PDI	W. of Dane Co. Line	---	1155+00	0.03	500	0.09	1150+00

Table A.11 USH 18/151 Iowa County EB Research PDI Location Relative to RP

Section Name (1)	Start Description (2)	End Description (3)	Start Sta (4)	Distance to Butt Joint, mile (5)	Length, feet (6)	Length, mile (7)	End Sta. (8)
Control			361+80		3683	0.70	488+32
1	E. of CTH 'Y'	E. of Cemetery Road	488+32	0.76	5280	1.00	541+12
Research PDI	Past Butt Joint		504+16	0.30	528	0.10	509+44
2	E. of Cemetery Road	500' West of HHH to Ridgeway	541+12	0.09	5280	1.00	595+00
Research PDI	Past Butt Joint		556+96	0.30	528	0.10	562+24
Control			595+00		5184	0.98	651+31
Bridge			651+51				652+96
Control			653+23		200	0.04	655+23
3	E. of W. End of CTH 'H' Overpass	E. of HHH to Ridgeway	655+23	0.04	5200	0.98	707+23
Research PDI	Past Butt Joint		655+23	0.30	528	0.10	660+51
4	E. of HHH to Ridgeway	E. of W. Brigham Road	707+23	0.38	5200	0.98	760+70
Research PDI	Past Butt Joint		707+23	0.30	528	0.10	712+51
5	E. of W. Brigham Road	E of Boe Harris Road	760+70	0.33	5200	0.98	812+70
Research PDI	Past Butt Joint		760+70	0.30	528	0.10	765+98
6	E of Boe Harris Road	W. of CTH 'ID'	812+70	0.06	5200	0.98	864+70
Research PDI	Past Butt Joint		812+70	0.30	528	0.10	817+98
7	E. of W. Industrial Drive	W. of S. Jones St. Overpass	864+70	0.10	5201	0.98	816+86
Research PDI	Past Butt Joint		864+70	0.30	528	0.10	869+98
GAP	W. of S. Jones St. Overpass		816+86	0.01	295	0.06	819+81
8	E. of S. Jones St. Overpass	E. of CTH 'K'	819+81	0.00	4740	0.90	867+21
Research PDI	Past Butt Joint		819+81	0.30	528	0.10	825+09
9	E. of CTH 'K'	E . of E. Mounds View Road	867+21	0.34	4700	0.89	914+21
Research PDI	Past Butt Joint		867+21	0.30	528	0.10	872+49
10	E of Mounds View Road	Dane Co. Line	914+21	0.37	4701	0.89	961+22
Research PDI	Past Butt Joint		914+21	0.30	528	0.10	919+49

Table A.12 USH 18/151 Dane County EB Research PDI Location Relative to RP

Section Name (1)	Start Description (2)	End Description (3)	Start Sta (4)	Distance to Butt Joint, mile (5)	Length, feet (6)	Length, mile (7)	End Sta. (8)
11	Iowa Co. Line	E. of CTH 'F'	961+22	---	5280	1.00	1014+02
Research PDI	past butt joint		961+22	0.30	528	0.10	966+50
12	E. of CTH 'F'	E. of Cave of the Mounds Road	1014+02	0.29	5280	1.00	1066+82
Research PDI	past butt joint		1014+02	0.30	528	0.10	1019+30
13	E. of Cave of the Mounds Road	E. of Erbe Road	1066+82	0.55	5280	1.00	1232+57
Research PDI	past butt joint		1066+82	0.30	528	0.10	1072+10
14	E. of Erbe Road	STH 78	1232+57	0.23	5280	1.00	1285+37
Research PDI	past butt joint		1232+57	0.30	528	0.10	1237+85
15	E. of CTH 'E'	Sand Rock Road	1285+37	0.46	5313	1.01	1221+50
Research PDI	past butt joint		1285+37	0.30	528	0.10	1290+65

Table A.13 USH 18/151 Iowa County WB Research PDI Location Relative to RP

Section Name (1)	Start Description (2)	End Description (3)	Start Sta (4)	Distance to Butt Joint, mile (5)	Length, feet (6)	Length, mile (7)	End Sta. (8)
3	W. of E. end of CTH 'H' Overpass	HHH to Ridgeway	634+74	0.32	6294	1.19	573+80
Research PDI	past butt joint		634+74	0.30	528	0.10	629+46
2	E. of Ridgeview Road	Center of CTH 'BB'	573+80	0.07	5280	1.00	521+00
Research PDI	past butt joint		573+80	0.30	528	0.10	568+52
1		CTH 'Y'	521+00	CL CTH 'BB'	5281	1.00	378+50
Research PDI	past butt joint		521+00	0.30	528	0.10	515+72

An example of adjusting the performance data to the Research PDI is provided in Figure A.1. This example is provided for Section 3 located in the west bound lane of USH 18/151, which begins 0.32 miles west of CTH 'H' (Grove Street) overpass. The butt joint is located here and the research PDI starts 0.3 miles from this point and runs 0.1 miles west along USH 18/151.

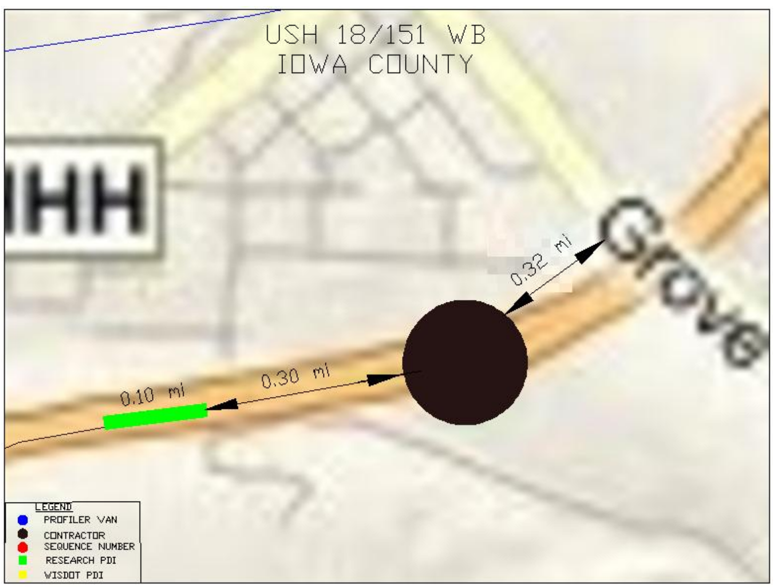

Figure A.1 Locating Research PDI for Section 3 in Westbound USH 18/151

Appendix B – Pavement Cross Sections

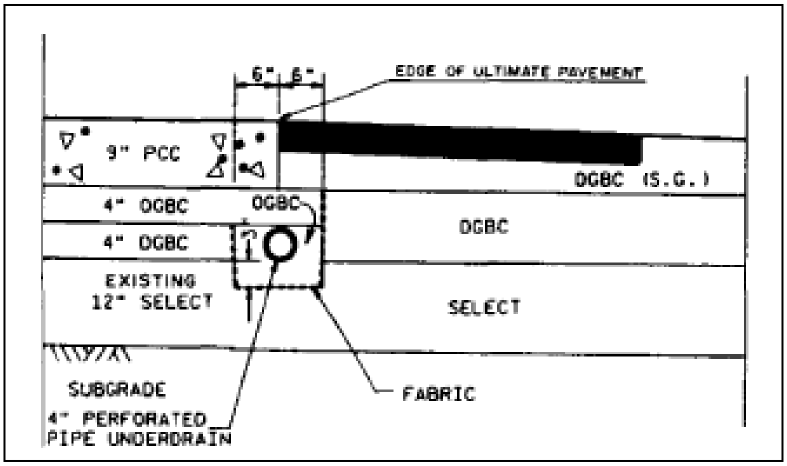

Figure B.1 USH 18/151 Sections 1 and 2 with Non-Stabilized OGBC

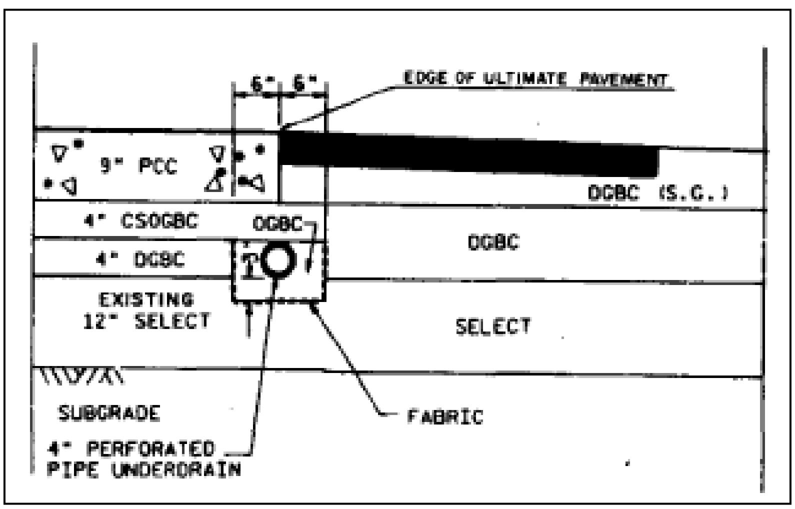

Figure B.2 USH 18/151 Sections 3, 4, and 11 with Cement-Stabilized OGBC

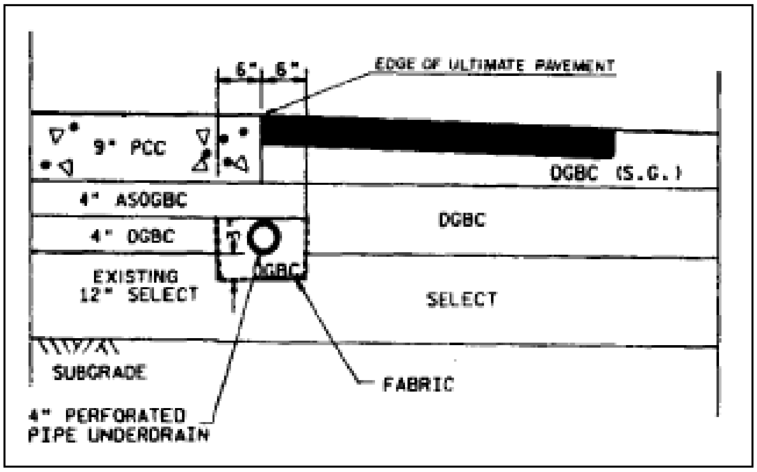

Figure B.3 USH 18/151 Sections 5, 6, and 12 with Asphalt-Stabilized OGBC

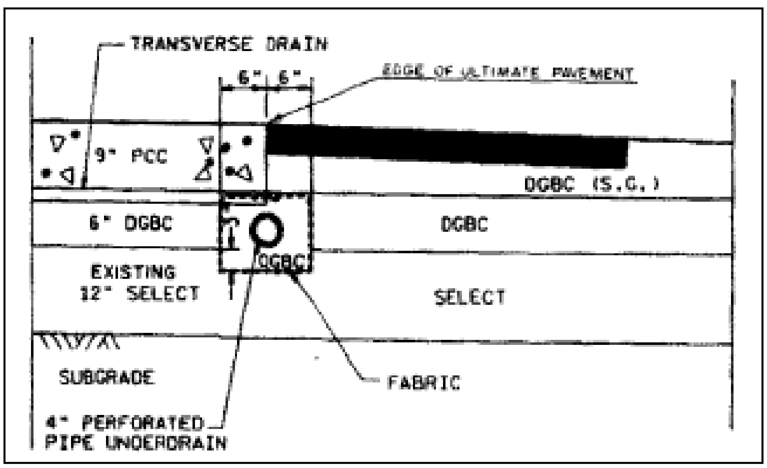

Figure B.4 USH 18/151 Sections 7 and 10 with TIC Drain

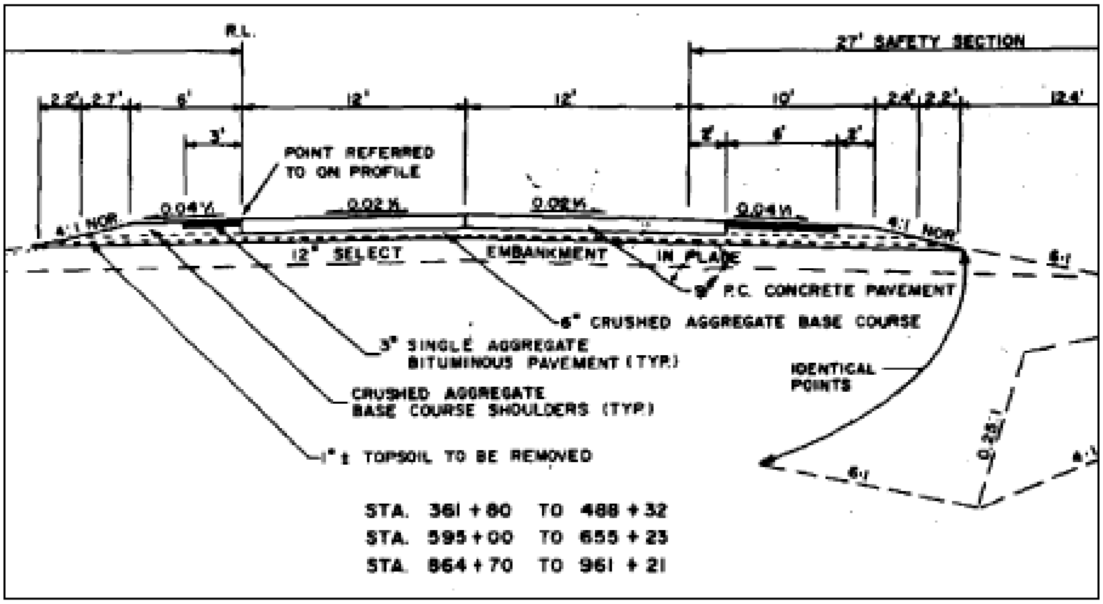

Figure B.5 USH 18/151 Sections 8, 9, 14 and 15 with Dense Graded Base Course

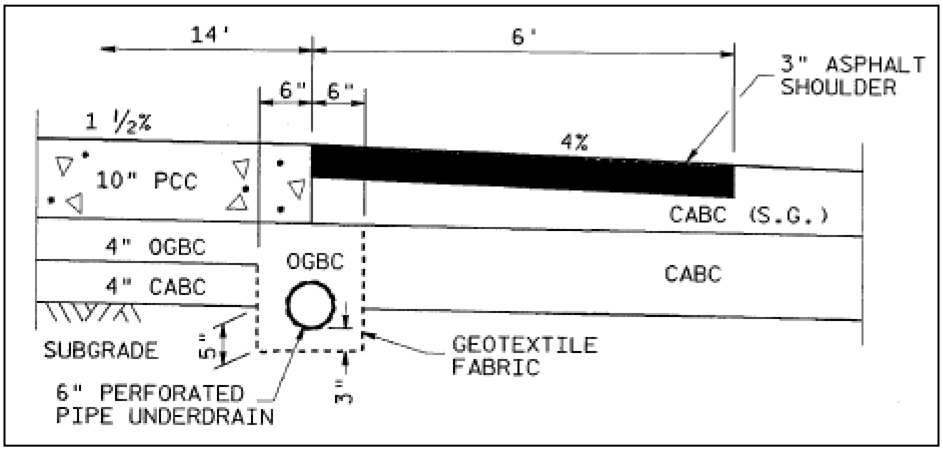

Figure B.6 STH 29 Sections 1 through 4 with Non-Stabilized OGBC

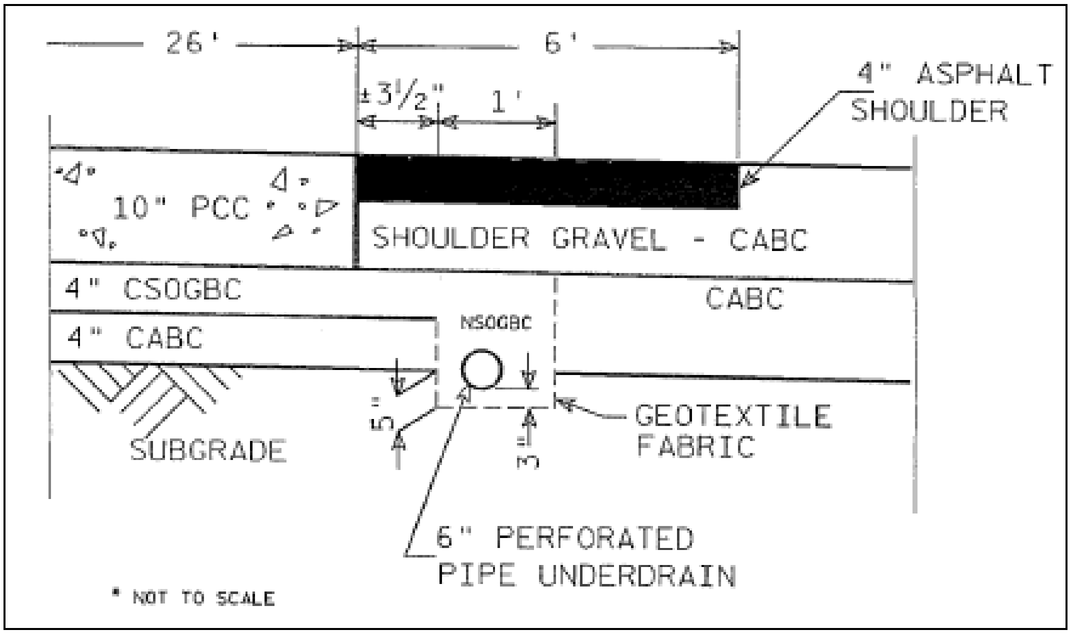

Figure B.7 USH 151 Section 1 with Cement-Stabilized OGBC

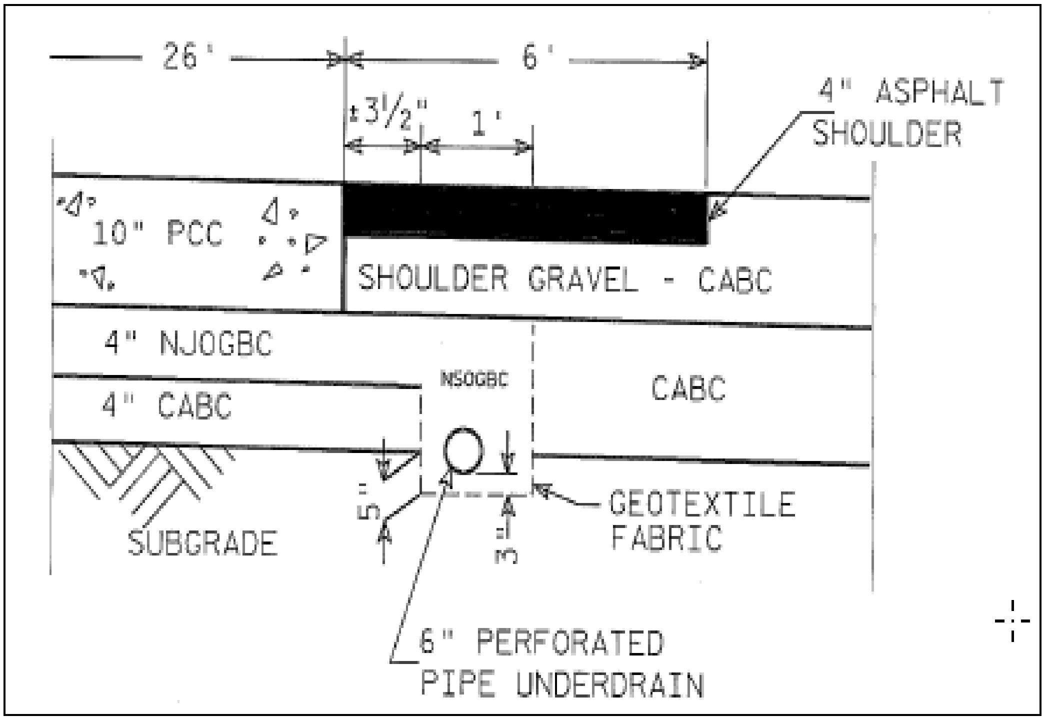

Figure B.8 USH 151 Section 2 with New Jersey Non-Stabilized OGBC

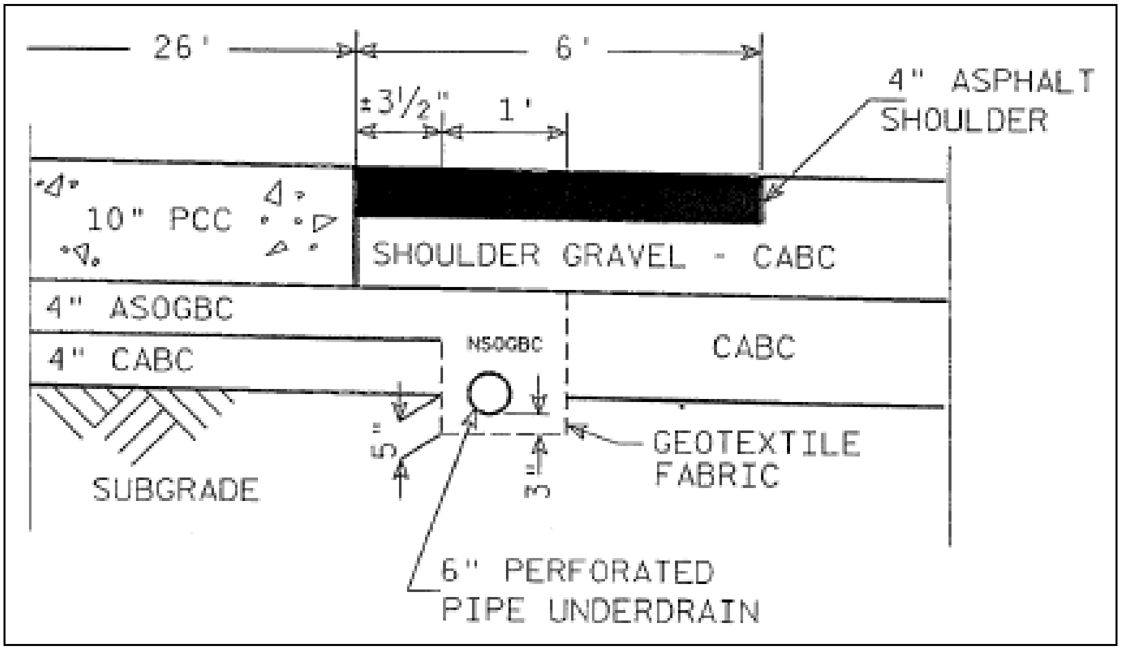

Figure B.9 USH 151 Section 3 with Asphalt-Stabilized OGBC

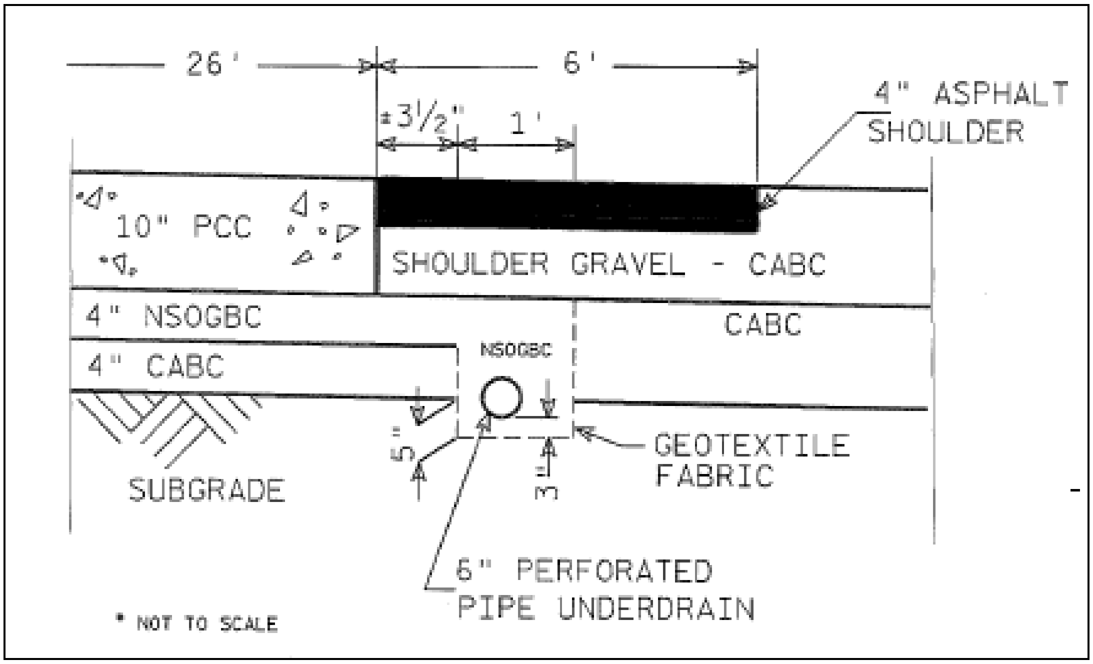

Figure B.10 USH 151 Section 4 with Non-Stabilized OGBC

Appendix C – Correlation Matrix

Pearson Correlation Coefficients, N = 25
Prob > |r| under H0: Rho=0

	Dowel1Y0N	B1D2O3C4A5T	Seal1Y0N	slbext	slbsev
Dowel1Y0N	1.00000	-0.02414 0.9088	-0.29244 0.1560	-0.24019 0.2475	0.02746 0.8963
B1D2O3C4A5T	-0.02414 0.9088	1.00000	-0.30890 0.1330	-0.40202 0.0464	-0.55587 0.0039
Seal1Y0N	-0.29244 0.1560	-0.30890 0.1330	1.00000	0.19302 0.3553	0.23681 0.2544
slbext	-0.24019 0.2475	-0.40202 0.0464	0.19302 0.3553	1.00000	0.68599 0.0002
slbsev	0.02746 0.8963	-0.55587 0.0039	0.23681 0.2544	0.68599 0.0002	1.00000
crkfill	0.29244 0.1560	0.30890 0.1330	-1.00000 <.0001	-0.19302 0.3553	-0.23681 0.2544
distjtckext	0.21246 0.3079	-0.03077 0.8839	-0.32733 0.1102	0.03402 0.8717	-0.14003 0.5044
distjtcksev	-0.68641 0.0002	0.00000 1.0000	0.21822 0.2947	0.06804 0.7466	-0.14003 0.5044
patchext	0.28332 0.1699	-0.17782 0.3951	-0.18389 0.3789	-0.04915 0.8155	0.20229 0.3322
patchsev	0.28332 0.1699	-0.17782 0.3951	-0.18389 0.3789	-0.04915 0.8155	0.20229 0.3322
surfdistext	-0.19872 0.3410	-0.09656 0.6461	0.11412 0.5870	0.24019 0.2475	0.14417 0.4917
surfdistsev	-0.19872 0.3410	-0.09656 0.6461	0.11412 0.5870	0.24019 0.2475	0.14417 0.4917
LJDistext	0.28205 0.1719	-0.15691 0.4538	-0.06419 0.7605	0.50707 0.0097	0.48745 0.0135
LJDistsev	0.21602 0.2997	-0.19864 0.3411	-0.12857 0.5402	0.53334 0.0060	0.46809 0.0183
TranFaultext	-0.48765 0.0134	0.10006 0.6341	0.22957 0.2696	0.31235 0.1285	0.32140 0.1172
TranFaultsev	-0.57761 0.0025	-0.13999 0.5045	0.22908 0.2707	0.21429 0.3037	0.40425 0.0450
PDI	-0.47114 0.0174	-0.35788 0.0790	0.14978 0.4748	0.66441 0.0003	0.44566 0.0256
IRIavg	-0.54759 0.0046	-0.40880 0.0425	0.21129 0.3106	0.59203 0.0018	0.39375 0.0515
Faulting	-0.82040 <.0001	-0.23649 0.2551	0.19689 0.3455	0.56411 0.0033	0.27566 0.1823

Pearson Correlation Coefficients, N = 25
Prob > |r| under H0: Rho=0

	crkfill	distjtckext	distjtcksev	patchext	patchsev
Dowel1Y0N	0.29244	0.21246	-0.68641	0.28332	0.28332
	0.1560	0.3079	0.0002	0.1699	0.1699
B1D2O3C4A5T	0.30890	-0.03077	0.00000	-0.17782	-0.17782
	0.1330	0.8839	1.0000	0.3951	0.3951
Seal1Y0N	-1.00000	-0.32733	0.21822	-0.18389	-0.18389
	<.0001	0.1102	0.2947	0.3789	0.3789
slbext	-0.19302	0.03402	0.06804	-0.04915	-0.04915
	0.3553	0.8717	0.7466	0.8155	0.8155
slbsev	-0.23681	-0.14003	-0.14003	0.20229	0.20229
	0.2544	0.5044	0.5044	0.3322	0.3322
crkfill	1.00000	0.32733	-0.21822	0.18389	0.18389
		0.1102	0.2947	0.3789	0.3789
distjtckext	0.32733	1.00000	0.16667	0.06019	0.06019
	0.1102		0.4259	0.7750	0.7750
distjtcksev	-0.21822	0.16667	1.00000	-0.24077	-0.24077
	0.2947	0.4259		0.2463	0.2463
patchext	0.18389	0.06019	-0.24077	1.00000	1.00000
	0.3789	0.7750	0.2463		<.0001
patchsev	0.18389	0.06019	-0.24077	1.00000	1.00000
	0.3789	0.7750	0.2463	<.0001	
surfdistext	-0.11412	-0.21246	-0.13074	0.30692	0.30692
	0.5870	0.3079	0.5333	0.1356	0.1356
surfdistsev	-0.11412	-0.21246	-0.13074	0.30692	0.30692
	0.5870	0.3079	0.5333	0.1356	0.1356
LJDistext	0.06419	0.19612	-0.29417	0.30692	0.30692
	0.7605	0.3475	0.1535	0.1356	0.1356
LJDistsev	0.12857	0.17931	-0.20493	0.20353	0.20353
	0.5402	0.3911	0.3258	0.3292	0.3292
TranFaultext	-0.22957	-0.10361	0.41442	0.14967	0.14967
	0.2696	0.6221	0.0394	0.4752	0.4752
TranFaultsev	-0.22908	-0.33535	0.46657	-0.04213	-0.04213
	0.2707	0.1013	0.0187	0.8415	0.8415
PDI	-0.14978	0.24390	0.56767	0.07639	0.07639
	0.4748	0.2400	0.0031	0.7166	0.7166
IRIavg	-0.21129	-0.16374	0.36403	-0.04634	-0.04634
	0.3106	0.4342	0.0736	0.8259	0.8259
Faulting	-0.19689	-0.03628	0.55524	-0.26092	-0.26092
	0.3455	0.8633	0.0040	0.2078	0.2078

Pearson Correlation Coefficients, N = 25
Prob > |r| under H0: Rho=0

	surfdistext	surfdistsev	LJDistext	LJDistsev	Tran Faultext
Dowel1Y0N	-0.19872 0.3410	-0.19872 0.3410	0.28205 0.1719	0.21602 0.2997	-0.48765 0.0134
B1D2O3C4A5T	-0.09656 0.6461	-0.09656 0.6461	-0.15691 0.4538	-0.19864 0.3411	0.10006 0.6341
Seal1Y0N	0.11412 0.5870	0.11412 0.5870	-0.06419 0.7605	-0.12857 0.5402	0.22957 0.2696
slbext	0.24019 0.2475	0.24019 0.2475	0.50707 0.0097	0.53334 0.0060	0.31235 0.1285
slbsev	0.14417 0.4917	0.14417 0.4917	0.48745 0.0135	0.46809 0.0183	0.32140 0.1172
crkfill	-0.11412 0.5870	-0.11412 0.5870	0.06419 0.7605	0.12857 0.5402	-0.22957 0.2696
distjtckext	-0.21246 0.3079	-0.21246 0.3079	0.19612 0.3475	0.17931 0.3911	-0.10361 0.6221
distjtcksev	-0.13074 0.5333	-0.13074 0.5333	-0.29417 0.1535	-0.20493 0.3258	0.41442 0.0394
patchext	0.30692 0.1356	0.30692 0.1356	0.30692 0.1356	0.20353 0.3292	0.14967 0.4752
patchsev	0.30692 0.1356	0.30692 0.1356	0.30692 0.1356	0.20353 0.3292	0.14967 0.4752
surfdistext	1.00000	1.00000 <.0001	0.19872 0.3410	0.28635 0.1652	0.25320 0.2220
surfdistsev	1.00000 <.0001	1.00000	0.19872 0.3410	0.28635 0.1652	0.25320 0.2220
LJDistext	0.19872 0.3410	0.19872 0.3410	1.00000	0.91431 <.0001	0.33135 0.1057
LJDistsev	0.28635 0.1652	0.28635 0.1652	0.91431 <.0001	1.00000	0.32337 0.1148
TranFaultext	0.25320 0.2220	0.25320 0.2220	0.33135 0.1057	0.32337 0.1148	1.00000
TranFaultsev	0.00572 0.9784	0.00572 0.9784	0.00572 0.9784	0.09860 0.6391	0.69999 <.0001
PDI	0.25949 0.2104	0.25949 0.2104	0.41823 0.0375	0.50955 0.0093	0.56519 0.0032
IRIavg	0.17819 0.3941	0.17819 0.3941	-0.08515 0.6857	-0.02912 0.8901	0.21888 0.2932
Faulting	0.12436 0.5537	0.12436 0.5537	-0.12312 0.5577	-0.08631 0.6817	0.40761 0.0431

Pearson Correlation Coefficients, N = 25
Prob > |r| under H0: Rho=0

	Tran Faultsev	PDI	IRIavg	Faulting
Dowel1Y0N	-0.57761	-0.47114	-0.54759	-0.82040
	0.0025	0.0174	0.0046	<.0001
B1D2O3C4A5T	-0.13999	-0.35788	-0.40880	-0.23649
	0.5045	0.0790	0.0425	0.2551
Seal1Y0N	0.22908	0.14978	0.21129	0.19689
	0.2707	0.4748	0.3106	0.3455
slbext	0.21429	0.66441	0.59203	0.56411
	0.3037	0.0003	0.0018	0.0033
slbsev	0.40425	0.44566	0.39375	0.27566
	0.0450	0.0256	0.0515	0.1823
crkfill	-0.22908	-0.14978	-0.21129	-0.19689
	0.2707	0.4748	0.3106	0.3455
distjtckext	-0.33535	0.24390	-0.16374	-0.03628
	0.1013	0.2400	0.4342	0.8633
distjtcksev	0.46657	0.56767	0.36403	0.55524
	0.0187	0.0031	0.0736	0.0040
patchext	-0.04213	0.07639	-0.04634	-0.26092
	0.8415	0.7166	0.8259	0.2078
patchsev	-0.04213	0.07639	-0.04634	-0.26092
	0.8415	0.7166	0.8259	0.2078
surfdistext	0.00572	0.25949	0.17819	0.12436
	0.9784	0.2104	0.3941	0.5537
surfdistsev	0.00572	0.25949	0.17819	0.12436
	0.9784	0.2104	0.3941	0.5537
LJDistext	0.00572	0.41823	-0.08515	-0.12312
	0.9784	0.0375	0.6857	0.5577
LJDistsev	0.09860	0.50955	-0.02912	-0.08631
	0.6391	0.0093	0.8901	0.6817
TranFaultext	0.69999	0.56519	0.21888	0.40761
	<.0001	0.0032	0.2932	0.0431
TranFaultsev	1.00000	0.52795	0.47352	0.54976
		0.0067	0.0168	0.0044
PDI	0.52795	1.00000	0.54863	0.65873
	0.0067		0.0045	0.0003
IRIavg	0.47352	0.54863	1.00000	0.74595
	0.0168	0.0045		<.0001
Faulting	0.54976	0.65873	0.74595	1.00000
	0.0044	0.0003	<.0001	

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