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Cost Effective Concrete Pavement Cross Sections - Final Report

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COST EFFECTIVE CONCRETE PAVEMENT CROSS SECTIONS

FINAL REPORT

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16. Abstract

This report presents the findings of a study of 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 was collected out to 5 and 7 years after construction.

The study results indicate that 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 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.

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FINAL REPORT WI/SPR-03-05
WisDOT Highway Research Study # 95-03

by

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for

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DIVISION OF TRANSPORTATION SYSTEM DEVELOPMENT
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CHAPTER 1
INTRODUCTION

1.0 Introduction

This report presents details relating to the design, construction, and performance of concrete pavement test sections constructed in the State of Wisconsin along WIS 29 in Clark, Marathon and Shawano Counties. These test sections were constructed during the summers of 1997 and 1999 to validate the constructability and performance of cost-effective alternative concrete pavement designs incorporating variable dowel bar placements, dowel bar materials, slab thicknesses, and drainage details.

Chapter 1 of this report provides project background information. Results of laboratory tests conducted on test specimens fabricated prior to construction are provided in Chapter 2. Details on the construction of each test section are provided in Chapter 3. Chapter 4 provides the results of performance testing conducted immediately after construction and over the study duration of each test section. Chapter 5 provides an analysis of initial construction costs for the various test sections. A summary of all research results and recommendations for further research is provided in Chapter 6.

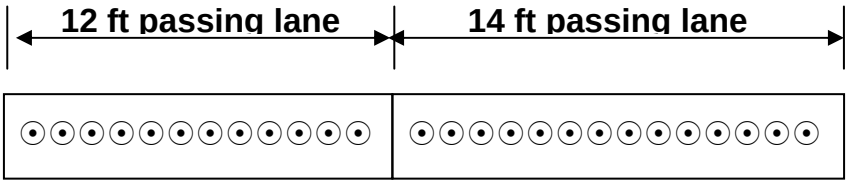
1.1 Project Background

The present pavement selection policy of the Wisconsin Department of Transportation (WisDOT), as provided in Procedure 14-10-10 of the Facilities Development Manual, limits the design alternatives for Portland cement concrete (PCC) pavements and inhibits the designer's ability to select cross-sections deviating from uniform slab thicknesses with doweled transverse joints. Currently, uniform slab thicknesses and conventional joint load transfer devices are incorporated into the design based on the heavy truck traffic in the driving lane. While this strategy provides for adequate pavement structure in this truck lane to limit faulting and slab cracking to tolerable levels, there is a potential for over-design in other traffic lanes which may experience significantly lower Equivalent Single Axle Load (ESAL) applications over the service life of the pavement. Pavement design analyses were conducted to investigate the effects of variable slab thickness within and/or across traffic lanes, variable load transfer designs, and alternative

base layer drainage designs.

Four alternate dowel patterns were developed to reduce the number of dowel bars installed across transverse pavement joints. These patterns were developed with the constraint that dowel positions had to be consistent with dowel bar insertion (DBI) equipment currently used within the State of Wisconsin. This constraint allowed for the removal of certain dowels but did not allow for any repositioning of dowels, i.e., the 12-inch center-to-center placement openings could not be changed. A minimum of three dowels per wheel path was established and used for one alternate to provide marginal load transfer capacity across the transverse joints of both travel lanes. Additional dowels were positioned within the outer wheel path of the driving lane and/or near the slab edge to increase the load transfer capacity of these critical pavement locations. This selection strategy resulted in four dowel placement alternates which are illustrated in Figure 1.1.1.

In addition to the dowel placement alternates, test sections were also constructed using alternative dowel materials which may be considered as corrosion resistant, including fiber reinforced polymer (FRP) composite dowels, solid stainless steel dowels, and hollow-core, mortar-filled (hollow-filled) stainless steel dowels. Variable thickness slab designs were also developed in an effort to reduce the initial paving costs while maintaining the constructability of the pavement structure. Two trapezoidal PCC slab cross-sections were developed, each with the fully-reduced slab thickness coincident with the median edge of the pavement. For one design alternate, the reduced median-edge slab thickness increases linearly to the full design thickness at the center-lane joint, resulting in a trapezoidal passing lane and full thickness driving lane. For the second design alternate, the reduced median-edge slab thickness increases linearly across both lanes. For the variable slab thickness designs, the passing lane width was increased to 15 ft (striped at 12 ft) to minimize the potential for extreme edge loadings along the thinnest portion of the slab. Figure 1.1.2 provides illustrations of the trapezoidal slab thickness designs.



Standard placement – 12 inch c-c spacing, 26 dowels per joint

Alternate 1- 3 dowels in each wheel path at 12 in c-c spacing,
12 dowels per joint

Alternate 2 - 4 dowels in outer wheel path and 3 dowels in other wheel paths,
12 in c-c spacing, 13 dowels per joint

Alternate 3 - 4 dowels in outer wheel path and 3 dowels in other wheel paths,
12 in c-c spacing, 1 dowel at outer edge, 14 dowels per joint

Alternate 4 - 3 dowels in each wheel path at 12 in c-c spacing,
1 dowel near outer edge, 13 dowels per joint

⊙ Dowel Location ⊗ Removed Dowel

Figure 1.1.1 Standard and Alternate Dowel Placements

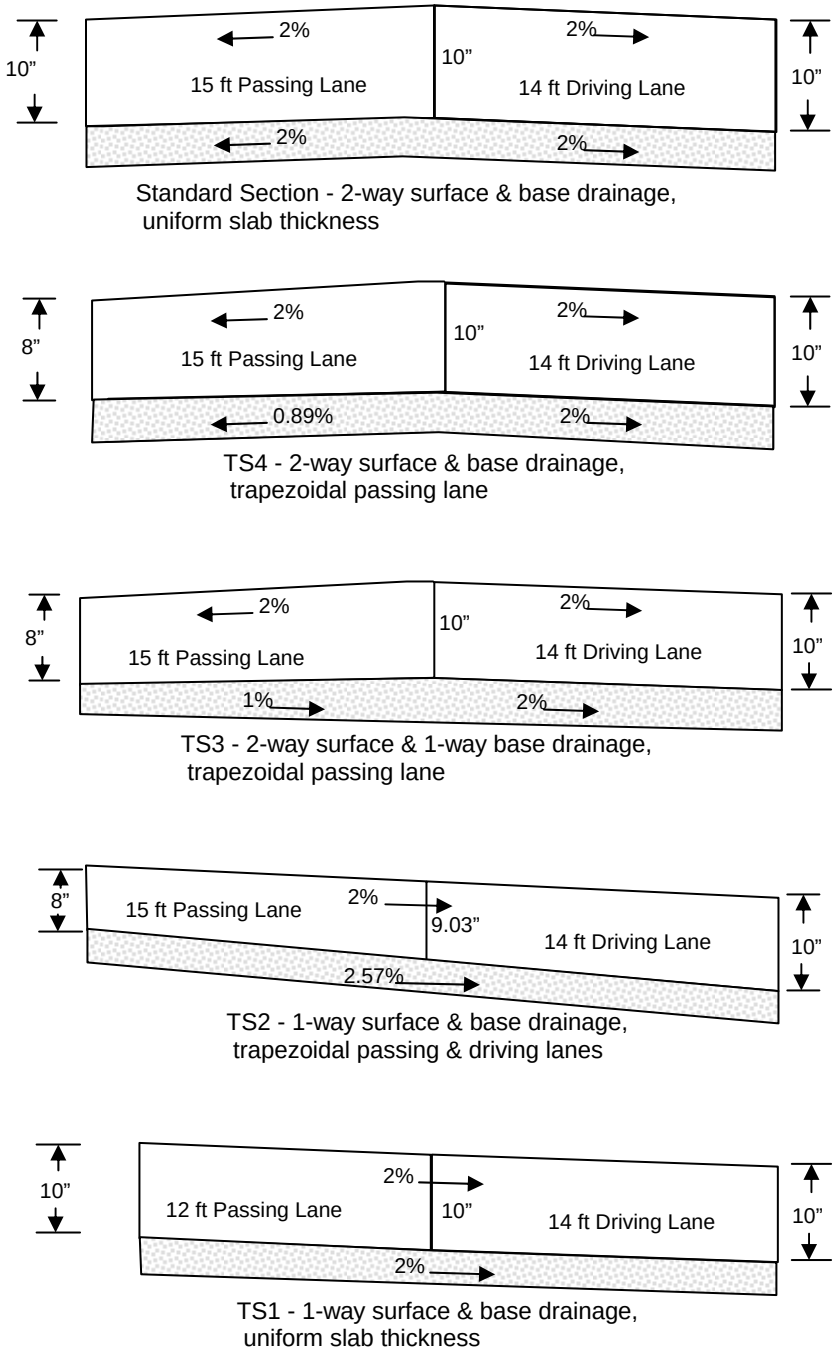


Figure 1.1.2: Variable Slab Thickness and Drainage Designs (not to scale)

Alternative subsurface drainage layer designs were also developed in an effort to reduce initial paving costs. The primary focus of these designs was to eliminate the median side drainage details for typical tangent pavement sections, including the removal of the longitudinal drainage trench/pipe and the transverse pipe/outlets. This focus was expanded to include alternate surface drainage designs and variable slab thicknesses, resulting in four separate design alternates as illustrated by Test Sections (TS) 1, 2, 3 and 4 in Figure 1.1.2. Note that TS 1 represents a tangent pavement section incorporating the typical design details of a super-elevated pavement section.

1.2 Test Section Description

Ten test sections incorporating all four alternative dowel patterns and all of the alternate dowel materials were constructed in 1997 within the eastbound lanes of WIS 29 in Clark County between Owen and Abbotsford, herein referred to as WIS 29 Abbotsford.

Test sections incorporating alternative dowel placements, alternate dowel materials and variable slab thicknesses were constructed in 1997 within the eastbound and westbound lanes of WIS 29 in Marathon County between Hatley and Wittenberg, herein referred to as WIS 29 Wittenberg. Three test sections constructed along the eastbound lanes of WIS 29 Wittenberg incorporated FRP composite and solid stainless steel dowel bars. One test section incorporating variable slab thickness, and another incorporating placement alternate 1 with standard epoxy coated steel dowels, were constructed within the westbound lanes of WIS 29 Wittenberg. All test sections constructed on WIS 29 Wittenberg are designated Strategic Highway Research Program (SHRP) test sections.

Test sections incorporating variable slab thicknesses and non-traditional surface and/or base layer drainage details, including one-way surface and/or one-way base drainage, were constructed in 1999 within the westbound lanes of WIS 29 in Shawano County between Tilleda and Wittenberg, herein referred to as WIS 29 Tilleda. WIS 29 Tilleda test sections with variable slab thickness were constructed with a passing lane width of 15 ft. A test section incorporating one-way surface and one-way base drainage with a constant slab thickness was also constructed within the westbound lanes of WIS 29 Tilleda.

Descriptions of all test section design details, including test section codes utilized in this report as well as SHRP test section designations, where applicable, are provided in Tables 1.2.1 through 1.2.3. Appendix A provides location maps for all constructed test sections.

Table 1.2.1 WIS 29 Abbotsford Test Section Design Details

Description	Report Code
11-inch doweled JPCP, placement alternate 1 using standard epoxy coated dowels (3 dowels in each wheelpath, 12 per joint)	1E
11-inch doweled JPCP, placement alternate 2 using standard epoxy coated dowels (4 dowels in outer wheelpath of driving lane, 3 in other wheelpaths, 13 per joint)	2E
11-inch doweled JPCP, placement alternate 3 using standard epoxy coated dowels (4 dowels in outer wheelpath of driving lane, 3 in other wheelpaths, one at slab edge, 14 per joint)	3E
11-inch doweled JPCP, placement alternate 3 using solid stainless steel dowels supplied by Avesta Sheffield (4 dowels in outer wheelpath of driving lane, 3 in other wheelpaths, one at slab edge, 14 per joint)	3S
11-inch doweled JPCP, placement alternate 4 using standard epoxy coated dowels (3 dowels in each wheelpath, one near edge, 13 per joint)	4E
11-inch doweled JPCP, placement alternate 4 using solid stainless steel dowels supplied by Avesta Sheffield (3 dowels in each wheelpath, one near edge, 13 per joint)	4S
11-inch doweled JPCP, standard dowel placement using FRP composite dowels supplied by Creative Pultrusions (26 per joint)	CP
11-inch doweled JPCP, standard dowel placement using FRP composite dowels supplied by Glasforms (26 per joint)	GF
11-inch doweled JPCP, standard dowel placement using FRP composite dowels supplied by RJD (26 per joint)	RJD
11-inch doweled JPCP, standard dowel placement using hollow-core, mortar-filled stainless steel dowels supplied by Damascus-Bishop Tube Company (26 per joint)	HF
11-inch doweled JPCP, standard dowel placement (Control) using standard epoxy coated dowels (26 per joint)	C1, C2

Table 1.2.2 WIS 29 Wittenberg Test Section Design Details

Description	Report Code	SHRP Code
11-inch doweled JPCP, dowel placement alternate 1 using epoxy coated dowels (3 dowels in each wheelpath,12 per joint)	1E	550260
11-inch doweled JPCP, standard dowel placement using FRP composite dowels supplied by MMFG, Glasforms, and Creative Pultrusions (26 per joint)	FR	550264
11-inch doweled JPCP, standard dowel placement using FRP composite dowels supplied by RJD (26 per joint)	RJD	550266
11-inch doweled JPCP, standard dowel placement using solid stainless steel dowels supplied by Slater Steels (26 per joint)	SS	550265
8 - 11-inch doweled JPCP, variable thickness across both lanes, standard dowel placement using epoxy coated dowels (26 per joint)	TR	550263
11-inch doweled JPCP, standard dowel placement (Control) using epoxy coated steel dowels (26 per joint)	C1, C2, C3	550259 (C3)

Table 1.2.3 WIS 29 Tilleda Test Section Design Details

Description	Report Code
Doweled JPCP, variable passing lane slab thickness (8 – 10 inches), widened passing lane (15 ft), two-way surface drainage (2%), two-way base layer drainage with passing lane base slope reduced from 2% to 0.89%, uniform drainage layer thickness (4-inch)	TS4
Doweled JPCP, variable passing lane slab thickness (8 – 10 inches), widened passing lane (15 ft), variable passing lane drainage layer thickness (4 – 7.3 inches) and uniform driving lane drainage layer thickness (7.3 inches), two-way surface drainage (2%), one-way base layer drainage, passing lane base slope reduced from 2% to1%, no inside shoulder edge drain	TS3
Doweled JPCP, uniform slab thickness (10-inch), widened passing lane (15 ft), uniform drainage layer thickness (4-inch), two-way surface and base layer drainage (2%)	STD
Doweled JPCP, variable pavement thickness across both lanes (8 – 10 inches), one-way surface drainage (2%), one-way base layer drainage (2.57%), uniform drainage layer thickness (4-inch), no inside shoulder edge drain	TS2
Doweled JPCP, uniform pavement thickness across both lanes (10 inches), one-way surface drainage and base layer drainage (2%), uniform drainage layer thickness (4-inch), no inside shoulder edge drain	TS1

CHAPTER 2
LABORATORY TESTS

2.1 Introduction

Laboratory testing, including joint deflection tests and dowel bar pull-out tests, were conducted at Marquette University to investigate the behavior of doweled joints under various loading conditions. Initial tests were conducted prior to pavement construction using sample dowels provided by the manufacturers. Additional tests were conducted using dowels obtained during the construction of WIS 29 Abbotsford.

2.2 Load-Deflection Tests

Load-defection tests were conducted in accordance with AASHTO Designation T 253-76 (1993), *Standard Method of Test for Coated Dowel Bars*. These tests provide an indication of the load transfer capacity of the dowels under extreme loading conditions. The transverse joint is simulated as a wide crack with no available aggregate interlock across the joint (no shear transfer across joint faces) and the loaded slab is fully unsupported. While these conditions are not likely to occur under normal service loading, they do serve to isolate the contribution of the dowel in transferring load between adjacent slabs. Under normal service conditions, this contribution reduces slab edge and corner deflections under loading and reduces the potential for slab faulting, corner cracking, and/or base pumping.

Rectangular test specimens, 12 inches wide by 11 inches deep by 48 inches long were constructed using paving grade concrete supplied by Tews Company. Two full-depth joints, each 3/8 inches wide, were formed 12 inches from each specimen end using wood inserts. Centered holes on each insert allowed for the placement of an 18-inch long dowel bar (1.5 inch diameter) across each joint. Dowel bars were positioned at the mid-depth of the test specimens. Figure 2.2.1 provides a schematic illustration of the fabricated specimens.

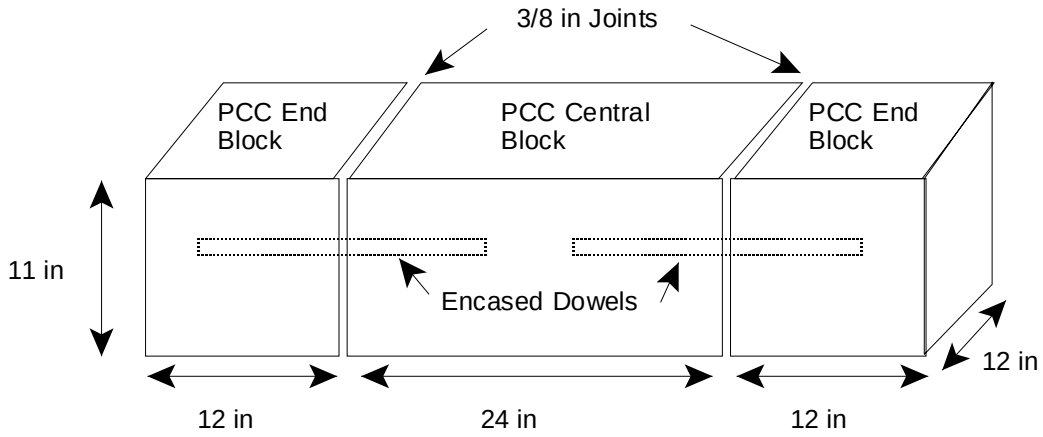


Figure 2.2.1: Schematic Illustration of Joint Deflection Test Specimen

Test specimens were fabricated with the various dowel bar materials envisioned for construction, including standard epoxy coated steel (control), polished solid stainless steel, and three types of composite dowels as manufactured by MMFG, Creative Pultrusions, and Glasforms. Cast specimens were cured for 21 days prior to the start of testing. The specimen ends were then placed on neoprene capped steel support pedestals and clamped to restrict rotation during loading. The formed joints were positioned approximately ½ inches inwards from the edge of the support pedestals to allow for the placement of a linear variable displacement transducer (LVDT) on the underside of each end to monitor displacement during loading. LVDTs were also positioned on the underside of the central (loaded) portion of the specimen to monitor displacement.

The test load was applied using a manually actuated ENERPAC hydraulic ram mounted on a steel reaction frame. The load ram was centered on the test specimen. Steel plates and arched steel blocks were positioned over the central portion of the specimen to distribute the load uniformly across the center section of the specimen. Four load cells were positioned near the corners of the arched steel block to monitor the applied load. Load cell and LVDT data were collected with a Datronic data collection system using a 2 Hz sampling rate. The load was increased at a rate of approximately 2000 lb/min until a maximum of 5000 lb was obtained. Figure 2.2.2 provides a photo of the test set-up during loading.

Figure 2.2.2: Joint Deflection Test Set-up

The maximum relative joint deflections, recorded at a load of 4,000 lb, are provided in Table 2.2.1 and Figure 2.2.3. Figures 2.2.4 to 2.2.8 provide plots of the collected test data. AASHTO T 253 test protocol stipulates a maximum relative joint deflection of 0.01 inches at a test load of 4,000 lb. As shown in Table 2.2.1 and the figures provided, all test results, with the exception of the Glasforms specimen, met this criterion. Furthermore, the composite dowel specimens exhibited higher relative joint deflections as compared to the epoxy coated and solid stainless steel dowels, which may indicate the potential for lower load transfer for in-service pavements constructed with composite dowels of this type.

Table 2.2.1: Summary of Joint Deflection Test Results

Dowel Type	Dowel Diameter (inch)	Relative Joint Deflection, inches		
		Joint 1	Joint 2	Average
Epoxy Coated	1.52	0.006	0.008	0.0070
Stainless Steel	1.50	0.006	0.006	0.0060
Glasforms	1.50	0.013	0.016	0.0145
Creative Pultrusions	1.50	0.009	0.010	0.0095
MMFG	1.49	0.008	0.007	0.0075

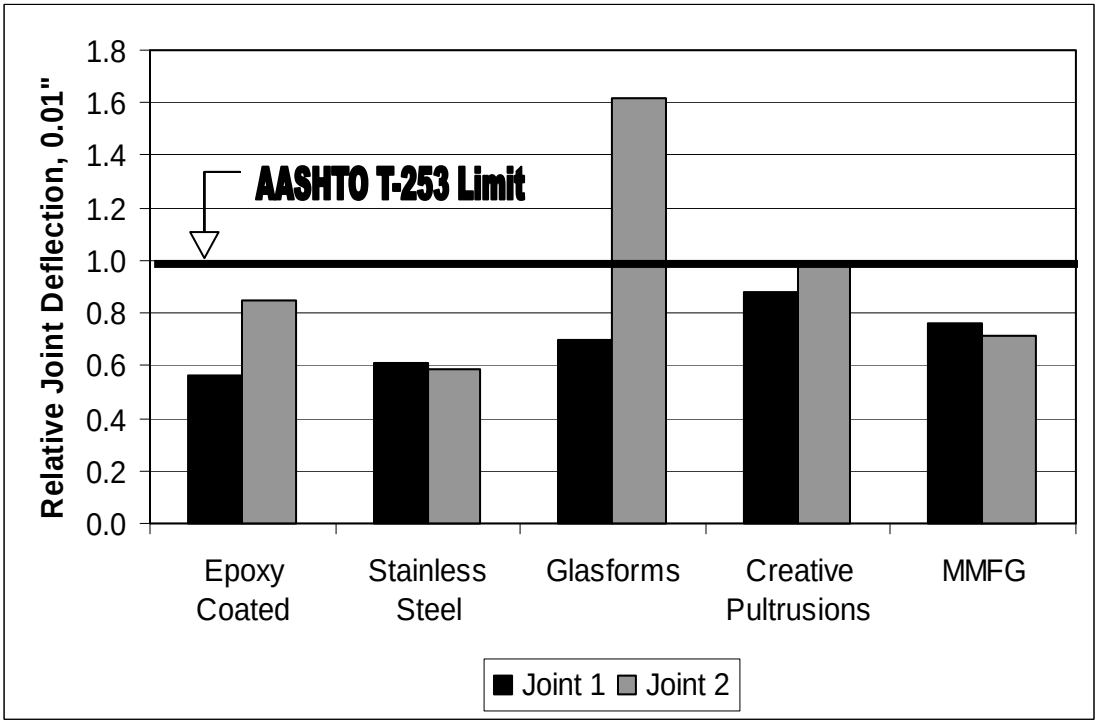


Figure 2.2.3: Joint Deflection Test Results

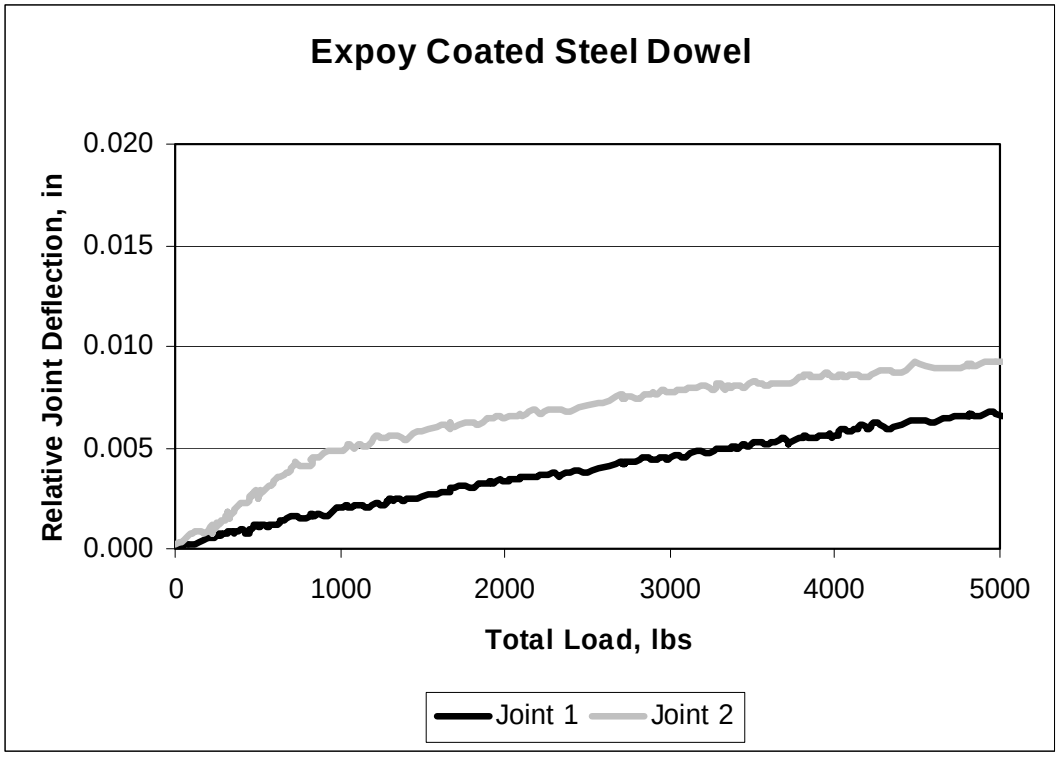


Figure 2.2.4: Test Results for the Epoxy Coated Steel Dowels

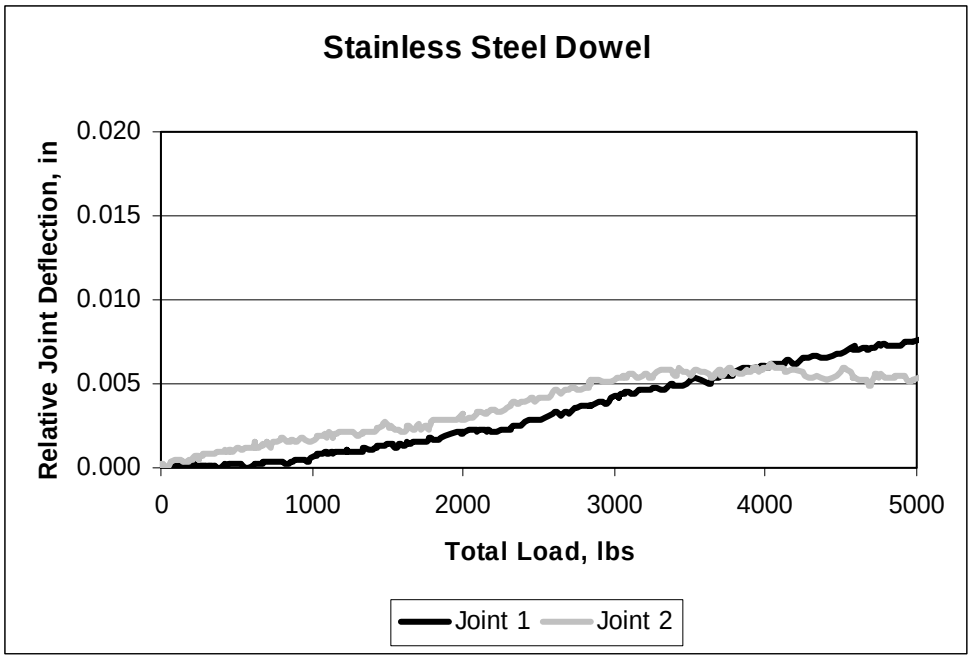


Figure 2.2.5: Test Results for the Solid Stainless Steel Dowels

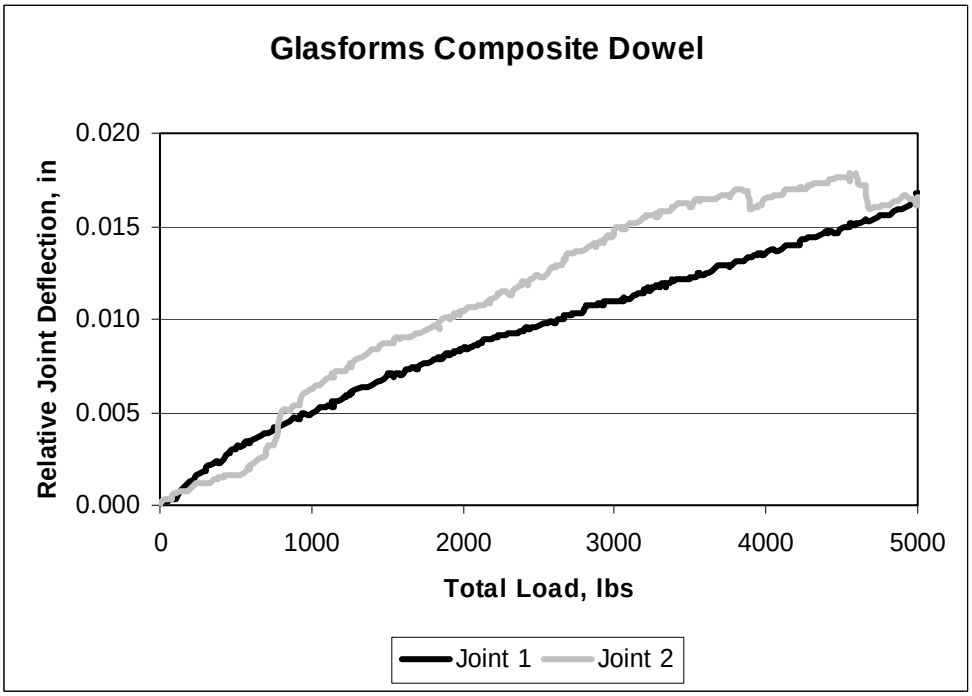


Figure 2.2.6: Test Results for the Glasforms Composite Dowels

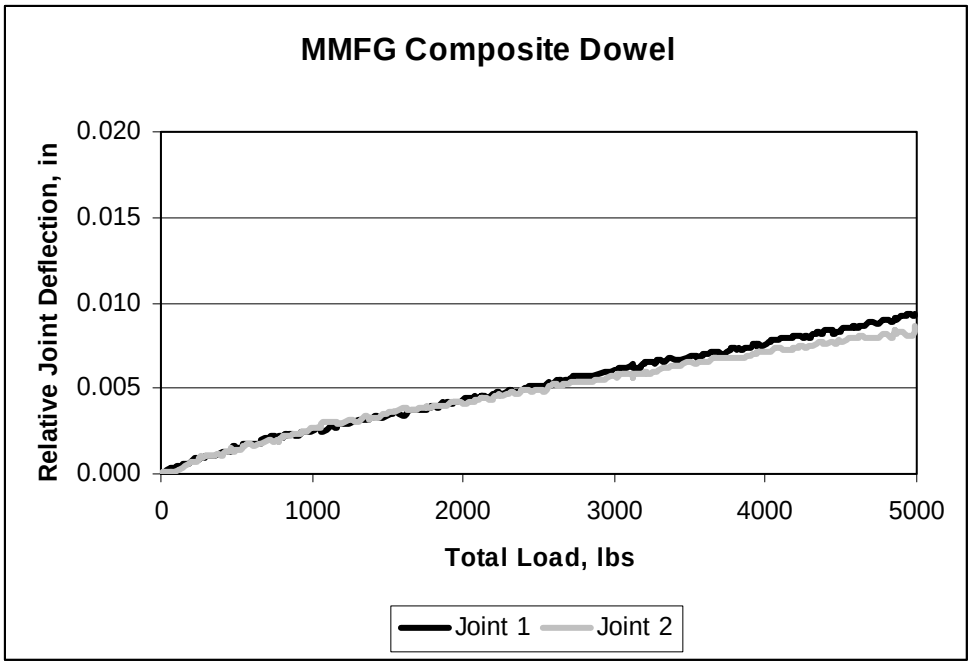


Figure 2.2.7: Test Results for the MMFG Composite Dowels

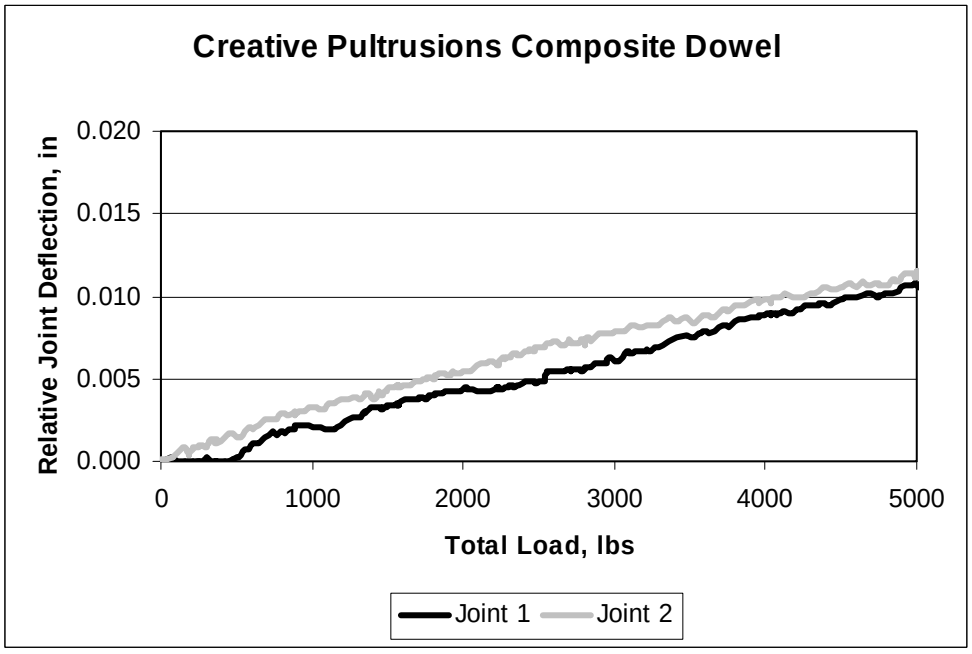


Figure 2.2.8: Test Results for the Creative Pultrusions Composite Dowels

2.3 Pull-Out Tests – Non-oiled Dowels

Dowel bar pull-out tests were conducted in accordance with AASHTO Designation T 253-76 (1993), *Standard Method of Test for Coated Dowel Bars*. Rectangular test specimens, 6 inches x 6 inches x 18 inches were cast in wooden forms using paving grade concrete supplied by Tews Company. Dowel bars were positioned at the center of the 6 x 6-inch face, extending approximately 9 inches into the concrete beam. Figure 2.3.1 provides a schematic illustration of the fabricated specimens.

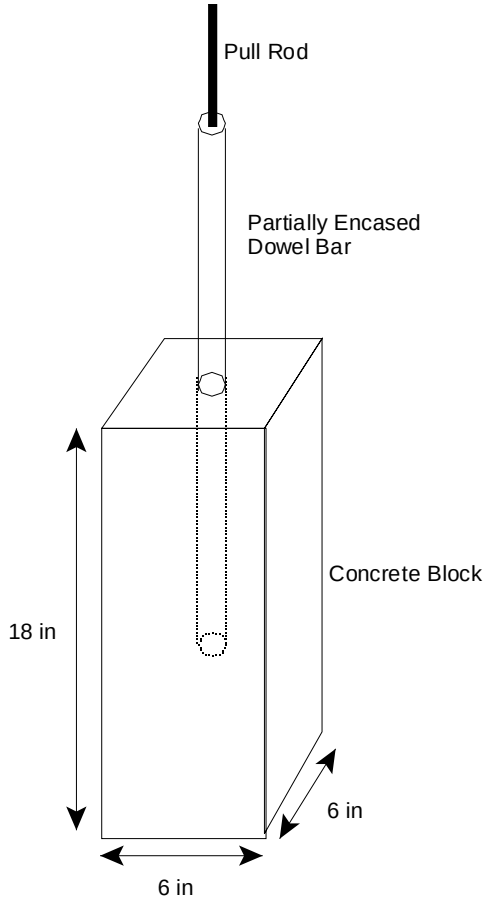


Figure 2.3.1: Schematic Illustration of Pull-Out Specimen

Pull-out tests were conducted prior to construction with non-oiled dowels supplied by the manufacturers, including a standard epoxy coated steel bar (control), a polished solid stainless steel bar, a brushed stainless steel bar, and three composite dowels as manufactured by MMFG, Creative Pultrusions, and Glasforms. Cast specimens were cured for 48 hours prior to the start of testing. Holes were drilled into the exposed ends of the dowels to allow for the placement of a steel pull rod. Pull rods were threaded into the steel dowels and epoxied into the composite dowels.

The pull-out specimens were mounted into a Riehle compression machine and the pull rod was placed through the upper stationary head and capped. A dial gauge was mounted onto the dowel with the indicator rod resting on the movable crosshead to monitor relative displacements between the dowel and the moveable crosshead. Corresponding pull-out loads were manually recorded off the digital display of the Riehle compression machine. Figure 2.3.2 provides a photo of the pull-out test set-up.

Figure 2.3.2 Pull-Out Test Set-up

Tests were conducted using a crosshead movement rate of 0.03 in/min. This movement slowly pushed the concrete block away from the restrained dowel. Load readings were recorded for every 0.005 inches of relative dowel/concrete displacement, to a total relative displacement of 0.05 inches. Additional readings were taken for every 0.05 inches of relative displacement to a total relative displacement of 0.5 inches.

The maximum pull-out loads and calculated maximum pull-out stresses are provided in Table 2.3.1. Maximum pull-out stresses were calculated based on maximum pull-out loads divided by the circumferential contact area between the dowel and the concrete at the start of testing. The maximum pull-out load for the steel dowels (epoxy coated, brushed stainless steel, polished stainless steel) typically occurred during the initial 0.05 inches of relative displacement and then reduced significantly to a residual load level. The roughened surface on the brushed stainless steel dowel resulted in a maximum pull-out load which was 44% greater than the epoxy coated dowel whereas the maximum pull-out load for the polished stainless steel dowel was approximately 39% lower than the epoxy coated dowel.

Table 2.3.1: Summary of Pull-Out Tests on Non-Oiled Dowels

Dowel Bar Type	Maximum Pull-Out Load, lb	Circumferential Contact Area, in^2	Maximum Pull-Out Stress, psi
Epoxy Coated	4000	43.0	93
Polished Stainless Steel	2420	42.8	57
Brushed Stainless Steel	5725	42.7	134
Glasforms	430	43.3	10
Creative Pultrusions	155	41.7	4
MMFG	640	40.8	16

The maximum pull-out load for the composite dowels generally occurred within the initial 0.05 inches of relative dowel displacement. Unlike the steel dowels, the residual loads thereafter did not reduce significantly from the maximum value; however, the maximum pull-out loads for all composite dowels tested were significantly reduced as compared to the steel dowels.

2.4 Pull-Out Tests - Oiled Dowels

Pull-out tests were also conducted using the six different 1.5-inch nominal diameter dowel types obtained during construction on WIS 29 Abbotsford, including the standard epoxy coated steel dowels (control), polished solid stainless steel, polished hollow-core stainless steel (grout filled), and composite dowels as manufactured by RJD, Creative Pultrusions, and Glasforms. Rectangular test specimens, 6 inches x 6 inches x 12 inches were cast in a specially fabricated steel form using fly ash concrete produced in the Marquette lab. The mixture was proportioned according to the job mix used during construction on WIS 29 Abbotsford. All dowel bars were oiled prior to casting using form oil obtained during pavement construction. The dowels were positioned such that the dowel would extend 9 inches into the beam at the center of the 6 inch x 6 inch face.

Initial pull-out tests were conducted after 48 hours of concrete curing. The test specimens were then cured an additional 12 days prior to subjecting to 50 cycles of freeze-thaw in a 10% by mass sodium chloride solution. After freeze-thaw conditioning, a second pull-out test was conducted. During both test series, the data recording apparatus was modified from the initial apparatus used in the uncoated tests to allow for continuous data collection during the test. The modified apparatus utilized four load cells and two LVDTs for monitoring load and relative dowel displacement, respectively. Load cell and LVDT data were collected with a Strawberry Tree data collection system set at a 5 Hz sampling rate. Figure 2.4.1 illustrates the modified test set-up.

Figure 2.4.1: Modified Pull-Out Test Set-Up

The maximum pull-out loads and calculated maximum pull-out stresses and residual pull-out stresses for the pre-freeze thaw tests are provided in Table 2.4.1. Table 2.4.2 provides maximum values for the post-freeze thaw testing. Maximum pull-out stresses were again calculated based on maximum pull-out loads divided by the circumferential contact area between the dowel and the concrete at the start of each series of testing. Figure 2.4.2 illustrates a summary of the maximum pull-out stresses for all tests as well as the residual pull-out stress for the pre-freeze thaw testing. Figures 2.4.3 to 2.4.8 illustrate the pull-out stress trends for all tested dowels.

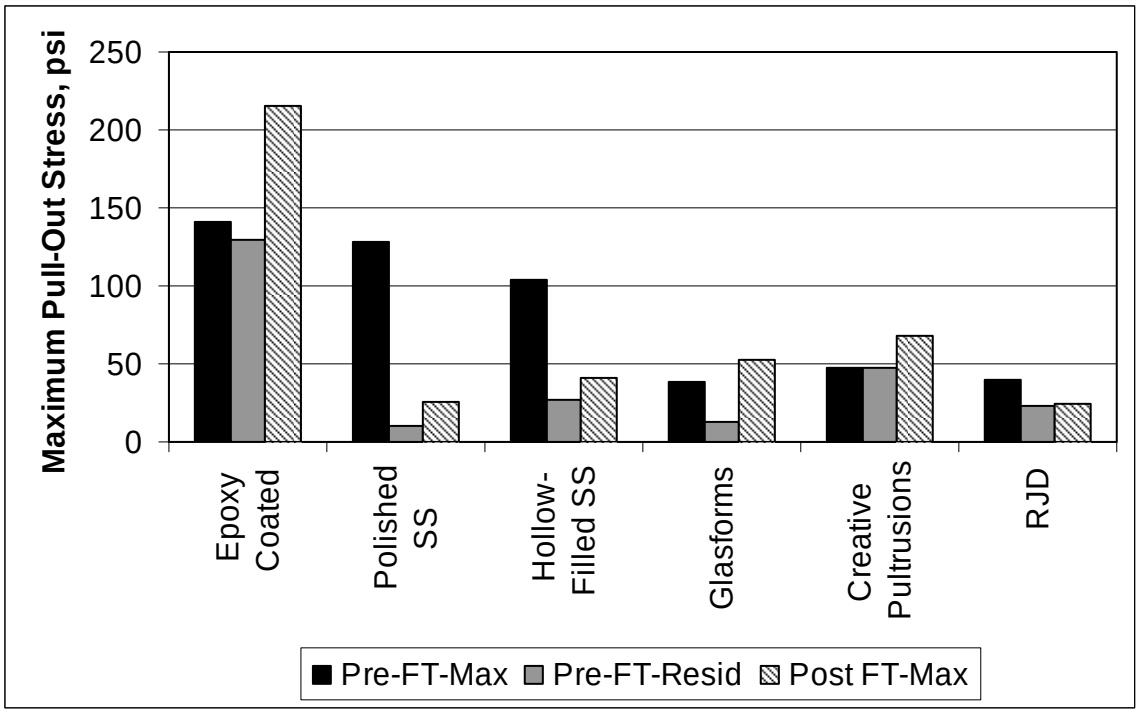


Figure 2.4.2: Summary of Pull-Out Test Results

Table 2.4.1: Summary of Pre-Freeze Thaw Pull-Out Tests on Oiled Dowels

Dowel Bar Type	Maximum Pull-Out Load lb	Circumferential Contact Area in^2	Maximum Pull-Out Stress psi	Residual Pull-Out Stress psi
Epoxy Coated	5876	41.6	141	130
Polished Stainless Steel	5159	40.3	128	10
Hollow-Filled Stainless Steel	4576	43.8	104	27
Glasforms	1604	41.2	38	13
Creative Pultrusions	1943	41.3	46	48
RJD	1694	42.4	40	23

Table 2.4.2: Summary of Post-Freeze Thaw Pull-Out Tests on Oiled Dowels

Dowel Bar Type	Maximum Pull-Out Load lb	Circumferential Contact Area in^2	Maximum Pull-Out Stress psi
Epoxy Coated	8493	39.4	216
Polished Stainless Steel	995	38.0	25
Hollow-Filled Stainless Steel	1716	41.5	41
Glasforms	2064	38.9	53
Creative Pultrusions	2630	38.9	68
RJD	974	40.1	24

The maximum pull-out stresses recorded during pre-freeze thaw testing of the oiled dowels typically occurred during the initial 0.002 inches of dowel displacement, likely indicating the force necessary to release the bond between the dowel end and concrete. After peak readings, the pull-out stresses typically reduced to a significantly lower residual level. After freeze-thaw conditioning, the peak pull-out stresses again typically occurred during the initial 0.002 inches of displacement. In some cases this post-freeze thaw maximum pull-out stress was approximately equal to the pre-freeze thaw residual pull-out stress. This may be expected due to the breaking of the bond between the dowel end and the PCC during pre-freeze thaw testing. However, in other cases the post-freeze thaw maximum pull-out stress was greater than the pre-freeze thaw maximum value, which cannot be explained by the dowel end release during pre-freeze thaw testing.

A notable exception to this trend was the epoxy coated dowel (Figure 2.4.3). During pre-freeze thaw testing, the peak pull-out load occurred at approximately 0.05 inches of displacement and only reduced slightly to a residual load that remained essentially constant to a displacement of approximately 0.35 inches. The pull-out load then began to increase with increasing displacements for the remaining 0.15 inches of displacement. After freeze-thaw conditioning, pull-out loads again continually increased with increasing displacement, with the most significant increase occurring during the initial 0.05 inches of displacement.

Pull-out stresses recorded for the composite dowels also revealed some inconsistencies in behavior. As shown in Figures 2.4.6 and 2.4.7 for the RJD and Glasforms dowels, the stress paths during relaxation include noticeable oscillations, resulting in short-term stress “bumps” up to approximately 5 psi. In Figure 2.4.8, which illustrates the stress paths for the Creative Pultrusions dowel, the post-freeze thaw stress gain after initial relaxation is accompanied by significant “stepping” approaching 20 psi.

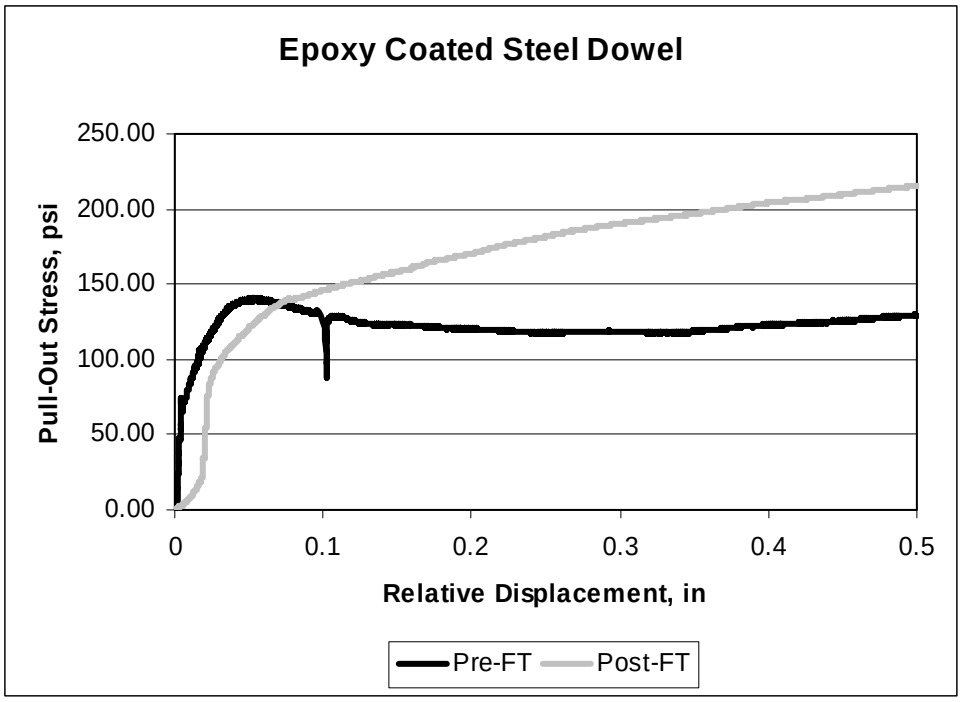


Figure 2.4.3: Pull-Out Stress Trends for the Epoxy Coated Steel Dowel

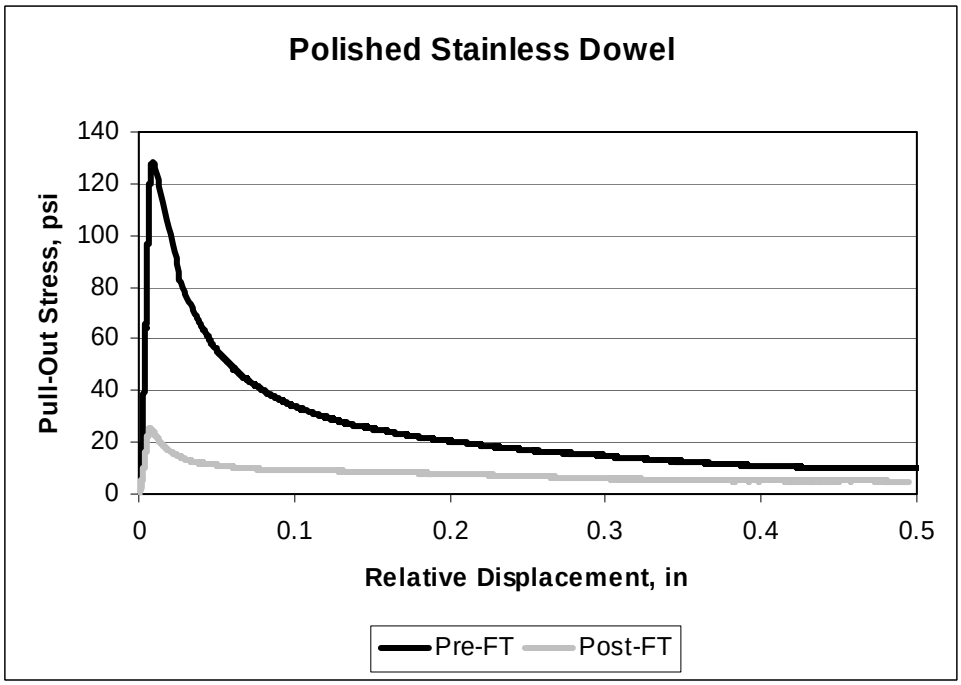


Figure 2.4.4: Pull-Out Stress Trends for the Solid Stainless Steel Dowel

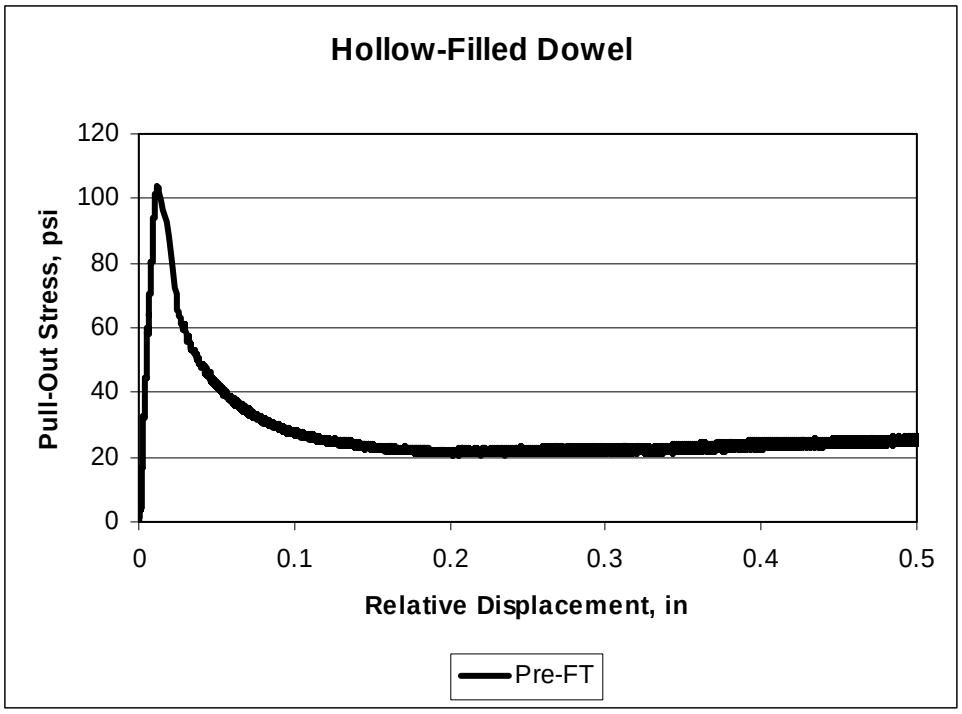


Figure 2.4.5: Pull-Out Stress Trends for the Hollow-Filled Dowel

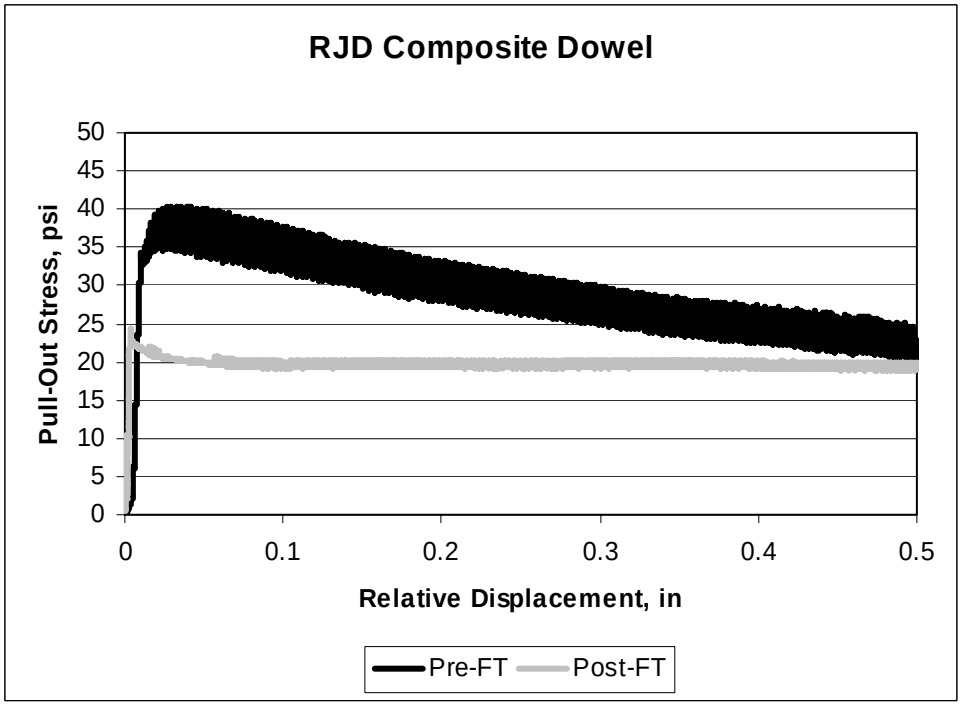


Figure 2.4.6: Pull-Out Stress Trends for the RJD Composite Dowel

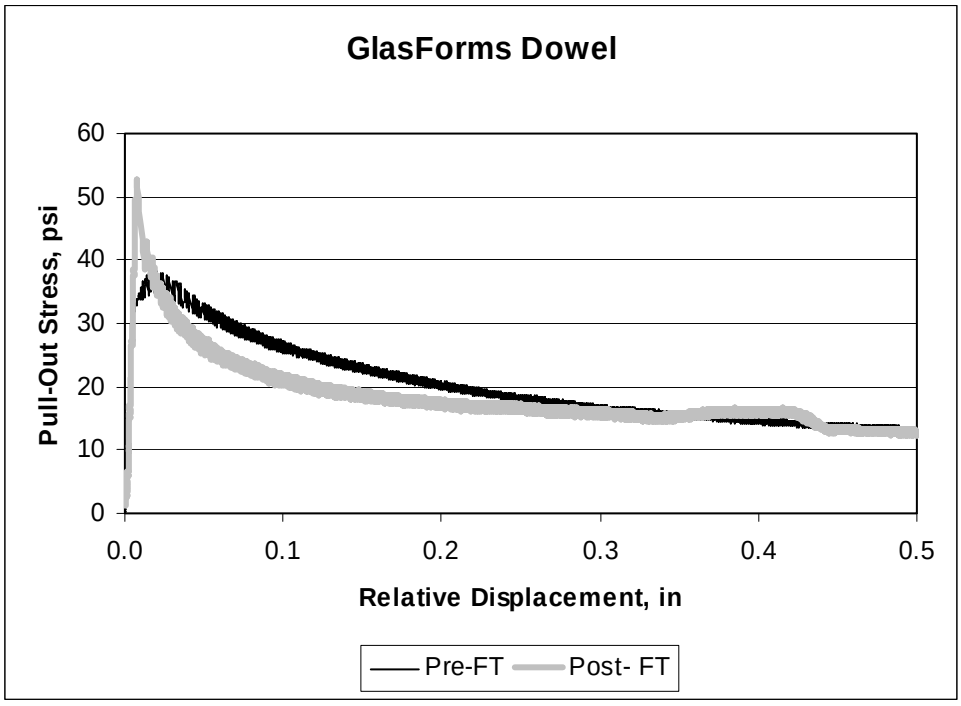


Figure 2.4.7: Pull-Out Stress Trends for the Glasforms Composite Dowel

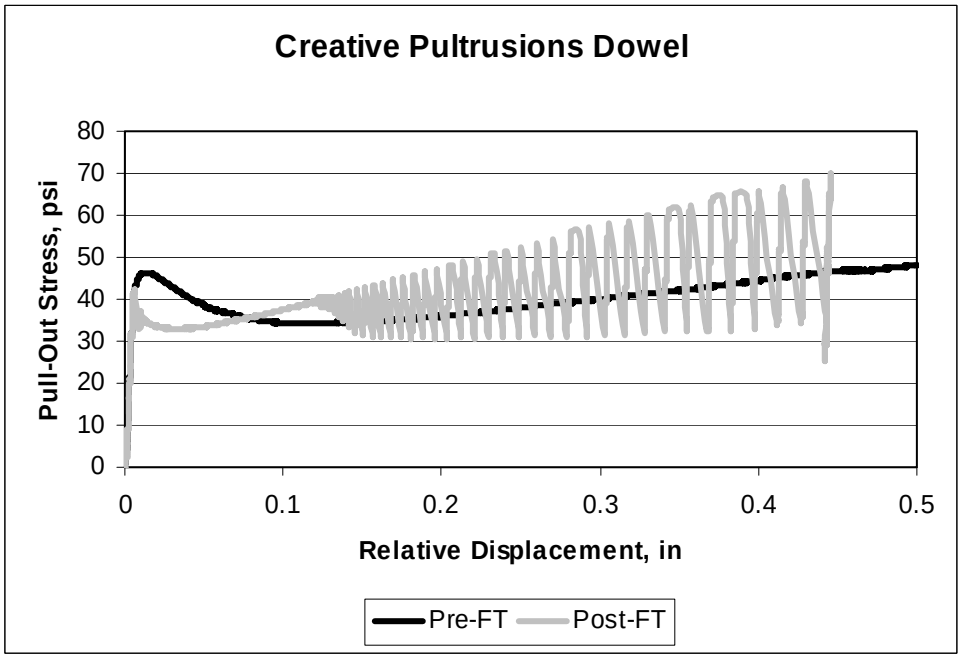


Figure 2.4.8: Pull-Out Stress Trends for the Creative Pultrusions Composite Dowel

After completion of the pull-out tests the concrete blocks were split to reveal the surface of the embedded dowels. No signs of corrosion were observed. Striations were noted on the surfaces of all dowels and the exposed surfaces of the polished stainless steel dowels resembled the brushed stainless steel surfaces of the dowels used during the initial, non-oiled tests.

CHAPTER 3
TEST SECTION CONSTRUCTION

3.1 Introduction

This chapter provides details relating to the construction of each test section. Information on each section was obtained from project plans and from observations of the on-site research staff during construction operations.

3.2 WIS 29 Abbotsford

Paving of the eastbound lanes on WIS 29 Abbotsford incorporating all test sections was completed by Streu Construction Company during the period of September 3 - 18, 1997 using a Gomaco paver equipped with an automatic dowel bar inserter. The limits of paving were included as part of two separate paving projects. The western portion of paving was included under State project number 1052-08-79 which was designed as a metric project. The eastern portion of paving was included under State project number 1052-08-77. Both projects were part of planned WIS 29 improvements and represented a reconstruction of the pre-existing 2-lane WIS 29 jointed plain concrete pavement (JPCP). Planned improvements completed during the previous year added two westbound lanes to WIS 29 in this project location. These lanes were used for bi-directional traffic during construction of the WIS 29 Abbotsford test sections.

The standard pavement section includes a 26-ft wide, 11-inch thick doweled JPCP with hot mix asphalt shoulders. The JPCP slab was placed over the existing 6-inch crushed aggregate base and 9-inch granular subbase. Crushed aggregate materials from the existing shoulders were used in combination with new crushed aggregates to provide a re-shaped base layer of variable thickness above the existing crushed aggregate base layer. The dowel bars were 1.5 inches in diameter and were placed at 12-inch c-c spacings across the transverse joints (26 per joint). The eastern end of the project (1052-08-79) was designed for a 20-year ESAL value of 11,366,100 based on WisDOT design procedures using a 1993 construction year ADT of 7,925, a 2013 design year ADT of 10,300 and 18% heavy truck traffic. The western end of the project (1052-08-77) was designed for a 20-year ESAL value of 9,380,500 based on WisDOT design procedures using a 1993

construction year ADT of 6,450, a 2013 design year ADT of 8,600 and 27% heavy truck traffic.

All paving within the limits of test section construction was completed using a single paver configuration, which provided for a 25.6-ft paved width with repetitive random joint spacings of 17-20-18-19 ft. The dowel bar inserter utilized fixed dowel spacings of 12 inches throughout the central portions of the slabs. The spacing between the outer dowel and the next dowel inwards was reduced to approximately 9 inches on both slab edges to account for the reduced paving width (25.6 ft versus the 26-ft standard). Each outer dowel was positioned at 6 inches from the slab edge.

Paving progressed from west to east with minimal disruptions due to weather and/or alternate dowel materials and placement configurations. On four of the twelve days of paving, the dowel bar inserter was modified during paving to adjust for changes in dowel bar placement alternates. These modifications required approximately five minutes and resulted in minimal paving delays. A slight reduction in the travel speed of the dowel bar carriage was required during placement of the composite dowels due to their light weight which caused excessive rebound at normal carriage speeds.

Table 3.2.1 provides a daily summary of the paving operations and related test section construction. Placement markers denoting the limits of test section paving were fabricated and placed by WisDOT staff near the right-of-way limits on the south edge of the highway. After construction, representative sections of approximately 528 ft were selected from within each test section for long-term monitoring. Each monitoring section included 29 transverse joints with the exception of the hollow-filled stainless steel dowels where only 20 joints were constructed. Table 3.2.2 provides the station limits for each selected monitoring section, which represent the center of each slab directly outside the first and last joints included within the monitoring sections. Blue markers denoting the limits of each monitoring section were placed by WisDOT staff along the south edge of the highway near the ROW limits.

Table 3.2.1 Paving Summary - WIS 29 Abbotsford

Date	Start Station	End Station	Comments [1]
09-03-97	80+730	79+760	Paving with standard dowel placement using epoxy coated dowels.
09-04-97	79+760	78+777	Paving with standard dowel placement using epoxy coated dowels.
09-05-97	78+777	78+484	Paving with Alternate 1 using epoxy coated dowels. Paving suspended at 9:15 AM due to heavy rain.
09-08-97	78+484	77+352	Paving Alternate 1 using epoxy coated dowels.
09-09-97	77+352 77+171	77+171 76+250	Paving with Alternate 1 using epoxy coated dowels. Paving with Alternate 2 using epoxy coated dowels.
09-10-97	76+250 75+885	75+885 74+997	Paving with Alternate 2 using epoxy coated dowels. Paving with Alternate 3 using epoxy coated dowels.
09-11-97	74+997 74+257	74+257 73+546	Paving with Alternate 3 using epoxy coated dowels. Paving with Alternate 4 using epoxy coated dowels.
09-12-97	73+546	72+388	Paving with Alternate 4 using epoxy coated dowels.
09-15-97	72+388 72+354 71+878	72+354 71+878 71+688	Paving with Alternate 4 using epoxy coated dowels. Paving with Alternate 4 using Avesta Sheffield solid stainless steel dowels. Paving with Alternate 3 using Avesta Sheffield solid stainless steel dowels.
09-16-97	71+688 71+384	71+384 70+997	Paving with Alternate 3 using Avesta Sheffield solid stainless steel dowels. Paving with Alternate 3 using epoxy coated steel dowels. Paving suspended at 1:20 PM due to rain.
09-17-97	70+997 70+979 70+867 2308+52 2292+97	70+979 70+867 2308+52[2] 2292+97 2276+85	Paving with standard placement using epoxy coated dowels. Paving with standard placement using Damascus-Bishop hollow-filled stainless steel dowels. Paving with standard placement using RJD composite dowels. Paving with standard placement using Glasforms composite dowels. Paving with standard placement using Creative Pultrusions composite dowels.
09-18-97	2276+85	2264+29	Paving with standard placement using epoxy coated dowels.

[1] Placement alternates illustrated in Figure 1.1.1

[2] Station change from metric to English, Sta 70+680 (M) = Sta 2318+89.76 (E)

Table 3.2.2 - Monitoring Section Locations - WIS 29 Abbotsford

Section Code	Start Station	End Station	Comments
C1	2270+00	2275+37	Control 1 - Standard Placement with Epoxy Coated Dowels
CP	2280+00	2285+36	Standard Placement with Creative Pultrusions Composite Dowels
GF	2300+00	2305+32	Standard Placement with Glasforms Composite Dowels
RJD	2310+10	2315+43*	Standard Placement with RJD Composite Dowels
HF	70+867*	70+979	Standard Placement with Damascus-Bishop Hollow-Filled Stainless Steel Dowels
3Ea	71+047	71+210	Alternate 3 with Epoxy Coated Dowels
3S	71+523	71+681	Alternate 3 with Avesta Sheffield Solid Stainless Steel Dowels
4S	71+898	72+060	Alternate 4 with Avesta Sheffield Solid Stainless Steel Dowels
4E	72+800	72+961	Alternate 4 with Epoxy Coated Dowels
3Eb	75+680	75+841	Alternate 3 with Epoxy Coated Dowels
2E	76+600	756+761	Alternate 2 with Epoxy Coated Dowels
1E	77+560	77+721	Alternate 1 with Epoxy Coated Dowels
C2	78+900	79+061	Control 2 - Standard Placement with Epoxy Coated Dowels

* Station change from metric to English, Sta 70+680 (M) = Sta 2318+89.76 (E)

3.3 WIS 29 Wittenberg

Paving of the eastbound lanes on WIS 29 Wittenberg incorporating all eastbound test sections was completed by James Cape & Sons Co. during the period of October 16-17, 1997 under State project 1059-16-74. Paving was completed with a Rex paver and progressed from west to east with no disruptions due to weather and minimal disruptions due to dowel material supply problems. The standard pavement section includes a 26-ft wide, 11-inch doweled JPCP with hot mix asphalt shoulders. The JPCP slab was placed over a 4-inch open graded base course over a 6-inch dense graded crushed aggregate

base. The dowel bars are 1.5 inches in diameter and are placed at 12-inch c-c spacings across the transverse joints. Dowels were placed using traditional dowel baskets which were hand placed immediately in advance of paving operations. The project was designed for a 20-year ESAL value of 10,658,000 based on WisDOT design procedures using a 1995 construction year ADT of 6,650, a 2015 design year ADT of 8,700 and 29.5% heavy truck traffic.

Table 3.3.1 provides a daily summary of the paving operations related to eastbound test section construction observed by Marquette University staff. Construction of the westbound test sections was completed earlier in the paving season and was not observed by Marquette staff.

The shipment of composite dowels produced by RJD was delayed which caused this test section to be placed approximately one mile west of the remaining alternate dowel material test sections in a pre-existing paving gap. Furthermore, the remaining composite dowels were improperly distributed between the 12-ft and 14-ft basket lengths, resulting in all of the Glasforms composite bars being placed in 12-ft baskets and most of the MMFG composite bars being placed in the 14-ft baskets. As a result, of the 36 joints located within the composite section, 27 contained mismatches of manufacturers between the passing and driving lanes. Table 3.3.2 provides a listing of the composite dowel placement details.

After construction, representative monitoring sections of approximately 528 ft were selected from within each eastbound and westbound test section for long-term monitoring. All monitoring sections include 29 transverse joints with the exception of the RJD composite dowel section where only 9 joints were constructed. Table 3.3.3 provides the station limits for each selected section, which represent the center of each slab directly outside the first and last joints included within the monitoring sections.

Table 3.3.1: Paving Summary - WIS 29 Wittenberg

Day	Start Station	End Station	Comments
10-16-97	1194+30	1200+60	Paving with standard dowel placement using composite (MMFG, Glasforms, Creative Pultrusions) dowels
10-16-97	1200+76	1201+68	Paving with standard dowel placement using epoxy coated dowels
10-16-97	1201+86	1207+80	Paving with standard dowel placement using Slater Steels solid stainless steel dowels.
10-16-97	1207+98	1223+50	Paving with standard dowel placement using epoxy coated dowels
10-17-98	1144+68	1146+12	Paving with standard dowel placement using RJD composite dowels

Table 3.3.2: Composite Dowel Placement Details - WIS 29 Wittenberg

Joint Station	Driving Lane	Passing Lane
1144+68 - 1146+12	RJD	RJD
1194+30	MMFG	MMFG & Glasforms
1194+48 - 1194+66	MMFG	MMFG
1194+84 - 1197+36	MMFG	Glasforms
1197+54 - 1199+34	Creative Pultrusions	Glasforms
1199+52 - 1200+60	Creative Pultrusions	Creative Pultrusions

Table 3.3.3: Monitoring Section Locations - WIS 29 Wittenberg

Eastbound Lanes			
Section Code	Start Station	End Station	Comments
C1	1133+30	1138+55	Control 1 - Standard Placement with Epoxy Coated Dowels
RJD	1144+59	1146+21	Standard Placement with Composite Dowels (RJD)
FR	1194+22	1199+76	Standard Placement with Composite Dowels (Glasforms, Creative Pultrusions, MMFG)
SS	1202+14	1207+35	Standard Placement with Slater Steels Solid Stainless Steel Dowels
C2	1208+06	1213+31	Control 2 - Standard Placement with Epoxy Coated Dowels
Westbound Lanes			
Section Code	Start Station	End Station	Comments
1E	1207+44	1202+20	Alternate 1 with Epoxy Coated Dowels
C3	1200+23	1195+00	Control 3 - Standard Placement with Epoxy Coated Dowels
TR	1193+55	1188+28	Standard Placement with Epoxy Coated Dowels and Trapezoidal Slab Design

3.4 WIS 29 Tilleda

Paving of the westbound lanes on WIS 29 Tilleda, incorporating all test sections, was completed by James Cape & Sons Co. during the period of September 7-8,1999 under state metric project number 1059-16-80. Paving was completed with a Town & Country paver and progressed from east to west with no disruptions due to weather.

The standard pavement section includes a 26 ft wide, 10-inch doweled JPCP slab with hot mix asphalt shoulders. The JPCP slab was placed over a 4-inch open graded base course over a 6-inch dense graded crushed aggregate base. The dowel bars are 1.5 inches in diameter and are placed at 12-inch c-c spacings across the transverse joints (26 per joint). The pavement was designed for a 20-year ESAL value of 8,847,600 based on

WisDOT design procedures using a 2000 construction year ADT of 5,675, a 2020 design year ADT of 7,088 and 19.8% heavy truck traffic.

Dowels were placed using traditional dowel baskets which were hand placed well in advance of paving operations. A material transfer belt was used to move concrete materials from supply trucks positioned along the outer shoulder to the paver. Table 3.4.1 provides a daily summary of the paving operations related to westbound test section construction observed by Marquette University staff.

All dowel baskets were designed for a uniform depth, 10-inch (250 mm) PCC slab, which required adjustments to avoid improper placement depths for the variable slab thicknesses used within some of the WIS 29 Tilleda test sections. Placement adjustments were made using a vibrating plate compactor running along the top rails of the basket and sinking the baskets into the open graded permeable base layer to the desired depth. Hand measurements made by Marquette staff indicated this method was generally effective in positioning the dowels within 0.5 inches of the mid-depth of the PCC slab.

After construction, representative monitoring sections of approximately 500 ft were selected from within each 1,000 ft test section for long-term monitoring. All monitoring sections constructed with 15 ft joint spacings include 33 transverse joints. Test Section 1, which was constructed with 18 ft joint spacings, includes 28 joints. Table 3.4.2 provides the station limits for each selected section, which represent the center of each slab directly outside the first and last joints included within the monitoring sections.

Table 3.4.1: Paving Summary - WIS 29 Tilleda

Date	Start Station	End Station	Comments
9-7-99	64+270	63+955	Variable thickness passing lane (8 – 10 inches), widened passing lane (15 ft), 15-ft transverse joint spacing, two-way surface and base drainage
	63+955	63+937	Transition section
	63+937	63+622	Variable thickness passing lane (8 – 10 inches), widened passing lane (15 ft), 15-ft transverse joint spacing, two-way surface and one-way base drainage
	63+622	63+604	Transition section
	63+604	63+334	Uniform slab thickness (10-inch), widened passing lane (15 ft), 15-ft joint spacing, two-way surface and base drainage
	63+334	63+316	Transition section
9-8-99	63+316	63+001	Variable thickness across both lanes (8–10 inches), widened passing lane (15 ft), 15-ft transverse joint spacing, one-way surface and base drainage.
	63+001	62+983	Transition section
	62+983	62+664	Uniform slab thickness (10-inch), 18-ft transverse joint spacing, one-way surface and base drainage.

Table 3.4.2 - Monitoring Section Locations - WIS 29 Tilleda

Section Code	Start Station	End Station	Comments
TS4	64+189	64+036	Variable thickness and widened passing lane, two-way surface and base drainage
TS3	63+856	63+703	Variable thickness and widened passing lane, two-way surface and one-way base drainage
STD	63+545	63+392	Uniform thickness, widened passing lane, two-way surface and base drainage
TS2	63+235	63+082	Variable thickness across both lanes, widened passing lane one-way surface and base drainage
TS1	62+900	62+747	Uniform thickness, one-way surface and base drainage

CHAPTER 4
PERFORMANCE MONITORING

4.1 Introduction

Performance monitoring, including falling weight deflectometer (FWD) testing, distress measurements, and ride quality measurements, was initiated soon after construction and completed in subsequent years. FWD measurements were conducted by Marquette University and contract staff. Joint and slab distress measurements were recorded by Marquette University staff during visual surveys. Distress surveys were also completed by WisDOT staff following the Pavement Distress Index (PDI) procedures. Ride quality measurements were completed by WisDOT staff using automated survey equipment. The following sections provide details of the survey results.

4.2 Falling Weight Deflectometer (FWD) Analysis

Nondestructive deflection testing (NDT) using an 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. FWD testing was conducted using the Marquette University KUAB Model 50 2m-FWD and the Engineering and Research International (ERI) KUAB Model 150 2m-FWD. Both 2m-FWD models utilize a two-mass falling weight package which produces a smooth, haversine load pulse to the pavement surface over a 12-inch segmented load plate. The magnitude of the dynamic load is varied by adjusting the height of fall of the primary weight package. Deflection testing was conducted prior to paving operations, after paving and immediately prior to opening to public traffic, and at subsequent intervals after trafficking.

4.2.1 Pre-Paving Deflection Testing

Deflection tests conducted immediately prior to the paving operations provide a measure of the strength and uniformity of the foundation materials. The maximum deflection under loading, normalized to a reference load level, provides a general indication of the overall uniformity of support provided by the foundation materials, which include the natural subgrade and existing/constructed aggregate subbase and base layers. Deflections

measured at distances away from the center of loading may be used to estimate the elastic moduli of foundation materials. A small load level and/or a larger load plate is suggested to provide pre-paving top-of-base stress levels which are as close as possible to those which would be induced during post-paving FWD testing on the top of constructed JPCP slabs. It should be noted, however, that applied top-of-base stress levels during pre-paving testing are generally much greater than the stress levels which would be anticipated under a 9,000 lb load after a 10 to 11-inch concrete slab is in place. Therefore, foundation material properties which are derived from pre-paving surface deflections may be significantly lower than those computed from post-paving deflections due to the stress-dependent behavior of the foundation materials. However, a general comparison of foundation material properties between constructed test sections can serve to identify variances that may contribute to pavement performance variations.

Using single-layer elastic layer theory (Boussinesq 1885, Ahlvin and Ulery 1962), an approximation of the equivalent modulus of the combined base-subgrade may be obtained from the maximum deflection under loading using the equation:

$$E_{eq} = 1500\ P / (\pi\ a\ \delta_o) \qquad \text{Eqn 4.1}$$

where: E_{eq} = equivalent elastic modulus of foundation, psi
P = applied load, lb
a = load radius, in
δ_0 = maximum deflection, mils

The subgrade elastic moduli may be approximated using deflections away from the center of loading by the equation (AASHTO 1993):

$$E_{sg} = 0.24\ P / (\delta_r\ r) \qquad \text{Eqn 4.2}$$

where: E_{sg} = subgrade elastic modulus, psi
P = applied load, kips
δ_r = surface deflection at r inches from the center of loading, mils
r = distance from center of loading where deflection is measured, in

Based on previous research conducted by the author of this report, a reasonable estimate of E_{sg} may be obtained by first computing multiple values of E_{sg} from Eqn 4.2 using all deflections measured at locations of $r > 0$ and then selecting the minimum

computed E_{sg} as the estimate of the subgrade elastic modulus.

4.2.2 Post-Paving Backcalculation of Pavement Parameters

The foundation k-value and slab properties may be backcalculated from center slab and joint deflections using the following 7-step process which is applicable to highway pavements (Crovetti 1994):

Step 1: The deflection basin AREA (Hoffman, 1981) is computed from center slab deflections using the equation:

$$AREA = (6 / \delta_0) (\delta_0 + 2\delta_{12} + 2\delta_{24} + \delta_{36}) \quad \text{Eqn 4.3}$$

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, $\ell_{k\text{-}est}$ is backcalculated using the equation:

$$\ell_{k\text{-}est} = \{\ln[(36\text{-}AREA) / 1812.279133] / -2.55934\}^{4.387009} \quad \text{Eqn 4.4}$$

The dense-liquid radius of relative stiffness (Westergaard, 1926) is a combined term which incorporates slab and subgrade properties and is defined as:

$$\ell_k = [(E_c H_c^3) / (12 (1-\mu_c^2) k)]^{0.25} \quad \text{Eqn 4.5}$$

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 (Crovetti, 1994):

$$L_{eff} = L_{act} + \Sigma (L_{adj} * LT_\delta^{\,2}) \quad \text{Eqn 4.6}$$

$$W_{eff} = W_{act} + \Sigma (W_{adj} * LT_\delta^{\,2}) \quad \text{Eqn 4.7}$$

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
$LT_\delta = d_u / d_l$

d_u = deflection of unloaded slab at 12 inches from the load plate, mils
d_l = deflection of the loaded slab at the center of loading, mils

Step 4: Slab size correction factors are computed as (Crovetti, 1994):

$$CF_{\ell k\text{-}est} = 1 - 0.89434 \exp [-0.61662 (L_{eff} / \ell_{k\text{-}est})^{1.04831}] \quad \text{Eqn 4.8}$$

$$CF_{\delta i} = 1 - 1.15085 \exp [-0.71878 (W_{eff} / \ell_{k\text{-}est})^{0.80151}] \quad \text{Eqn 4.9}$$

where: $CF_{\ell k\text{-}est}$ = 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:

$$\ell_{k\text{-}adj} = \ell_{k\text{-}est} * CF_{\ell k\text{-}est} \quad \text{Eqn 4.10}$$

$$\delta_{i\text{-}adj} = \delta_i * CF_{di} \quad \text{Eqn 4.11}$$

Step 6: The subgrade dynamic k-value is backcalculated using the equation (Crovetti, 1994):

$$k_i = [1000 P / (\delta_{i\text{-}adj} \ell_{k\text{-}adj}^2)] [0.1253 - 0.008 a / \ell_{k\text{-}adj} - 0.028 (a/\ell_{k\text{-}adj})^2] \quad \text{Eqn 4.12}$$

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

Step 7: The elastic modulus or effective thickness of the concrete slab is estimated from previously backcalculated ℓ_k and k values by a rearrangement of Eqn 4.5 as follows:

$$E_c = 11.73 \ell_{k\text{-}adj}^4 k_i / H_c^3 \quad \text{Eqn 4.13}$$

$$H_c = [11.73 \ell_{k\text{-}adj}^4 k_i / E_c]^{1/3} \quad \text{Eqn 4.14}$$

where: H_c in Eqn 4.13 = known or assumed slab thickness, in
E_c in Eqn 4.14 = known or assumed PCC modulus, psi

The process described in analysis steps 1 - 7 generally provides reasonable estimates for slab and foundation properties when the slab is relatively flat (i.e., no temperature curling or moisture warping) and minimum effective slab dimension exceeds 3

times the radius of relative stiffness, ℓ_k. For typical highway applications, ℓ_k values of 36 +/- 12 inches are common, indicating effective slab dimensions of 9 +/- 3 feet are required. For 12-14 ft wide slabs with transverse joint spacings of 15-20 ft, this requirement is easily met. However, through-slab temperature gradients may produce sufficient downward temperature curling when the top portions of the slab are significantly warmer than the bottom portions and zones of non-contact near the slab center may be present. In these cases, incremental analysis using at least two test load levels must be used to provide reasonable estimates of slab and subgrade properties.

It may also be of interest to determine the elastic modulus of the subgrade instead of the subgrade k-value. This property may be determined following a process similar to that presented for the subgrade k-value with coefficients and exponents modified for elastic solid response. Based on research conducted by the author, a reasonable estimate of the subgrade elastic modulus may be computed directly from backcalculated k_i and $\ell_{k\text{-}adj}$ values using the equation (Crovetti 1994):

$$E_{sg} = 3.39\ k_i\ \ell_{k\text{-}adj} \qquad \text{Eqn 4.15}$$

where: E_{sg} = elastic modulus of subgrade, psi

4.2.3 Post-Paving Transverse Joint Analysis

Deflection readings from tests conducted across transverse joints can provide a number of useful parameters for assessing pavement performance. For maximum benefit, deflection testing should be conducted with the load plate positioned tangent to adjacent joints with deflection sensors located on both the loaded and unloaded slabs.

Load transfer measures can provide information on the ability of adjacent slabs to distribute stress and deflection from critical edge and corner loadings which may lead to joint faulting and/or load-induced transverse, longitudinal and corner cracking. In general, deflection load transfer is relatively unaffected by the magnitude of the applied load, provided the slab is uniformly supported. Marked reductions in load transfer at higher load levels may be an indication of poor support under the unloaded slab. Poor support under one slab may also result in significant differences in measured load transfer when the load is positioned on both sides of the joint during testing. For doweled JPCP, properly

performing joints are typically expected to have deflection load transfer efficiencies of approximately 85% or greater.

Maximum and total joint deflection can provide indications of existing or potential future loss of support in the vicinity of slab edges and corners, which can lead to joint faulting, pumping and/or slab cracking. For JPCP, the maximum joint deflection may vary due to seasonal changes in deflection load transfer; however, the total joint deflection should remain relatively constant, assuming there is no loss of support or temperature curling. For comparative purposes, maximum and total joint deflections are commonly normalized to a reference load level (e.g., 9 kips)

The deflection load transfer across joints may be simply calculated using the equation:

$$LT\% = \delta_u / \delta_l \times 100\% \qquad \text{Eqn 4.16}$$

where: LT% = deflection load transfer efficiency, %
δ_u = deflection on unloaded slab at 12 inches from load center, mils
δ_l = deflection on loaded slab at the load center, mils

The normalized total joint deflection may be computed using the equation:

$$\delta_t = 9 (\delta_l + \delta_u) / P \qquad \text{Eqn 4.17}$$

where: δ_t = normalized total joint deflection, mils@9k
δ_l = deflection on loaded slab at the load center, mils
δ_u = deflection on unloaded slab at 12 inches from load center, mils
P = applied load, kip

4.2.4 Pre-Paving Deflection Testing - WIS 29 Abbotsford

Deflection tests were conducted along WIS 29 Abbotsford in advance of paving operations to provide a measure of the strength and uniformity of the foundation materials. Deflection tests were conducted between September 3-14, 1997 with the Marquette University 2m-FWD from stations 70+680 to 79+900 (SPN 1052-08-79) and from 2289+01 to 2318+90 (SPN 1052-08-77, equivalent metric stations 69+769 to 70+680). Tests were conducted at approximately 300-ft intervals along the driving lane within the testing limits. Additional tests were conducted along the passing lane at 300-ft intervals, staggered 150-ft from the driving lane tests, between stations 72+150 and 79+650. The smallest load level of approximately 3,000 lb was used to provide top-of-base stress levels of approximately 27 psi. The maximum deflection under loading, normalized to a common load level, was used to provide a general indication of the overall uniformity of support provided by the foundation materials in the areas of testing, which include the natural subgrade and existing/constructed aggregate subbase and base layers. Table 4.2.1 provides overall summary statistics for the maximum deflections recorded along the passing and driving lanes, normalized to 3,000 lb load, as well as within test section values of average maximum deflection within the driving lane. Figure 4.2.1 provides a profile plot of the maximum deflection values.

Table 4.2.1: Maximum Pre-Paving Deflection Statistics - WIS 29 Abbotsford

Test Statistic	Test Lane	
	Driving	Passing
Overall Mean, mils@3k	21.36	25.52
Standard Deviation, mils@3k	9.88	14.68
Coefficient of Variation, %	46.2	57.5
Test Section	Driving Lane Mean Deflection, mils@3k [(1)]	
CP	24.23	
GF	24.02	
RJD	16.17	
HF	14.98	
3Ea	19.06	
3S	15.73	
4S	27.22	
4E	25.57	
3Eb	14.12	
2E	23.53	
1E	22.18	
C2	19.99	

[(1)] mils at 3,000 lb load level (1 mil = 0.001 inch)

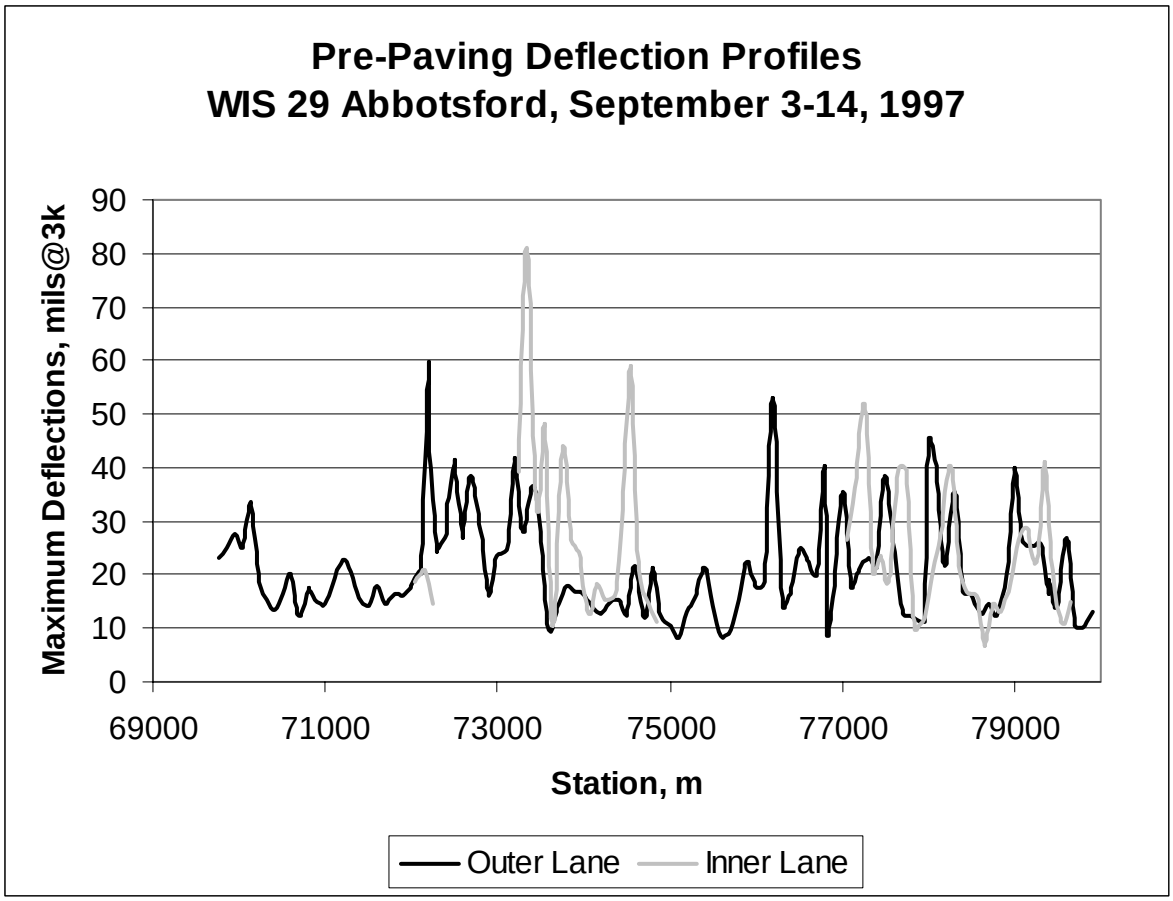


Figure 4.2.1: Pre-Paving Deflection Profiles, WIS 29 Abbotsford

The maximum deflection (r=0) and deflections away from the center of loading (r>0) were used to estimate the elastic moduli of foundation materials. Table 4.2.2 provides overall summary statistics for these estimated moduli values, determined by Eqns 4.1 and 4.2, as well as within section values based on measures within the driving lane. As shown, the mean equivalent modulus of the combined base-subgrade is substantially higher than the mean estimated E_{sg} value, which is expected due to the increased stiffness of the in-place base materials.

Table 4.2.2:Summary Statistics For Estimated Moduli Values - WIS 29 Abbotsford

Test Statistic	Combined Base/Subgrade Elastic Modulus, ksi		AASHTO Subgrade Elastic Modulus, ksi	
	Passing	Driving	Passing	Driving
Mean Value, ksi	12.4	13.5	8.2	8.8
Std. Deviation, ksi	6.4	5.4	4.5	4.1
Coeff. of Variation, %	51.7	39.8	55.1	46.5

Test Section	Mean Combined Base/Subgrade Value, ksi	Mean AASHTO Subgrade Elastic Modulus, ksi
CP	10.0	6.5
GF	10.9	7.0
RJD	15.4	9.8
HF	16.2	6.7
3Ea	13.1	8.1
3S	15.5	9.6
4S	10.6	6.0
4E	11.2	6.5
3Eb	18.7	11.7
2E	12.4	8.2
1E	13.3	10.2
C2	14.2	10.4

4.2.5 Post-Paving Deflection Testing - WIS 29 Abbotsford

Post-paving deflection tests were conducted within the driving lane of established test sections just before opening to traffic and at subsequent times after paving. The initial post-paving tests were conducted on October 29-30, 1997, approximately six weeks after paving, and included center slab and outer wheel path transverse joint tests on five selected slabs in each test section spaced at approximately 100 ft intervals. Additional mid-lane transverse joint tests were conducted within test sections containing alternate dowel placements and in the second control section. The analysis procedures outlined in Section 4.2.2 were used to estimate the subgrade dynamic k-value, the effective slab thickness, and the effective slab modulus for each test section. The effective slab thicknesses were backcalculated using an assumed PCC modulus of 3.8 Mpsi, which is equivalent to a compressive strength of approximately 4,500 psi. The effective slab moduli were backcalculated using an assumed thickness of 11 inches, which is equal to the design slab thickness. Normalized total joint deflections and joint deflection load transfers were also computed. Tables 4.2.3 and 4.2.4 provide summary statistics for these computed values. As shown in Table 4.2.3, mean subgrade dynamic k-values are generally consistent throughout all test sections, with test section values ranging from 294 to 378 psi/in. Within-section variability is relatively low, with coefficients of variation ranging from 7.7 to 20.0%. The mean effective thicknesses are also quite similar between test sections, with backcalculated values ranging from 10.8 to 11.9 inches. Within-section variability is also quite low, with coefficients of variation ranging from 3.3 to 12.9%. The mean effective slab moduli are also quite similar between test sections, with backcalculated values ranging from 3.4 to 4.6 Mpsi. Within-section variability is relatively high, with coefficients of variation ranging from 9.7 to 43.3%.

Table 4.2.3: Post-Paving Test Results - WIS 29 Abbotsford (Oct 1997)

Test Section	Subgrade k-value		Effective Slab Thickness[(1)]		Effective Slab Modulus [(2)]	
	Mean psi/in	COV %	Mean in	COV %	Mean Mpsi	COV %
C1	362	9.3	11.9	4.4	4.6	13.2
CP	367	19.6	11.8	12.9	4.6	43.3
GF	378	15.8	10.9	9.2	3.5	28.6
RJD	360	16.3	11.2	5.6	3.9	17.4
HF	376	16.0	10.8	10.9	3.5	33.9
3Ea	297	13.1	11.6	4.6	4.3	13.8
3S	343	14.9	11.6	6.1	4.3	17.9
4S	311	17.5	11.1	8.9	3.7	26.9
4E	330	10.8	10.8	3.3	3.4	9.7
3Eb	331	20.0	11.6	11.8	4.3	37.4
2E	354	7.7	11.4	7.2	4.0	22.3
1E	329	12.3	11.7	7.8	4.4	22.4
C2	294	11.0	11.4	6.9	4.1	20.3

(1) Backcalculated assuming Ec = 3.8 Mpsi
(2) Backcalculated assuming Hc = 11 in

Table 4.2.4: Post-Paving Test Results - WIS 29 Abbotsford (Oct 1997)

Test Section	Mean k-value psi/in	Mean Deflection Load Transfer, %		Mean Total Joint Deflection mils@9k	
		OWP[1]	Mid-Lane[2]	OWP[1]	Mid-Lane[2]
C1	362	90	n.a.	10.72	n.a.
CP	367	73	n.a.	8.96	n.a.
GF	378	71	n.a.	9.30	n.a.
RJD	360	62	n.a.	8.59	n.a.
HF	376	78	n.a.	8.24	n.a.
3Ea	297	78	59	8.43	7.75
3S	343	77	68	7.27	6.87
4S	311	73	62	8.53	7.75
4E	330	76	60	9.91	8.92
3Eb	331	79	63	9.34	7.98
2E	354	80	59	9.64	8.16
1E	329	68	57	8.88	8.36
C2	294	86	83	8.69	8.39
Overall Average		76.2	63.9	8.962	8.023

[1] OWP = outer wheel path of driving lane
[2] Mid-lane = center of driving lane

The average transverse joint load transfer(s) provided in Table 4.2.4 indicate a number of interesting trends which can be summarized as follows:

Dowel Bar Materials

- Average outer wheel path transverse joint load transfer provided by standard placements with FRP composite (CP, GF, RJD) and hollow-filled stainless steel (HF) dowels is markedly reduced as compared to conventional epoxy coated steel dowels (C1, C2). The overall average transverse joint load transfer for the FRP, HF and epoxy coated steel dowels was 69%, 78% and 88%, respectively.

- Average wheel path transverse joint load transfer provided by alternate placements with stainless steel (3S, 4S) is slightly lower than comparable placements with conventional epoxy coated steel dowels (3Ea, 3Eb, 4E). Mean test section values for the stainless steel and conventional epoxy coated steel dowels ranged from 73% to 77% and from 76% to 79%, respectively.

Alternate Placements

- Section average wheel path transverse joint deflection load transfer generally decreases with decreasing wheel path dowels. For those sections with 4 dowels in the outer wheel path (3Ea, 3Eb, 3S, 2E), section average deflection load transfers ranged from 77% to 80%. For those sections with 3 dowels in the outer wheel path (4E, 4S, 1E), section average deflection load transfers ranged from 68% (1E) to 73% (4S) to 76% (4E). The addition of a dowel near the slab edge increased available deflection load transfer (4E compared to 1E).

- Eliminating dowels from mid-lane placements (4E, 4S, 3Ea, 3Eb, 3S, 2E, 1E) resulted in a substantial reduction in mid-lane deflection load transfer. Within section comparisons of wheel path to mid-lane load transfer values indicate reductions ranging from 11% to 27%, with an overall average of 19%. In contrast, the mean mid-lane load transfer within the standard placement epoxy coated steel dowels (C2) was only 3% lower than the mean outer wheel path value (83% vs 86%).

The total joint deflections presented in Table 4.2.4 indicate general uniformity between test sections. In all cases with comparative wheel path and mid-lane data, mean outer wheel path total joint deflections are higher than the mid-lane values. This is expected as deflections will continually increase as the test location transitions from the mid-lane location to the pavement edge. For this test data, the overall average outer wheel path total joint deflections are 10% greater than mid-lane values.

Subsequent deflection testing surveys were conducted in June 1998, November 1998, June 1999, November 2002 and October 2004 to examine the impacts of seasonal variations and accumulated traffic loadings on deflection response parameters. Figure 4.2.2 summarizes mean outer wheel path deflection load transfer results for each test section during the first year of service and illustrates the impacts of pavement temperature on load transfer values. As shown, all sections exhibited high load transfers in June, 1998 when pavement temperatures ranged from 75 -100 ^{o}F, with computed load transfer values ranging from 86% to 95%. In contrast, load transfer results from the November, 1998 testing, when pavement temperatures were in the range of 40 - 50 ^{o}F are markedly lower than the June test results. Furthermore, the between-section comparisons of load transfer from the November 1998 testing results are similar to those observed from the October 1997 testing. The standard placements of alternate materials (CP, GF, RJD, HF) again show markedly lower deflection load transfer as compared to standard placements of epoxy coated steel dowels. Alternate placements of stainless steel bars (3S, 4S) again show slightly reduced deflection load transfer as compared to similar placements of epoxy coated steel bars (3Ea, 3Eb, 4E). Also, reducing dowel bars in the outer wheel path also tends to decrease load transfer, with section 1E (3 dowels in the outer wheel path) providing the lowest deflection load transfer.

Figure 4.2.3 summarizes mean outer wheel path deflection load transfer values from tests conducted out to 7 years of service. Included is one spring cycle and two fall cycles of testing. During June 1999 testing when pavement temperatures ranged from 60-90 ^{o}F, outer wheel path deflection load transfer is again high for all sections, ranging from 82% - 92%. During November 2002 testing, when pavement temperatures were in the range of 42 - 55 ^{o}F, markedly reduced deflection load transfer is noted for sections with conventional

placements of FRP dowels (GF, CP, RJD) and alternate placements of stainless steel dowels (3S, 4S). The remaining sections all exhibit similar load transfer values, which is in contrast to previous fall cycle test results for the alternate placements of epoxy coated steel dowels (1E, 2E, 3Ea, 3Eb, 4E) and standard placements of the hollow-filled stainless steel dowels (HF). Test results from the October 2004 testing, when pavement temperatures were in the range of 42 - 68 ^{o}F, again indicate reduced load transfer for the conventional placements of the FRP dowels (CP, GF, RJD) as compared to the epoxy coated steel dowels (C1, C2) and slightly reduced load transfer for the alternate placements of stainless steel dowels (3S, 4S) as compared to similar placements of epoxy coated steel dowels (3Ea, 3Eb, 4E).

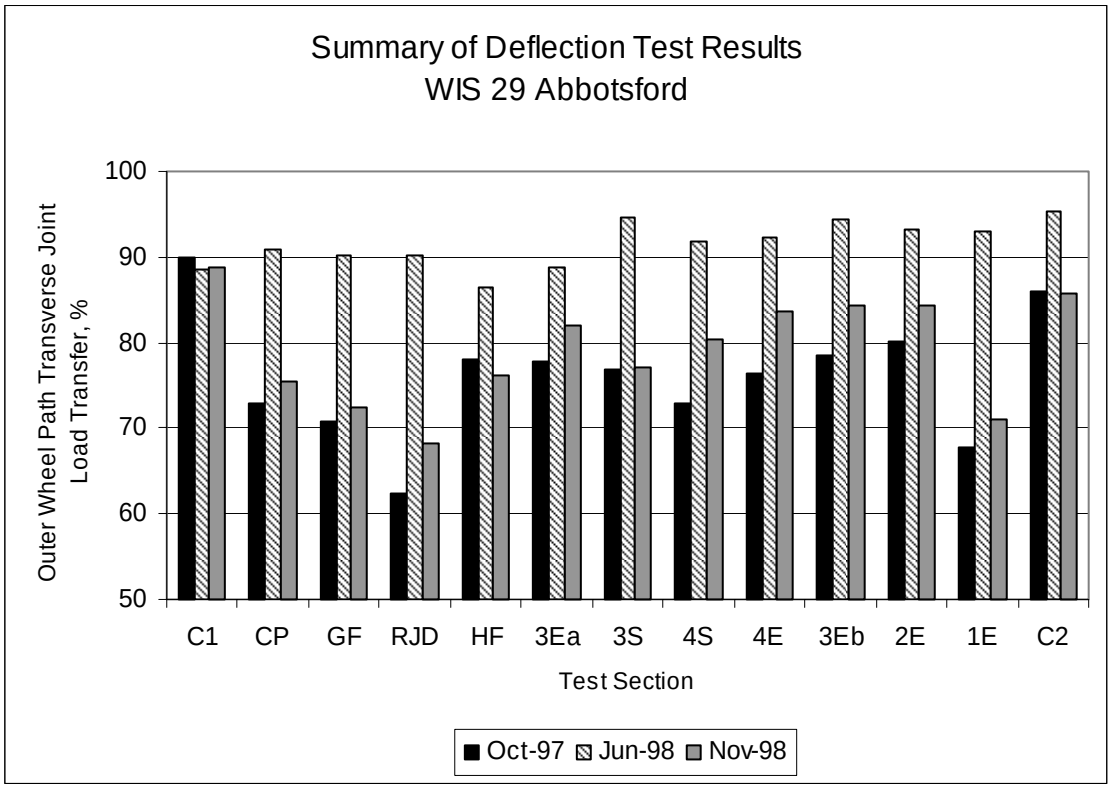


Figure 4.2.2: Transverse Joint Deflection Results - WIS 29 Abbotsford

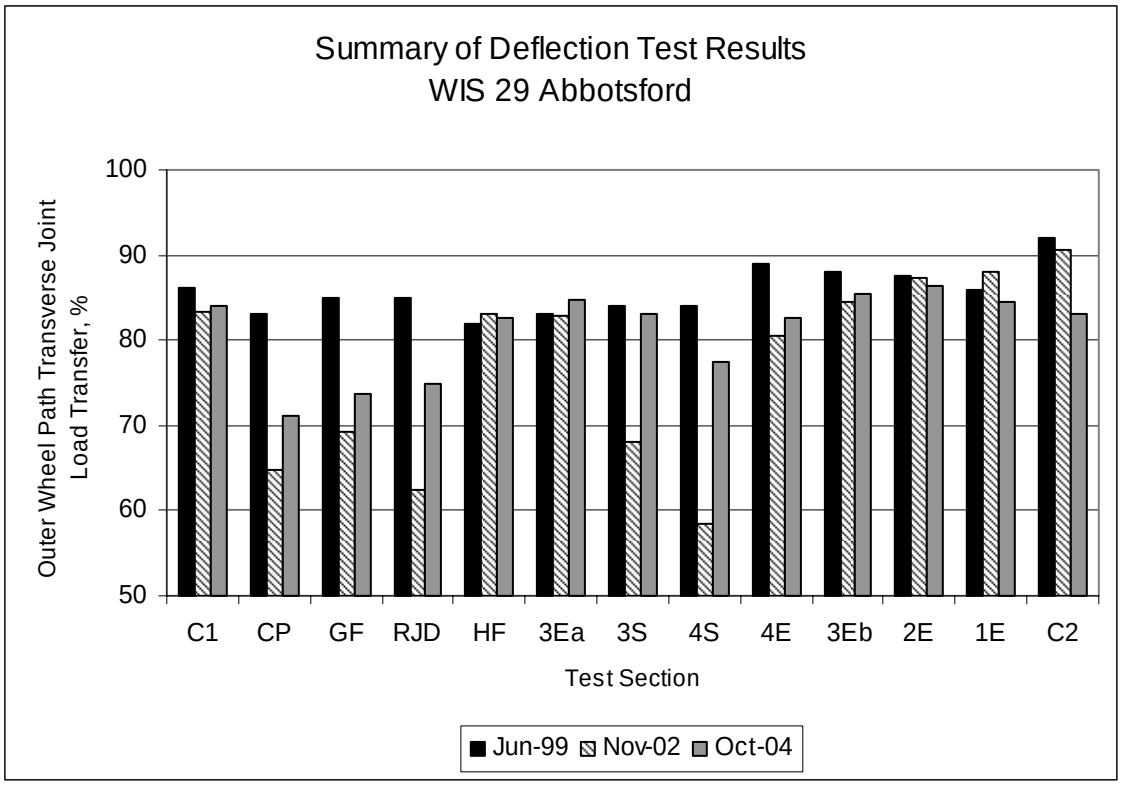


Figure 4.2.3: Transverse Joint Deflection Results - WIS 29 Abbotsford

Figures 4.2.4 and 4.2.5 illustrate mean backcalculated values of subgrade k-value and effective slab modulus, respectively, from the various test series which included center slab deflection measurements. The variability noted in both figures for test results subsequent to the initial October 1997 testing is most likely the result of temperature gradients which induced downward curling during testing of some sections. Downward curling may result in reduced support below the central portions of the slab, increased center slab deflections and reduced curvature of the deflection basin. Downward curling also tends to increase the deflection basin AREA and estimated dense liquid radius of relative stiffness, reduce backcalculated interior k-values, and increase backcalculated effective slab moduli. These negative effects are more pronounced as the stiffness of the subgrade layer increases.

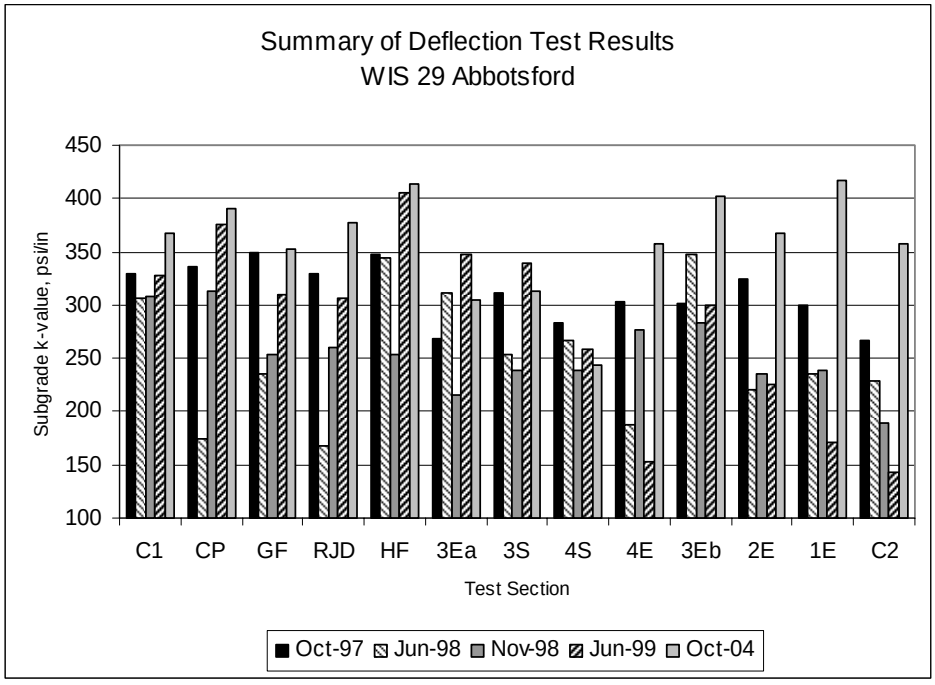


Figure 4.2.4: Backcalculated Subgrade k-values - WIS 29 Abbotsford

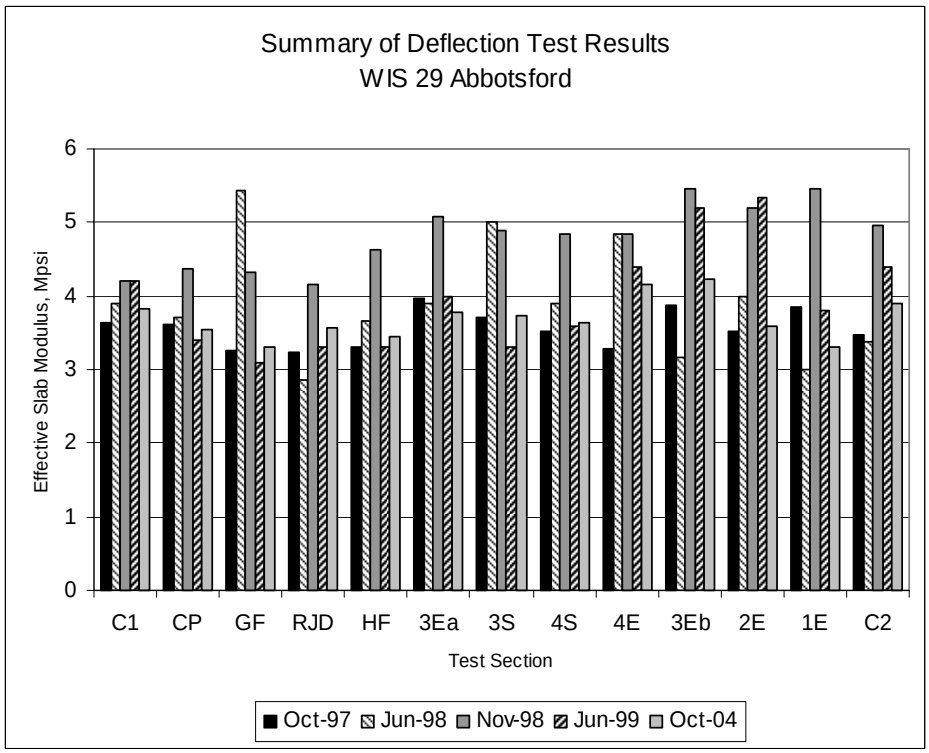


Figure 4.2.5: Backcalculated Slab Moduli - WIS 29 Abbotsford

Figures 4.2.6 and 4.2.7 illustrate mean outer wheel path total joint deflections obtained during the various test cycles. Data trends from testing conducted during the first year of service, illustrated in Figure 4.2.6, indicate general agreement between late season results (October 1997 and November 1998). Results from June 1998 testing indicate general uniformity amongst all test sections, with mean deflections markedly reduced from late season values. Data trends from subsequent testing, illustrated in Figure 4.2.7, again indicate reduced deflections during early season testing (June 1999) as compared to late season values (November 2002). For these two test series, there is general agreement between all test sections. The results from the final series of testing, completed in October 2004 illustrate wide variations in test section results, most likely due to temperature curling of the slabs. October 2004 testing was conducted over two days, with pavement surface temperatures beginning in the mid 40s and rising each day under generally cloudy skies. On the first day of testing, which included the testing of sections C1 through 4S, the deflection trends suggest initial upward curling transitioning to flat-slab to downward curling. This can be seen by the steep reductions in deflections as early testing progressed from sections C1 to HF, which is atypical when compared to previous late season results (October 1997, November 1998 and November 2002). By mid-day, continued testing in sections HF through 4S indicate upwards curling had been eliminated. Similar trends can be seen for the second day of testing, which was completed by mid-day and included testing in sections 4E through C2

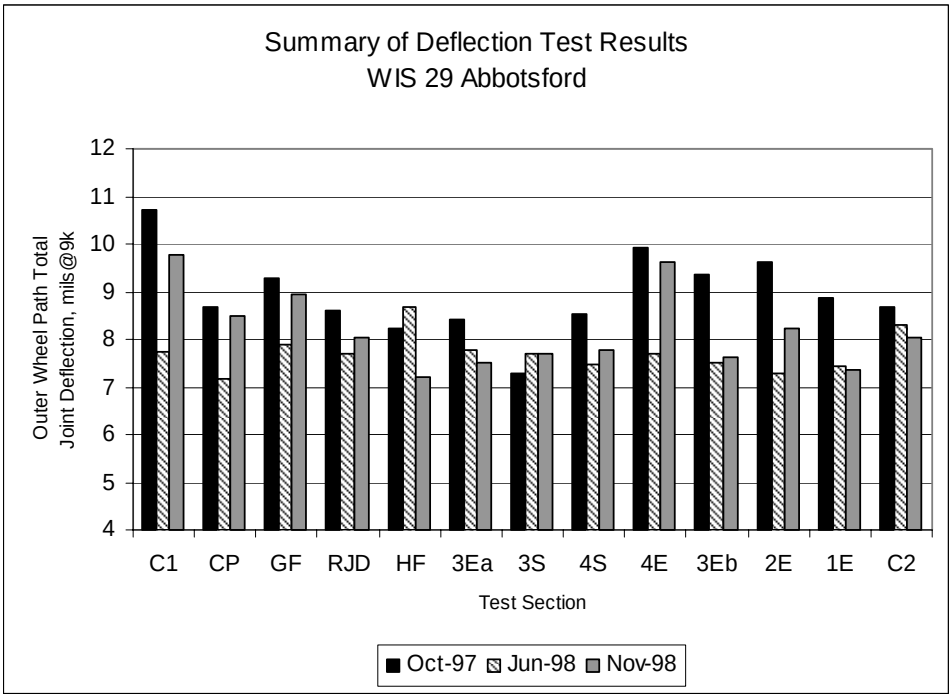


Figure 4.2.6: Transverse Joint Deflection Results - WIS 29 Abbotsford

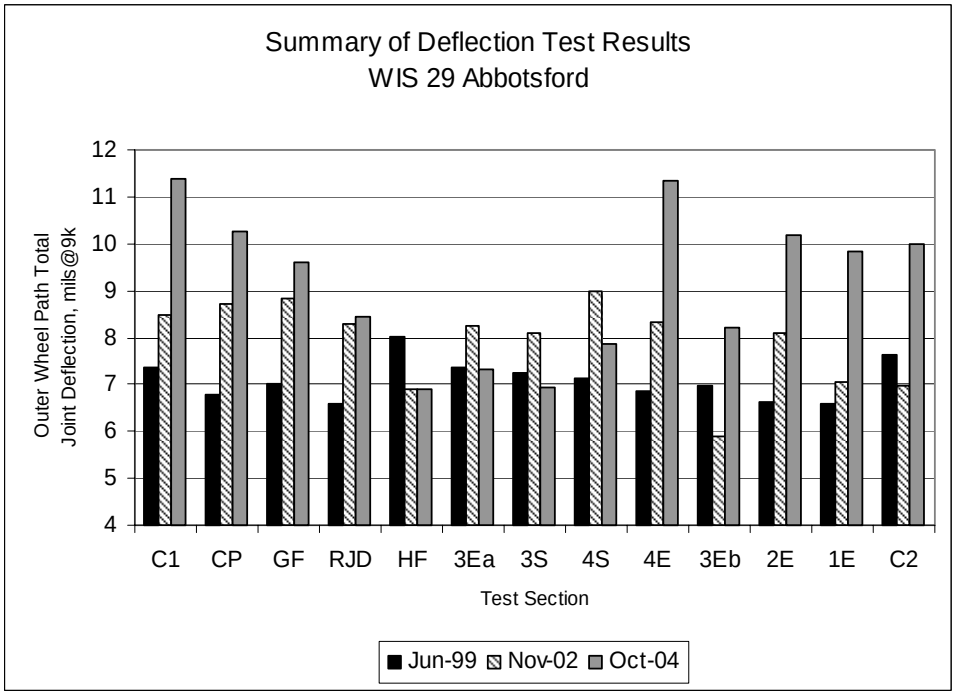


Figure 4.2.7: Transverse Joint Deflection Results - WIS 29 Abbotsford

4.2.6 Post-Paving Deflection Testing - WIS 29 Wittenberg

Post-paving deflection testing was initiated along the eastbound test sections on October 30, 1997 prior to opening to traffic. Pre-opening testing was not completed on the westbound test sections. Eastbound testing was completed approximately 2 weeks after paving, including center slab and transverse joint tests at mid-lane and outer wheel path locations. The analysis procedures outlined in Section 4.2.2 were used to estimate the subgrade dynamic k-value, the effective slab thickness (E_c assumed = 3.6 Mpsi), and the effective slab modulus (H_c assumed = 11 in) for each section. Normalized total joint deflections and joint deflection load transfer values were also computed. Tables 4.2.5 and 4.2.6 provide summary statistics for these computed values. As shown in Table 4.2.5, mean subgrade dynamic k-values are generally consistent throughout the eastbound test sections, with test section values ranging from 332 to 479 psi/in. Within-section variability is relatively high, with coefficients of variation ranging from 11.5 to 26.1%. The mean effective thicknesses are also quite similar between test sections, with backcalculated values ranging from 11.0 to 12.0 inches. Within-section variability is relatively low, with coefficients of variation ranging from 8.0 to 12.4%. The mean effective slab moduli are also quite similar between test sections, with backcalculated values ranging from 3.6 to 4.8 Mpsi. Within-section variability is relatively high, with coefficients of variation ranging from 22.6 to 37.6%.

The average transverse joint load transfers and total deflections are provided in Table 4.2.6. As shown, there is general uniformity among the load transfer measures between test sections, with the RJD composite section exhibiting somewhat lower load transfer, which is consistent with deflection test results obtained at WIS 29 Abbotsford. Total joint deflection values are also generally comparable between test sections with the stainless steel test section having somewhat lower values.

Table 4.2.5: Post-Paving Test Results - WIS 29 Wittenberg (Oct 1997)

Test Section	Subgrade Dynamic k-value		Effective Slab Thickness[1]		Effective Slab Modulus [2]	
	Mean psi/in	COV %	Mean in	COV %	Mean Mpsi	COV %
C1	352	12.3	12.0	8.0	4.8	23.7
RJD	421	21.4	11.0	8.3	3.6	22.6
FR	332	26.1	12.0	10.1	4.8	31.1
SS	423	11.5	11.7	12.4	4.5	37.6
C2	479	22.1	11.2	12.4	3.9	35.7

[1] Backcalculated assuming Ec = 3.6 Mpsi
[2] Backcalculated assuming Hc = 11 in

Table 4.2.6: Post-Paving Test Results - WIS 29 Wittenberg (Oct 1997)

Test Section	Mean Dynamic k-value psi/in	Mean Deflection Load Transfer, %		Mean Total Joint Deflection mils@9k	
		OWP[1]	Mid-Lane[2]	OWP[1]	Mid-Lane[2]
C1	352	87	83	6.92	6.53
RJD	421	82	81	6.63	6.11
FR	332	87	85	6.88	6.81
SS	423	89	88	6.14	5.62
C2	479	88	82	6.86	6.24
Overall Average		86.6	83.8	6.69	6.26

[1] OWP = outer wheel path of driving lane
[2] Mid-lane = center of driving lane

Additional deflection data was collected at subsequent times up to 7 years after paving. Figures 4.2.8 to 4.2.11 provide plots of comparative section data. The backcalculated pavement parameters illustrated in Figures 2.4.8 and 2.4.9 show marked variability between test periods. There is general agreement between test sections during each test period; however, the subgrade support within the FR, SS and TR sections appears markedly reduced during the June 1999 testing. Furthermore, subgrade support within the TR section appears lower than other sections throughout the test periods.

The outer wheelpath load transfer results provided in Figure 4.2.10 indicate general uniformity between sections with the exception of the November 1998 testing where the composite (RJD, FR) and the alternate dowel placement (1E) sections provide markedly reduced load transfer. Load transfers within these sections during subsequent testing are similar to other sections, which is contrary to expectations. The total joint deflection results displayed in Figure 4.2.11 indicate general uniformity amongst sections during each test period; however, joint deflections within the composite (RJD, FR) and stainless steel (SS) sections are somewhat higher than other sections during later test periods.

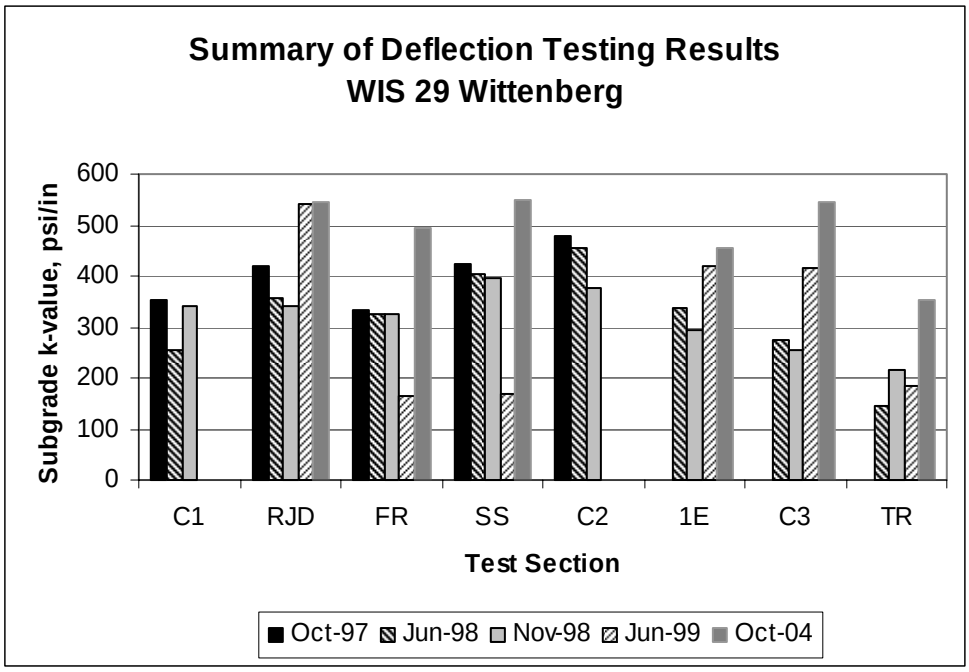


Figure 4.2.8: Backcalculated Subgrade k-values – WIS 29 Wittenberg

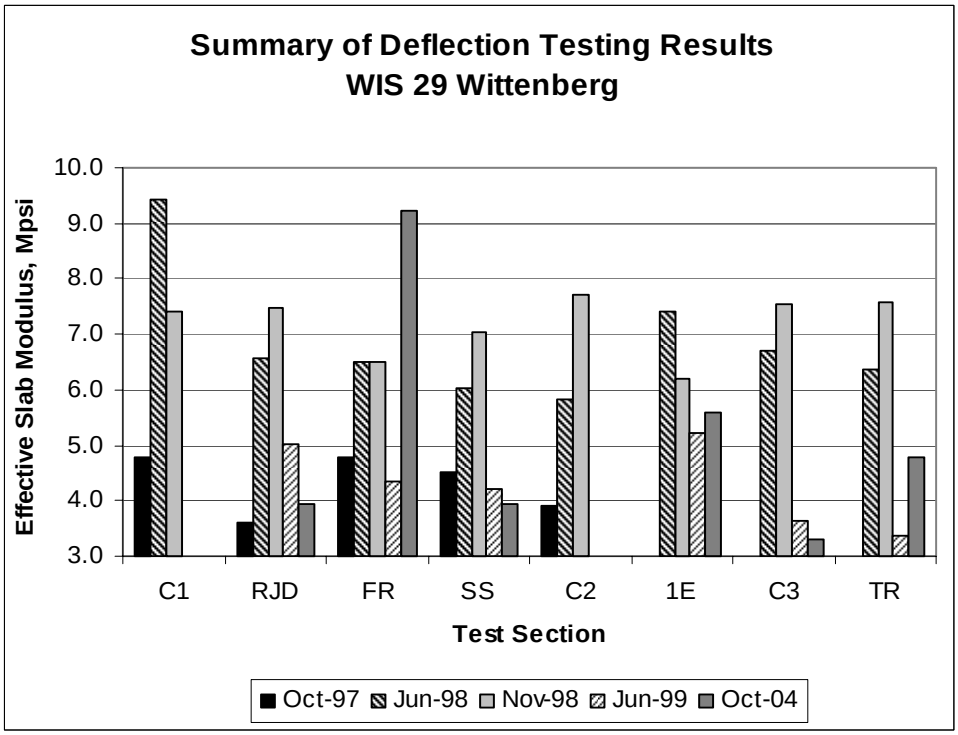


Figure 4.2.9: Backcalculated Slab Moduli – WIS 29 Wittenberg

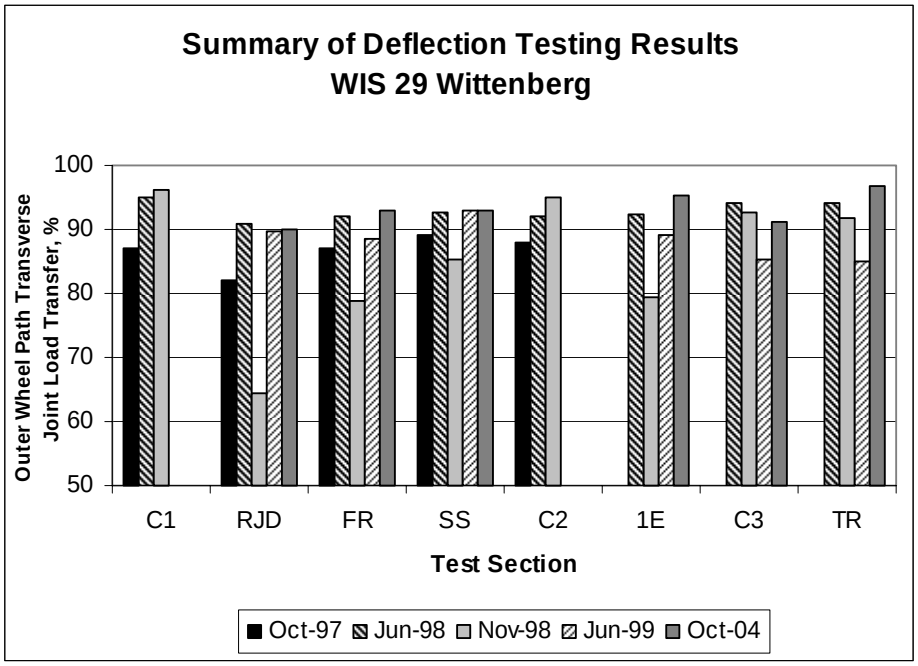


Figure 4.2.10: Transverse Joint Load Transfer – WIS 29 Wittenberg

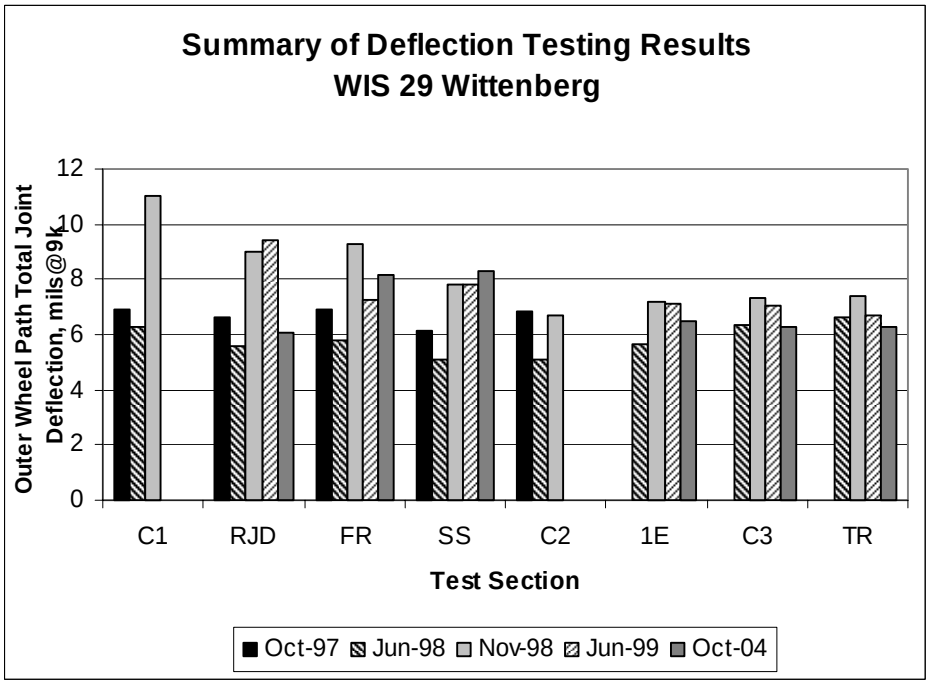


Figure 4.2.11: Total Joint Deflection – WIS 29 Wittenberg

4.2.7 - Post-Paving Deflection Testing - WIS 29 Tilleda

Post-paving deflection testing was conducted along the westbound test sections on October 10, 1999 prior to opening to traffic. Testing was completed approximately 4 weeks after paving and included center slab and transverse joint tests along the Passing and driving lanes. The analysis procedures outlined in Section 4.2.2 were used to estimate the dynamic subgrade k-value and effective slab thickness (E_c assumed = 3.6 Mpsi) for each section. Normalized total joint deflections and joint deflection load transfer values were also computed. Tables 4.2.7 and 4.2.8 provide summary statistics for these computed values. Section average values are also provided in Figures 4.2.12 to 4.2.15.

The backcalculated dynamic k-values provided in Table 4.2.7 and Figure 4.2.12 indicate a general trend toward increased subgrade stiffness moving from east to west, with the exception of the westernmost test section (TS1) which exhibits the lowest support. The average backcalculated effective slab thicknesses provided in Table 4.2.7 and Figure 4.2.13 are in good agreement with design thicknesses of each travel lane. Total joint deflections provided in Table 4.2.8 and Figure 4.2.14 indicate a general trend of reduced deflection moving east to west, with a noted increase for the westernmost section. These results are in agreement with interior k-value trends noted earlier.

Transverse joint load transfer values provided in Table 4.2.8 and Figure 4.2.15 indicate generally low values (78.2 to 82.0) for the two sections with highest subgrade support (TS3, TS2). The remaining sections exhibit generally uniform load transfer ranging from 86.3 to 89.1%.

Deflection tests were again conducted in October 2004. Figures 2.4.16 and 2.4.17 illustrate average section values of backcalculated slab parameters. As shown in Figure 2.4.16, the backcalculated average dynamic k–values are significantly different from original values, most notably within the passing lane where dynamic k-values are appreciably lower. The backcalculated effective slab modulus values provided in Figure 2.4.17 also display erratic behavior, which roughly parallels the results of the k-values in that higher k-values are associated with significantly reduced moduli values. This effect typically occurs with excessive temperature curling during testing, which was not evident during this test series.

Table 4.2.7: Post-Paving Test Results - WIS 29 Tilleda (Oct 1999)

Test Section	Dynamic Subgrade k-value				Effective Slab Thickness			
	Driving Lane		Passing Lane		Driving Lane		Passing Lane	
	Mean Psi/in	COV %	Mean Psi/in	COV %	Mean in	COV %	Mean in	COV %
TS4	420	31.7	375	25.6	10.4	15.6	9.0	13.2
TS3	608	29.6	422	35.0	9.9	11.7	9.1	10.5
STD	485	34.1	438	24.9	10.0	9.7	9.0	10.8
TS2	605	21.3	567	21.3	9.2	7.1	8.0	8.4
TS1	324	40.7	296	33.5	10.4	19.2	10.3	11.7

(1) Backcalculated assuming Ec = 3.6 Mpsi

Table 4.2.8: Post-Paving Test Results - WIS 29 Tilleda (Oct 1999)

Test Section	Total Joint Deflection				Deflection Load Transfer			
	Driving Lane		Passing Lane		Driving Lane		Passing Lane	
	Mean mils@9k	COV %	Mean mils@9k	COV %	Mean %	COV %	Mean %	COV %
TS4	12.48	8.3	14.08	7.7	87.3	5.5	87.5	4.1
TS3	10.66	10.1	12.22	7.1	81.0	12.4	78.2	29.6
STD	8.96	5.2	10.00	5.1	89.1	4.7	88.3	5.0
TS2	7.74	6.5	9.49	7.1	81.8	6.1	82.0	9.8
TS1	9.70	7.0	9.90	7.1	86.7	4.3	86.3	2.8

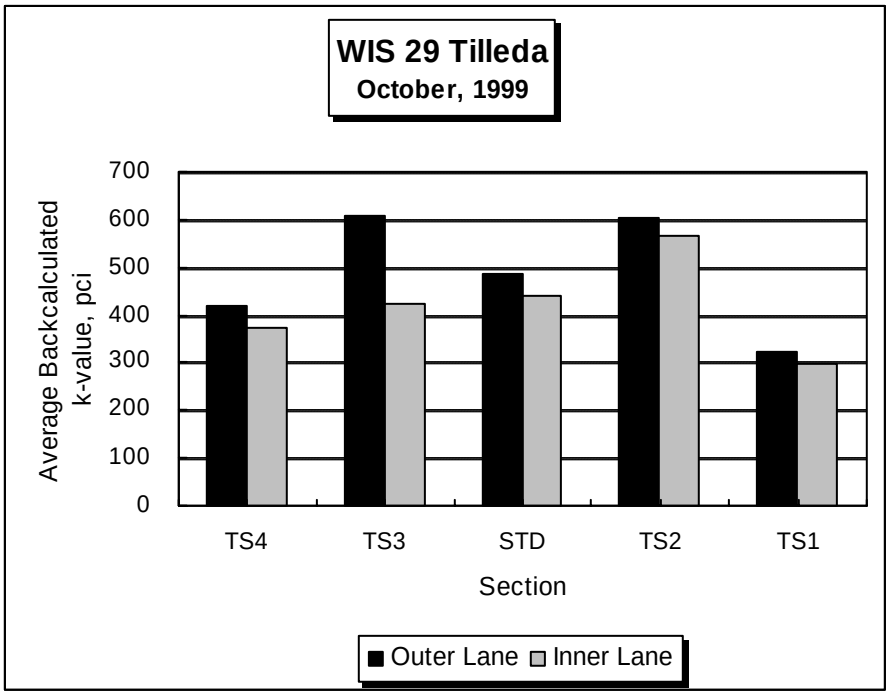


Figure 4.2.12: Backcalculated Subgrade k-values – WIS 29 Tilleda

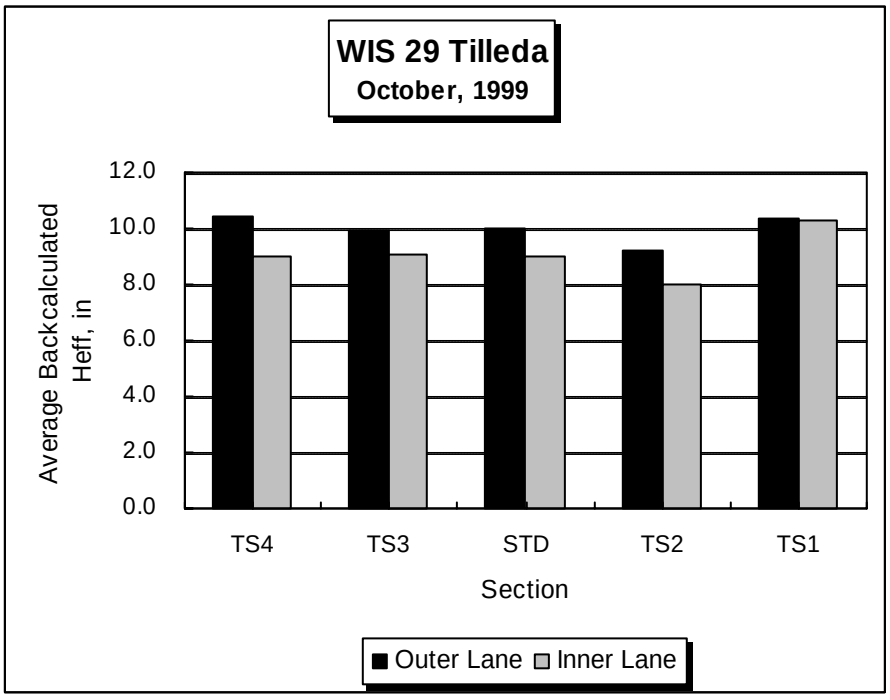


Figure 4.2.13: Backcalculated Effective Slab Thickness – WIS 29 Tilleda

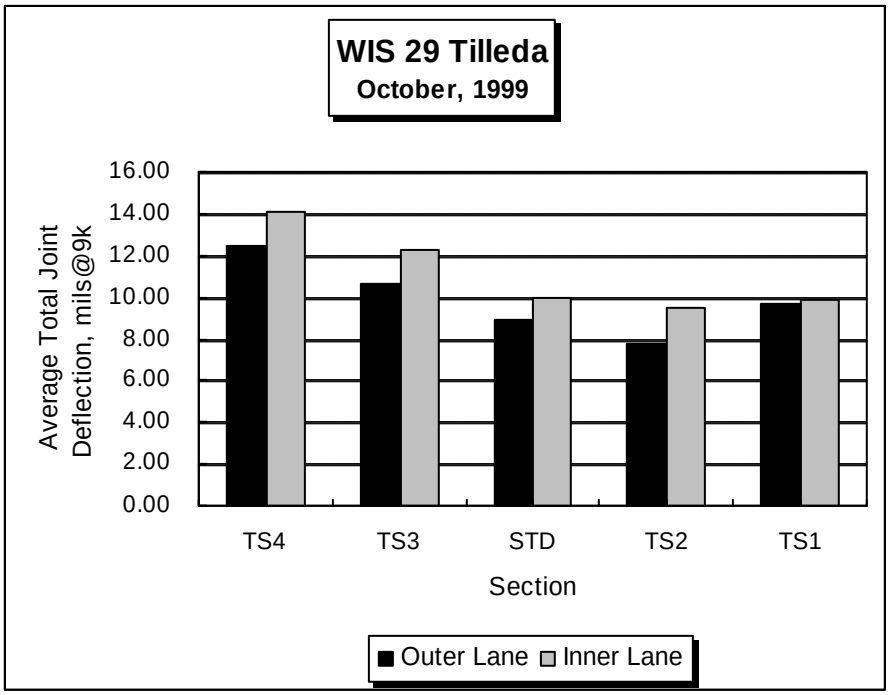


Figure 4.2.14: Total Joint Deflections – WIS 29 Tilleda

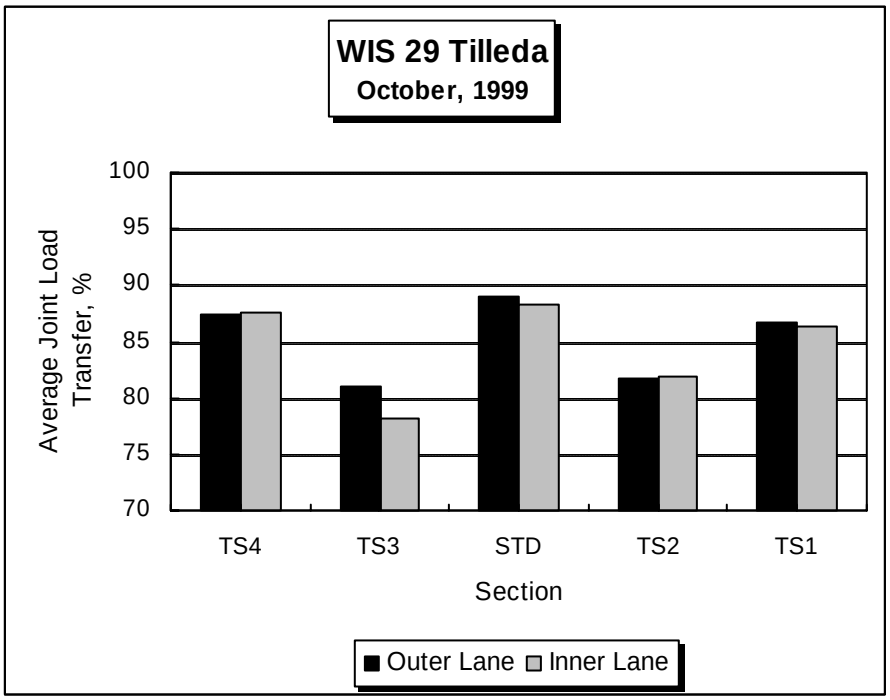


Figure 4.2.15: Joint Load Transfer – WIS 29 Tilleda

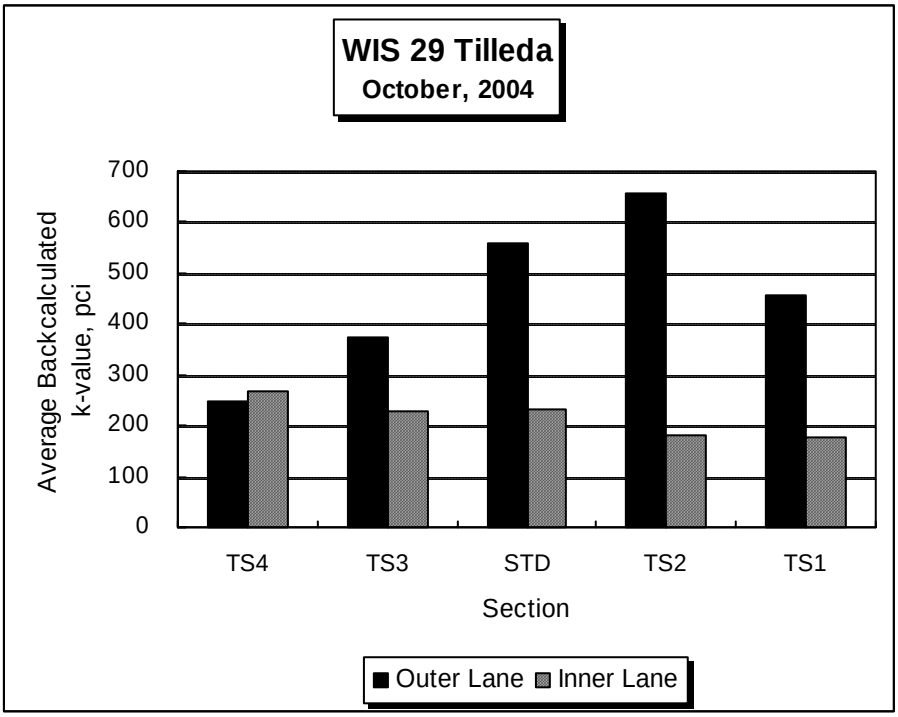


Figure 4.2.16: Backcalculated Subgrade k-values – WIS 29 Tilleda

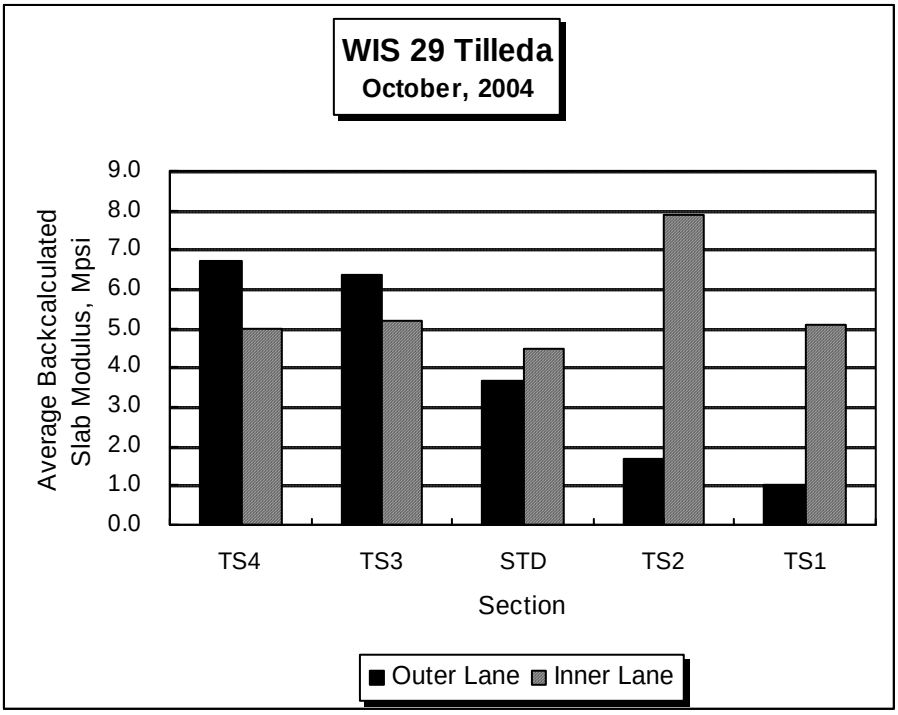


Figure 4.2.17: Backcalculated Effective Slab Modulus – WIS 29 Tilleda

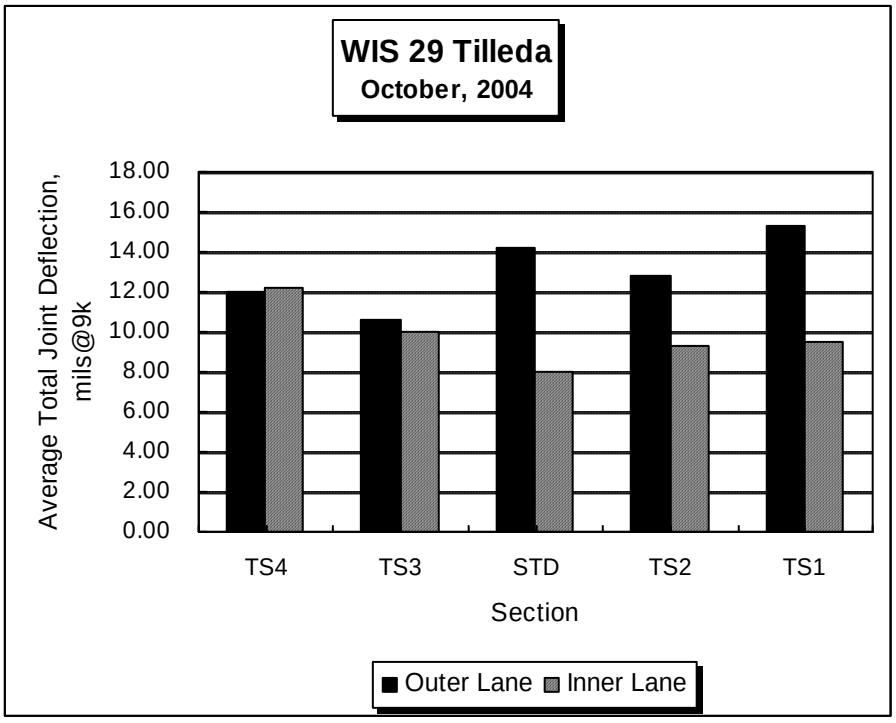


Figure 4.2.18: Total Joint Deflection – WIS 29 Tilleda

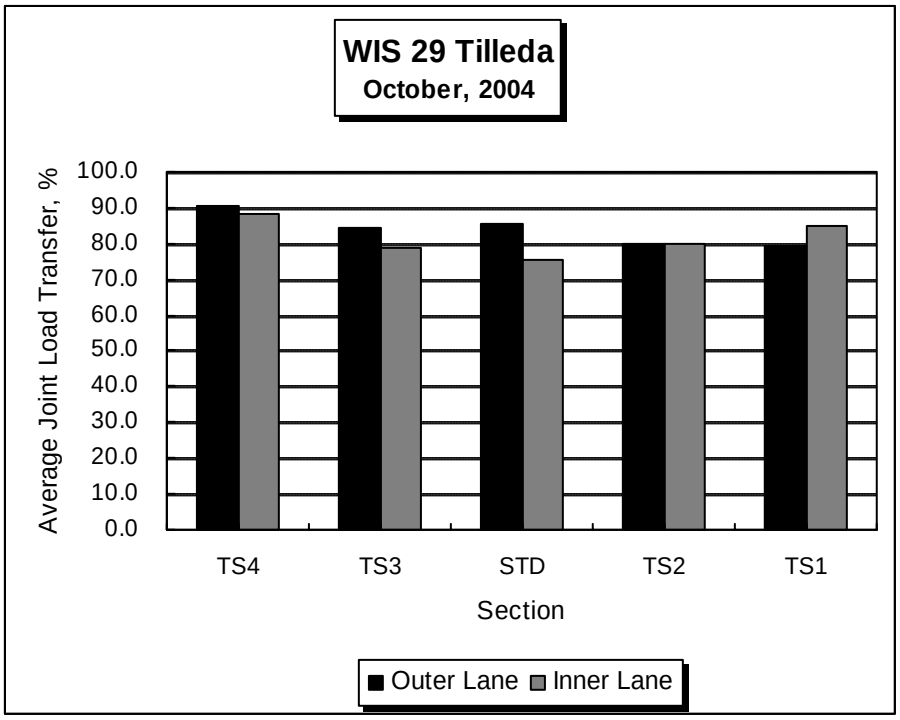


Figure 4.2.19: Total Joint Deflection – WIS 29 Tilleda

The joint deflection results displayed in Figure 4.2.18 are in general agreement with results obtained from pre-opening testing with the noted exception within the driving lanes of STD, TS2 and TS1 where deflections are substantially higher. Combined with the results of backcalculated slab parameters, it appears these anomalies may be the result of excessive upward slab warping within these sections. The joint deflection load transfer values displayed in Figure 4.2.19 are relatively consistent with pre-opening test results.

4.3 Ride Quality Measures

Ride quality measures were collected by WisDOT using their high-speed survey vehicle. Data was collected throughout the entire length of construction for each test section. International Roughness Index (IRI) values were provided for one or both wheelpaths for each test section in metric units of m/km. In general, IRI values below 1.5 m/km are indicative of a smooth ride while values in excess of 2.5 indicate a rough ride. When both wheelpaths were provided, these values were averaged to provide a general indicator of the ride quality within that test section. The provided IRI values were recorded at different periods from year to year and, as such, it is difficult to establish yearly trends for the various sections. Instead, IRI variations within each test section were examined to identify performance variations that may be attributed to any specific design alternate.

4.3.1 WIS 29 - Abbotsford

Figures 4.3.1 to 4.3.5 provide summary plots of yearly IRI values collected along the driving and passing lanes along WIS 29 Abbotsford during the period of 2000-2004. As shown in these figures, the passing lane values are predominantly lower than driving lane values, as expected. However, Year 2000 IRI values indicate slight higher IRI values within the passing lane of the westernmost sections (C1, CP, GF, RJD). The overall average IRI values within each lane are provided in Table 4.3.1. For comparative purposes, the yearly average IRI values within various design subsets are provided in Table 4.3.2.

Table 4.3.1: Overall Average IRI Values, WIS 29 Abbotsford

Year	Driving Lane IRI	Passing Lane IRI
2000	1.25	1.17
2001	1.30	1.12
2002	1.23	1.10
2003	1.67	1.68
2004	1.41	1.24

Table 4.3.2: Group Average IRI Values, WIS 29 Abbotsford

Lane	Group Subset	Year				
		2000	2001	2002	2003	2004
Driving	C1, C2	1.20	1.19	1.16	2.02	1.31
	RJD, GF, CP	1.24	1.45	1.19	1.51	1.31
	HF	1.39	1.78	1.44	1.49	1.63
	4E, 4S	1.24	1.22	1.31	1.59	1.54
	3Ea,3Eb,3S	1.27	1.24	1.24	1.52	1.41
	2E	1.29	0.92	1.03	1.93	1.48
	1E	1.18	1.36	1.33	2.06	1.38
Passing	C1, C2	1.19	1.05	1.01	1.49	1.19
	RJD, GF, CP	1.29	1.28	1.14	1.68	1.22
	HF	1.30	1.45	1.37	1.59	1.43
	4E, 4S	1.06	1.07	1.11	1.93	1.27
	3Ea,3Eb,3S	1.15	1.08	1.09	1.66	1.24
	2E	1.10	0.71	0.84	1.47	1.16
	1E	1.03	1.14	1.18	1.90	1.21

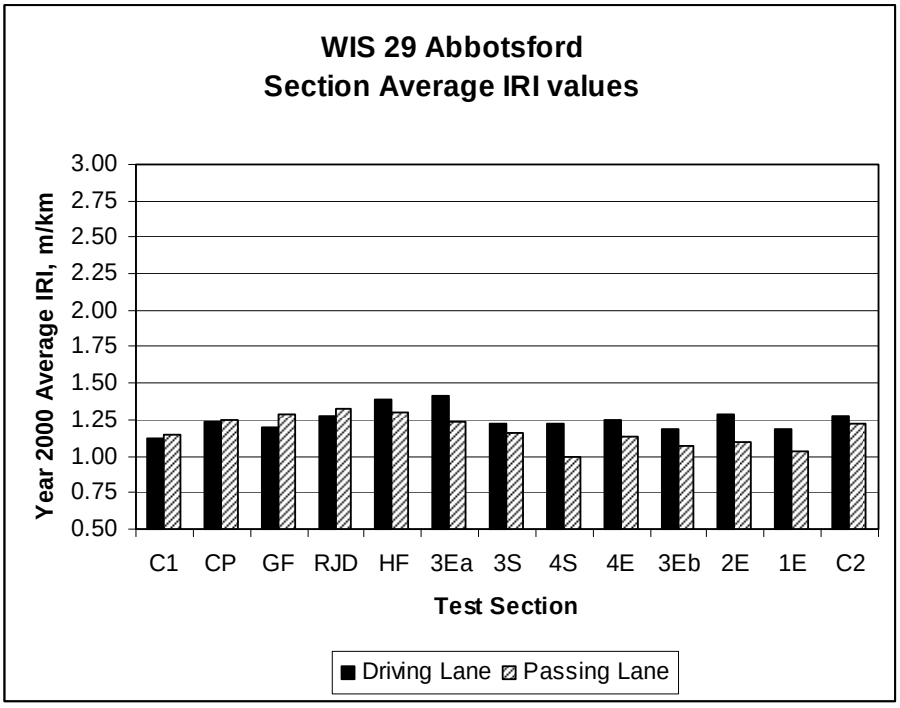


Figure 4.3.1 Year 2000 Profiler Readings - WIS 29 Abbotsford

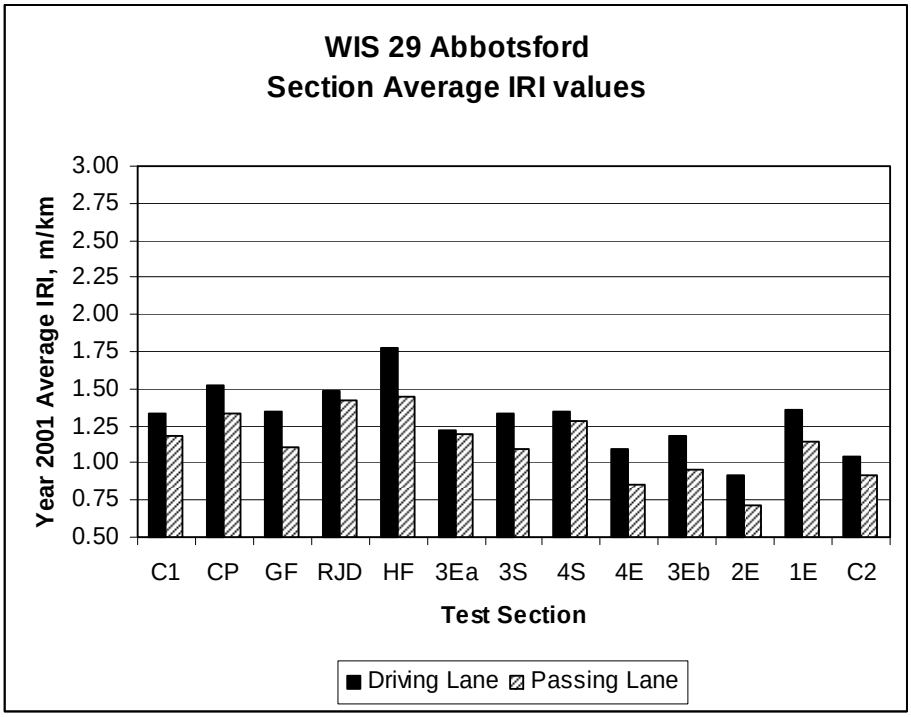


Figure 4.3.2 Year 2001 Profiler Readings - WIS 29 Abbotsford

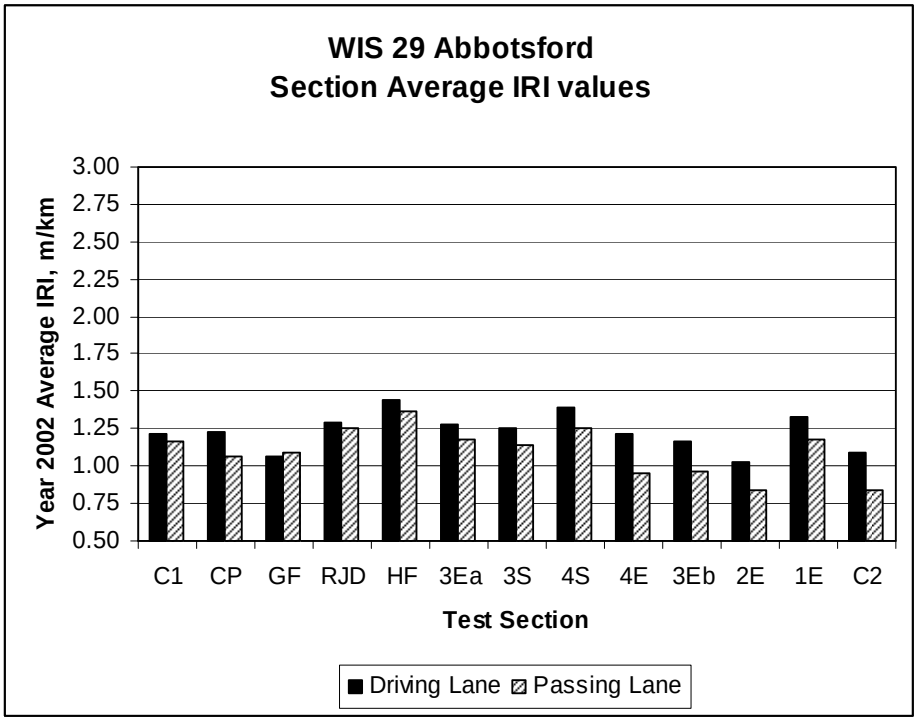


Figure 4.3.3 Year 2002 Profiler Readings - WIS 29 Abbotsford

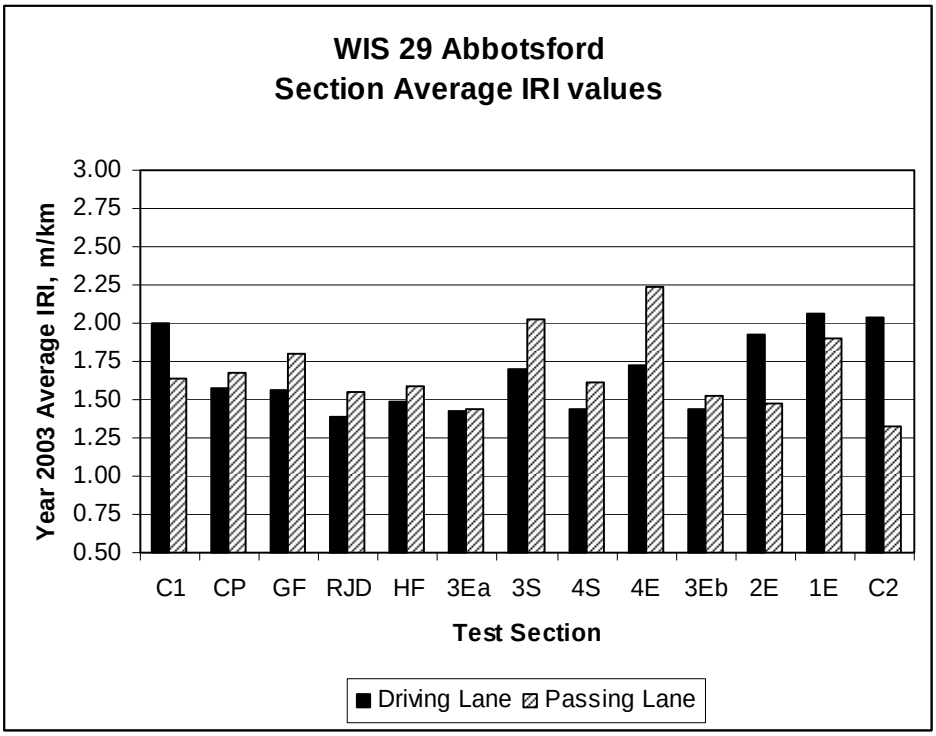


Figure 4.3.4 Year 2003 Profiler Readings - WIS 29 Abbotsford

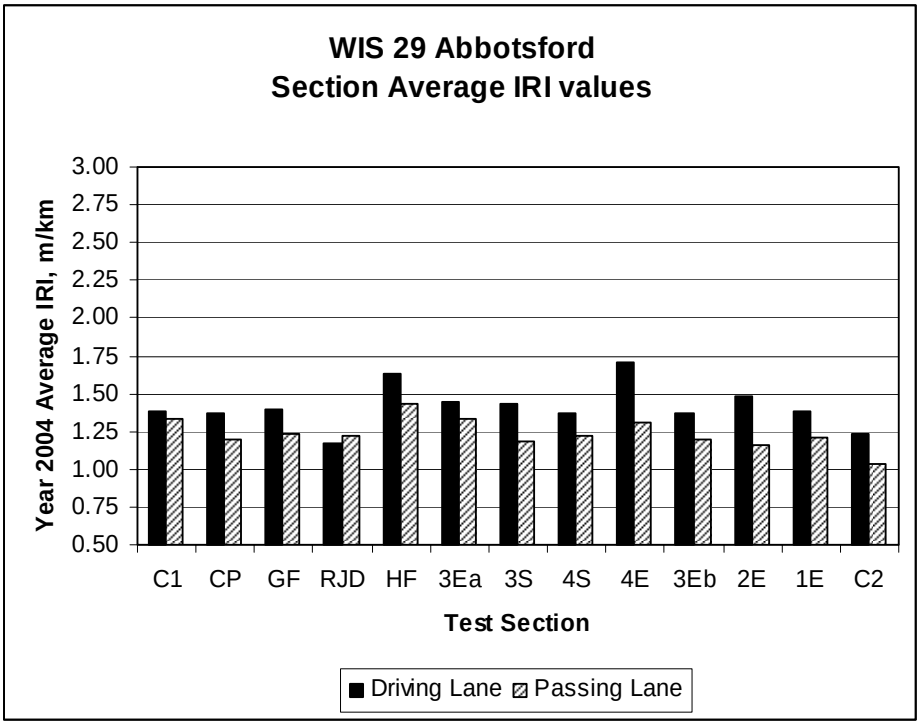


Figure 4.3.5 Year 2004 Profiler Readings - WIS 29 Abbotsford

The data provided in Table 4.3.2 can be utilized to asses the relative performance of the various test sections in relation to the standard design control group (C1,C2). The sections incorporating composite dowels (RJD, GF, CP) have consistently higher IRI values than the control group in both the passing and driving lanes for all years except 2003. Data from year 2003 appears to be biased for an unknown reason, yielding higher than expected IRI values for all sections. The hollow-filled section (HF) generally has the highest IRI values amongst all groups which may, to a large extent, be due to the fact that this is the shortest section and it contains a faulted mid-panel crack across both lanes which disproportionately contributes to higher roughness. The alternate dowel placement groups generally indicate higher roughness than the control group with the exception of alternate 2E, which includes 4 dowels in the outer wheel path of the driving lane. This section generally has the lowest IRI values amongst all section groups.

4.3.2 WIS 29 Wittenberg

Figures 4.3.6 to 4.3.10 provide summary plots of yearly IRI values collected along the driving and passing lanes along WIS 29 Wittenberg during the period of 2000-2004. The overall average IRI values within each lane are provided in Table 4.3.3. For comparative purposes, the yearly average IRI values within various design subsets are provided in Table 4.3.4. The IRI trends presented in the figures and tables are difficult to summarize due to the variability in performance trends. In general, it can be stated that the overall average passing lane IRI values are typically lower than driving lane values, as shown in Table 4.3.3. However, the data provided in Table 4.3.4 indicates the passing lane of the composite dowel section group (RJD, FR) consistently has IRI values equal to or greater than the driving lane values. Furthermore, based on the most recent data collected (2003 & 2004), the group average IRI values within the composite dowel section group are the highest amongst all groups. The best performance, in terms of the lowest average IRI value, is observed for the variable thickness slab (TR) section.

Table 4.3.3 Overall Average IRI Values – WIS 29 Wittenberg

Year	Eastbound Sections		Westbound Sections	
	Driving Lane	Passing Lane	Driving Lane	Passing Lane
2000	1.71	1.60	1.43	1.36
2001	1.29	0.93	1.48	0.90
2002	1.38	1.34	1.41	1.15
2003	1.34	1.35	1.32	1.18
2004	1.35	1.24	1.32	1.15

Table 4.3.4 Group Average IRI Values – WIS 29 Wittenberg

Year	Group	Group Average IRI Value	
		Driving Lane	Passing Lane
2000	C1, C2, C3	1.67	1.51
	RJD, FR	1.65	1.65
	SS	1.78	1.67
	1E	1.40	1.40
	TR	1.33	1.17
2001	C1, C2, C3	1.44	1.39
	RJD, FR	1.04	1.48
	SS	1.44	1.42
	1E	0.77	0.87
	TR	0.63	0.66
2002	C1, C2, C3	1.45	1.37
	RJD, FR	1.23	1.37
	SS	1.48	1.45
	1E	1.31	1.28
	TR	1.29	0.88
2003	C1, C2, C3	1.29	1.22
	RJD, FR	1.44	1.46
	SS	1.41	1.25
	1E	1.34	1.14
	TR	1.29	1.18
2004	C1, C2, C3	1.28	1.20
	RJD, FR	1.44	1.48
	SS	1.40	1.30
	1E	1.28	1.24
	TR	1.10	0.94

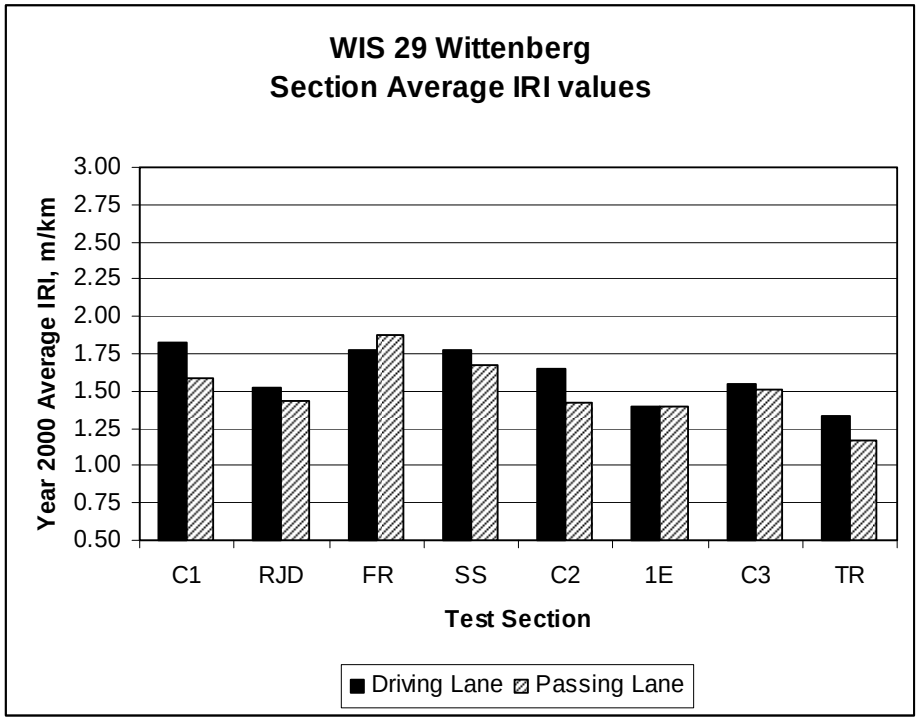


Figure 4.3.6 Year 2000 Profiler Readings - WIS 29 Wittenberg

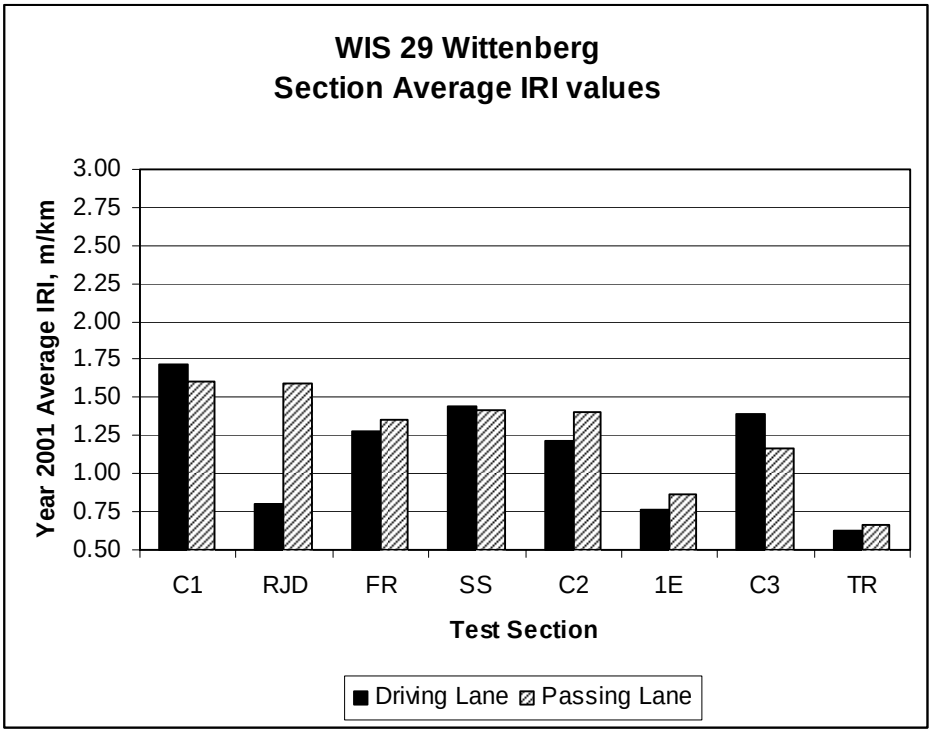


Figure 4.3.7 Year 2001 Profiler Readings - WIS 29 Wittenberg

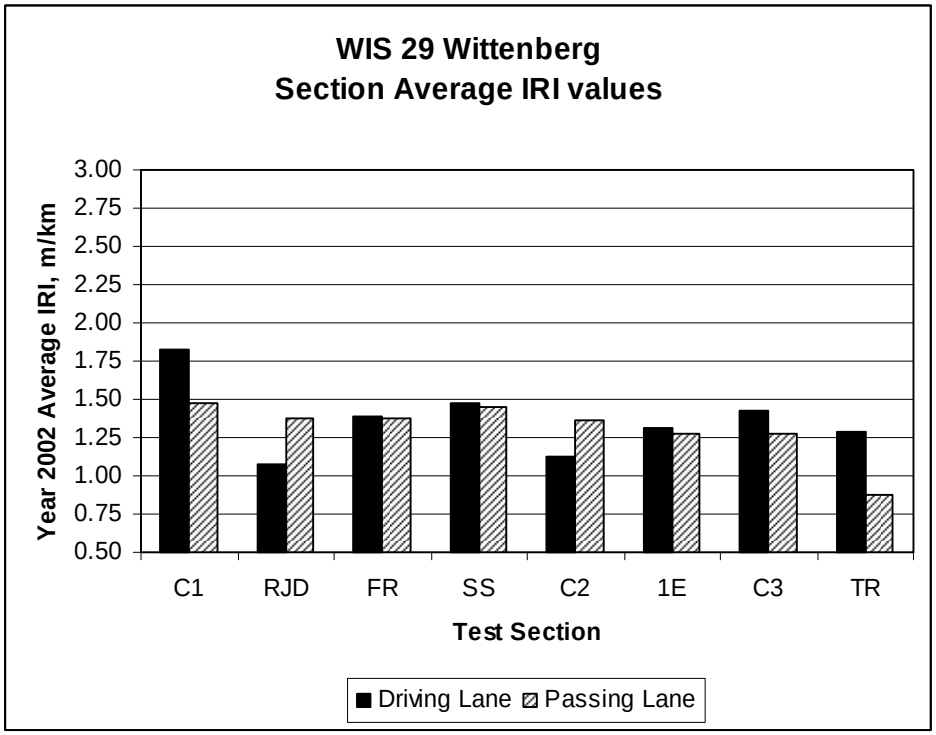


Figure 4.3.8 Year 2002 Profiler Readings - WIS 29 Wittenberg

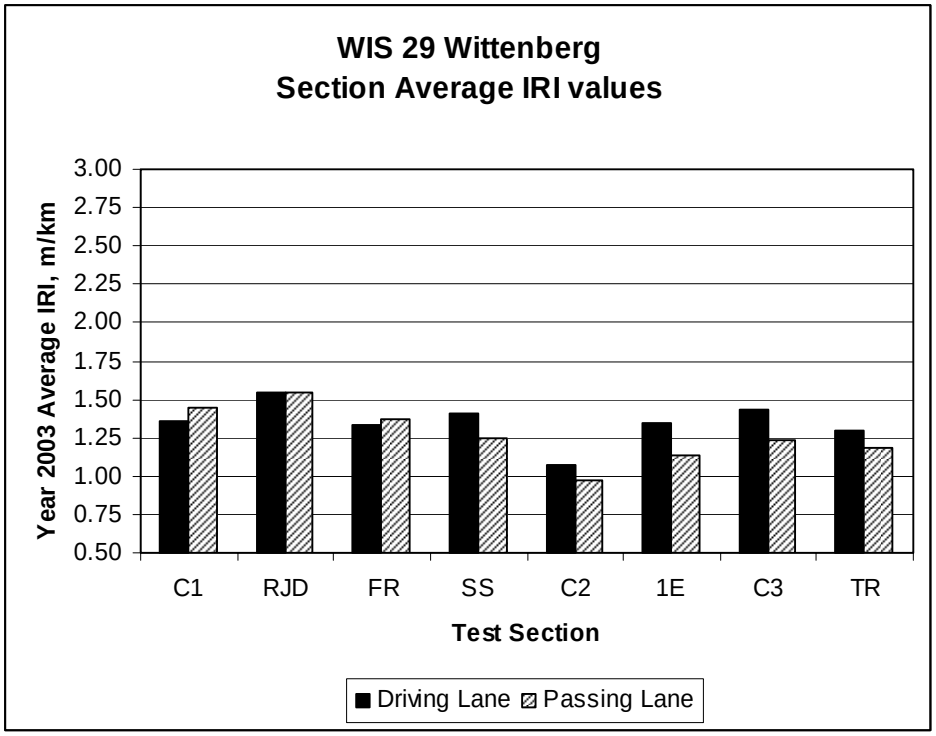


Figure 4.3.9 Year 2003 Profiler Readings - WIS 29 Wittenberg

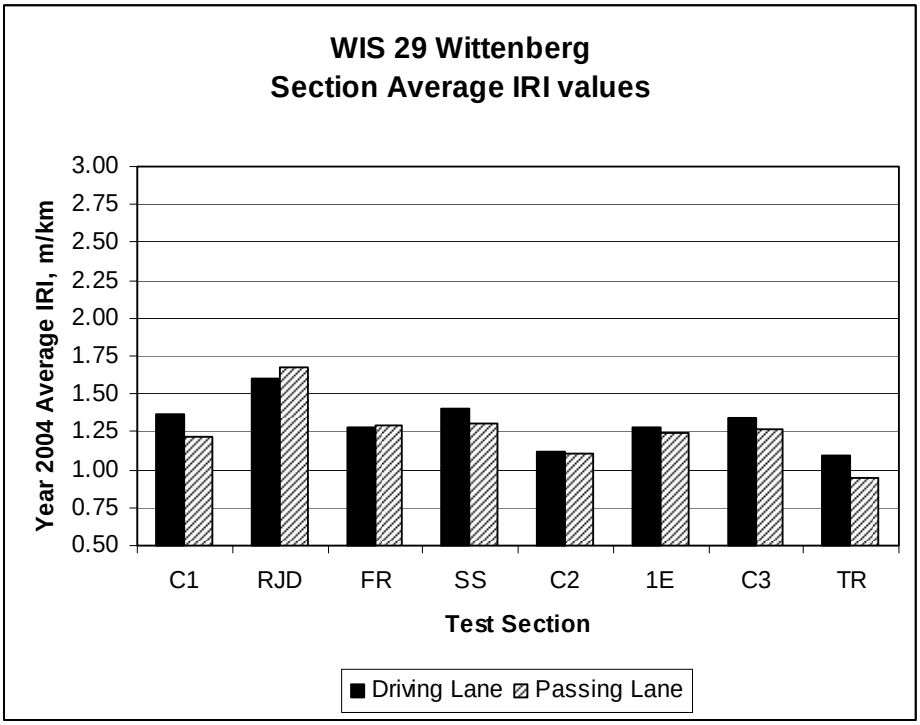


Figure 4.3.10 Year 2004 Profiler Readings - WIS 29 Wittenberg

4.3.3 WIS 29 Tilleda

Figures 4.3.11 to 4.3.13 provide summary plots of yearly IRI values collected along the driving and passing lanes along WIS 29 Tilleda during the period of 2002-2004. The overall average IRI values within each lane are provided in Table 4.3.5. For comparative purposes, the yearly average IRI values within various design sections are provided in Table 4.3.6.

The IRI trends presented in the figures and tables indicate that the overall and section average passing lane IRI values are consistently lower than driving lane values. Based on all data provided in Table 4.3.6, the best performance, in terms of the lowest average IRI value, is seen for the uniform slab thickness with one-way surface and base layer drainage (TS1). The one-way surface and base layer drainage section with variable slab thickness across both lanes (TS2) is also performing as good or better than the control section (STD) in terms of ride. Performance of the widened passing lane, in terms of

section average IRI values, is poorer than the section with the standard 12 ft. width passing lane (TS1). TS3, with variable slab thickness in the passing lane, two-way surface and one-way base drainage, consistently shows the roughest ride (i.e., highest IRI).

Table 4.3.5 Overall average IRI Values – WIS 29 Tilleda

Year	Overall Average IRI Values	
	Driving Lane	Passing Lane
2002	1.62	1.42
2003	2.07	1.70
2004	1.41	1.15

Table 4.3.6 Section Average IRI Values – WIS 29 Tilleda

Year	Group	Section Average IRI Value	
		Driving Lane	Passing Lane
2002	TS4	1.69	1.47
	TS3	2.30	1.75
	STD	1.47	1.39
	TS2	1.50	1.37
	TS1	1.14	1.10
2003	TS4	1.89	1.67
	TS3	2.95	2.16
	STD	1.94	1.88
	TS2	1.93	1.45
	TS1	1.66	1.36
2004	TS4	1.34	1.10
	TS3	2.08	1.50
	STD	1.33	1.22
	TS2	1.29	1.09
	TS1	1.01	0.85

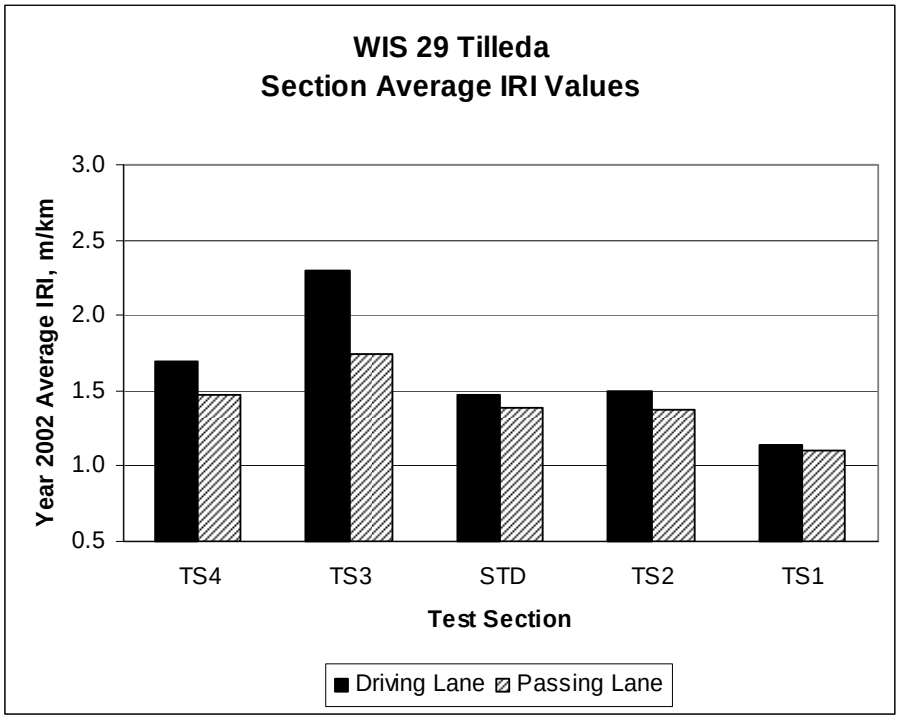


Figure 4.3.11 Year 2002 Profiler Readings - WIS 29 Tilleda

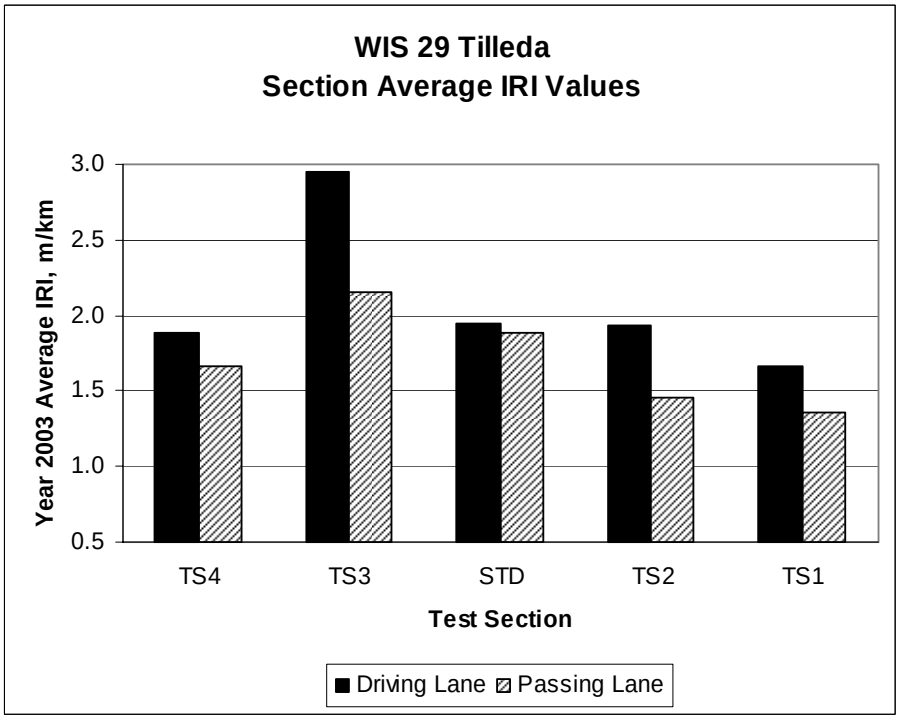


Figure 4.3.12 Year 2003 Profiler Readings - WIS 29 Tilleda

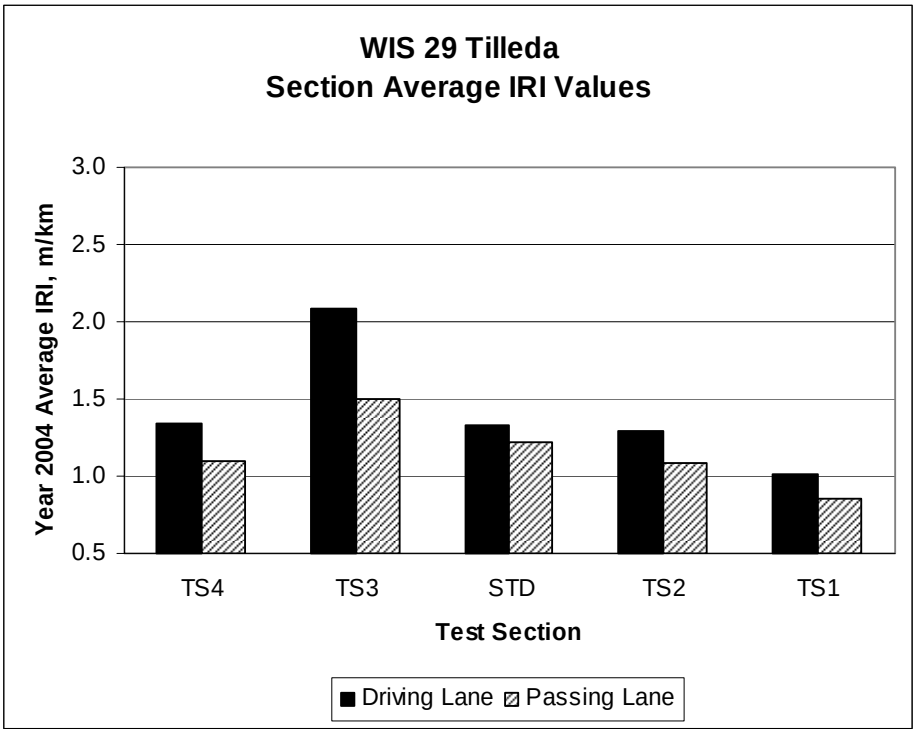


Figure 4.3.13 Year 2004 Profiler Readings - WIS 29 Tilleda

4.4 Distress Measures

Distress measures including joint spalling, slab cracking and joint faulting were recorded during FWD testing periods by Marquette University staff. Additional measures were made at selected times when no FWD measurements were made. Distress measures were visually identified and manually recorded. Joint faulting measurements were made using a portable fault meter provided by WisDOT staff, which provides for measurements with a resolution of approximately 1/16 inch.

4.4.1 WIS 29 Abbotsford

Distress measures recorded during the year 2004 survey are provided in Figures 4.4.1 to 4.4.4. As shown there is a limited amount of joint spalling and faulting and more extensive joint chipping. Observed joint chipping is related to the saw cutting of the transverse joints where localized coarse aggregates become dislodged soon after construction. Figure 4.4.5 provides a photo of a typical transverse joint with chipping. The

slab cracking data provided in Figure 4.4.4 indicates five test sections contain cracks. There was one low-severity transverse or longitudinal crack per section except for section 3S, which contained a longitudinal crack along nine consecutive slabs. This crack initiated during the second year of service and is most likely related to an ineffective parting strip installed during construction.

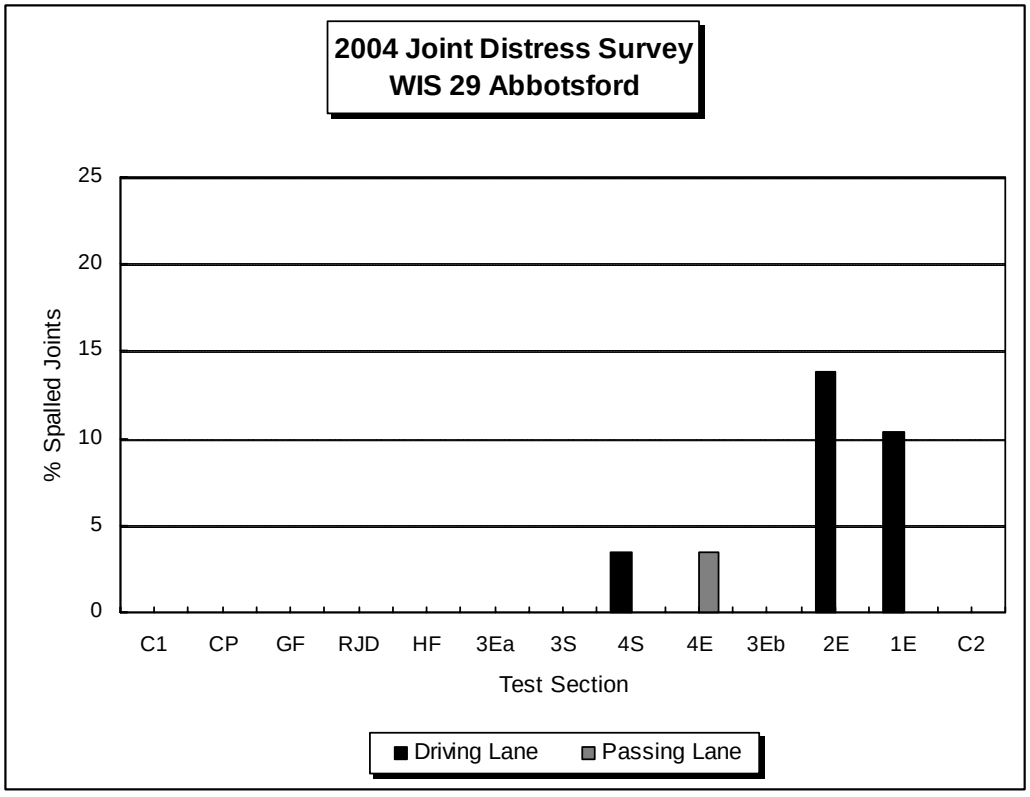


Figure 4.4.1 Joint Spalling Data – WIS 29 Abbotsford

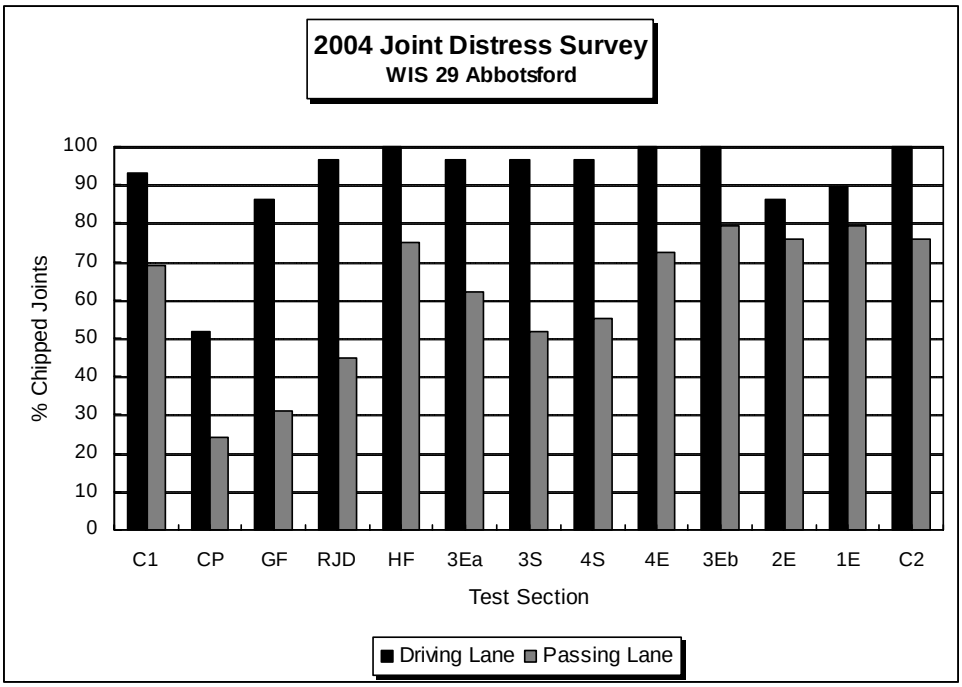


Figure 4.4.2 Chipped Joint Data – WIS 29 Abbotsford

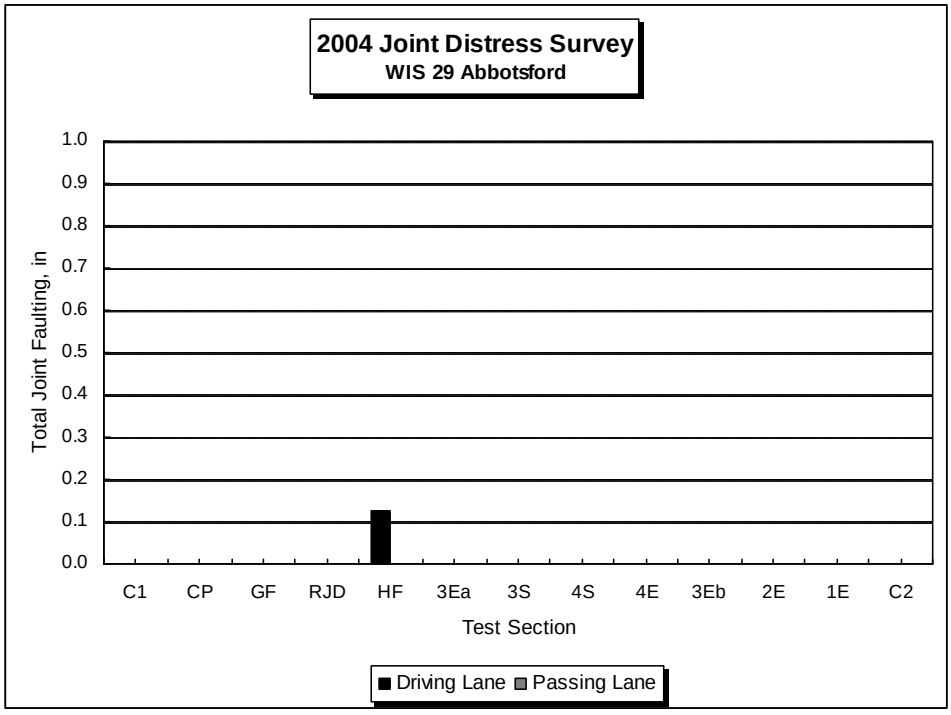


Figure 4.4.3 Joint Faulting Data – WIS 29 Abbotsford

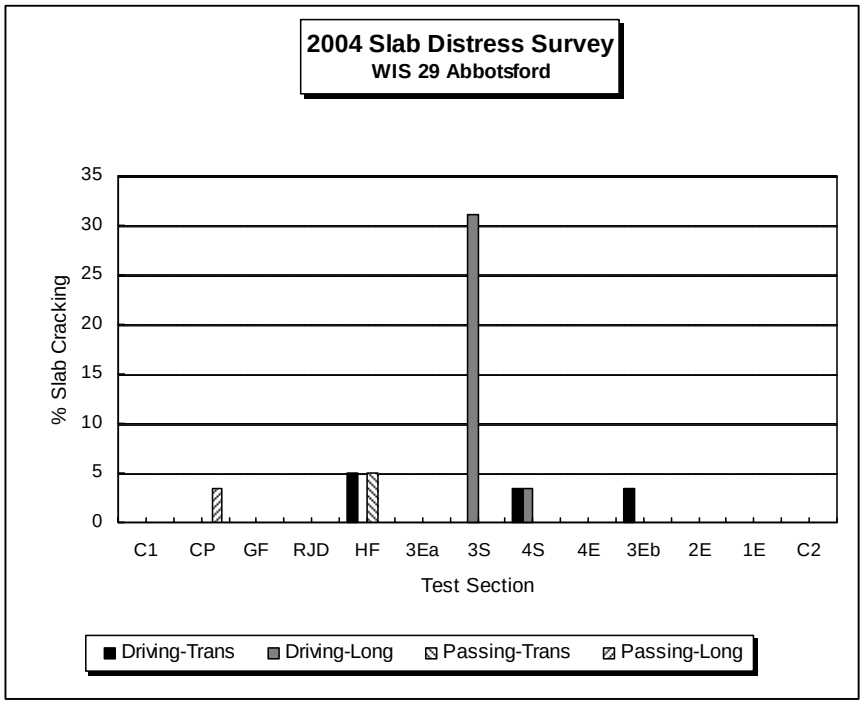


Figure 4.4.4 Slab Cracking Data – WIS 29 Abbotsford

Figure 4.4.5 Typical Chipped Joint – WIS 29 Abbotsford

4.4.2 WIS 29 Wittenberg

Distress measures recorded during the year 2004 survey are provided in Figures 4.4.6 to 4.4.8. As shown there is a limited amount of joint spalling and faulting and more extensive joint chipping which has been evident since soon after construction. In general, all sections are performing well with a limited amount of joint faulting measured within the FRP composite and stainless steel test sections. The total faulting measured was across one joint with the RJD and SS section and 2 joints within the FR section.

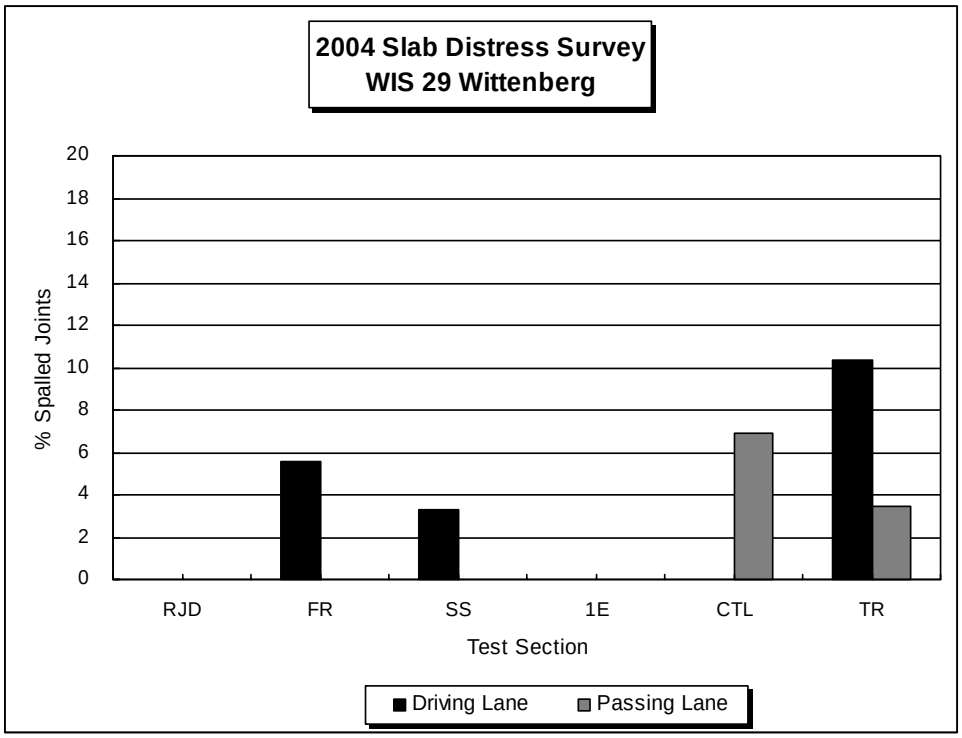


Figure 4.4.6 Joint Spalling Data – WIS 29 Wittenberg

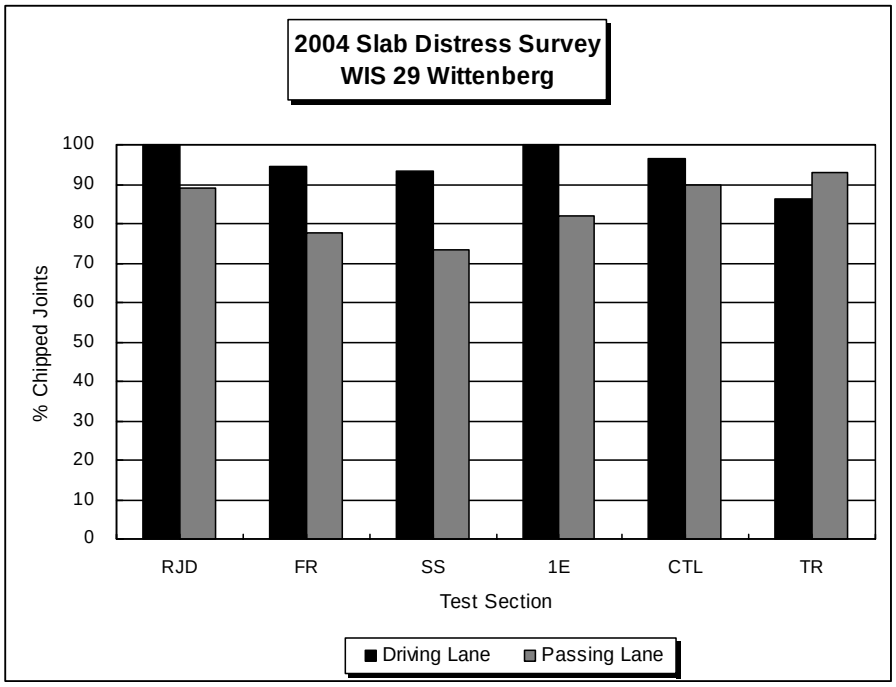


Figure 4.4.7 Chipped Joint Data – WIS 29 Wittenberg

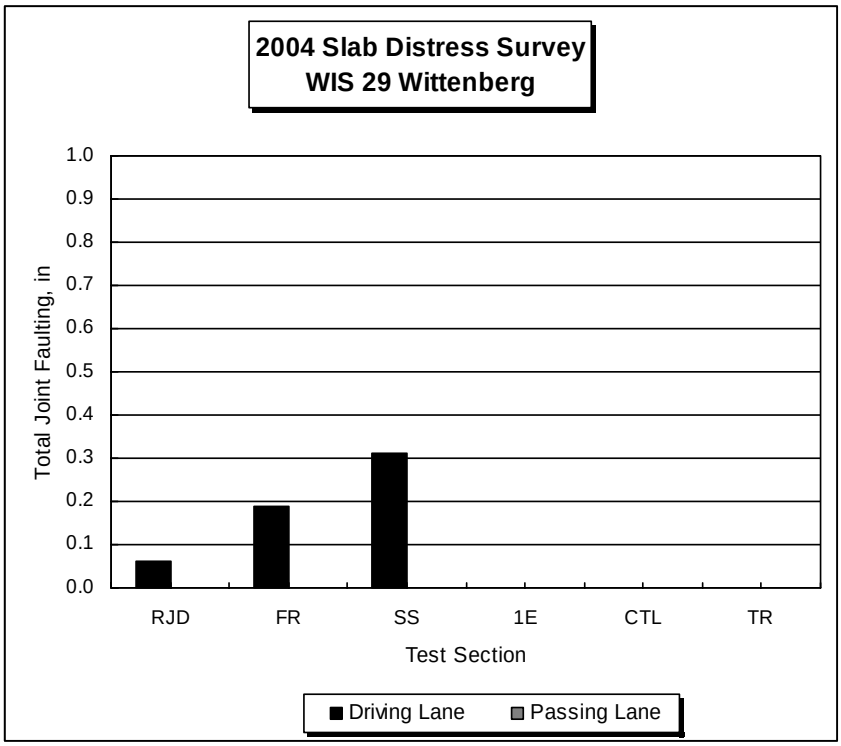


Figure 4.4.8 Joint Faulting Data – WIS 29 Wittenberg

4.4.3 WIS 29 Tilleda

Distress measures recorded during the year 2004 survey are provided in Figures 4.4.9 to 4.4.11. As shown there is an increased amount of joint spalling and joint faulting as compared to test sections at WIS 29 Abbotsford and WIS 29 Wittenberg and reduced joint chipping. Transverse contraction joints within these test sections were saw cut with a multiple blasé joint cutter which appears to have produced a more durable joint face.

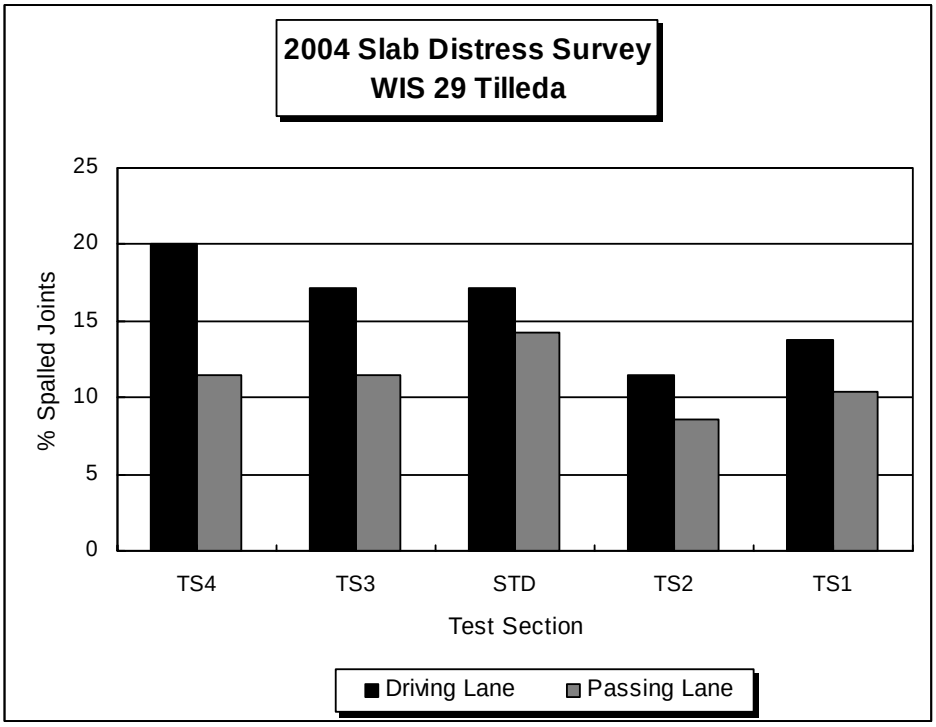


Figure 4.4.9 Joint Spalling Data – WIS 29 Tilleda

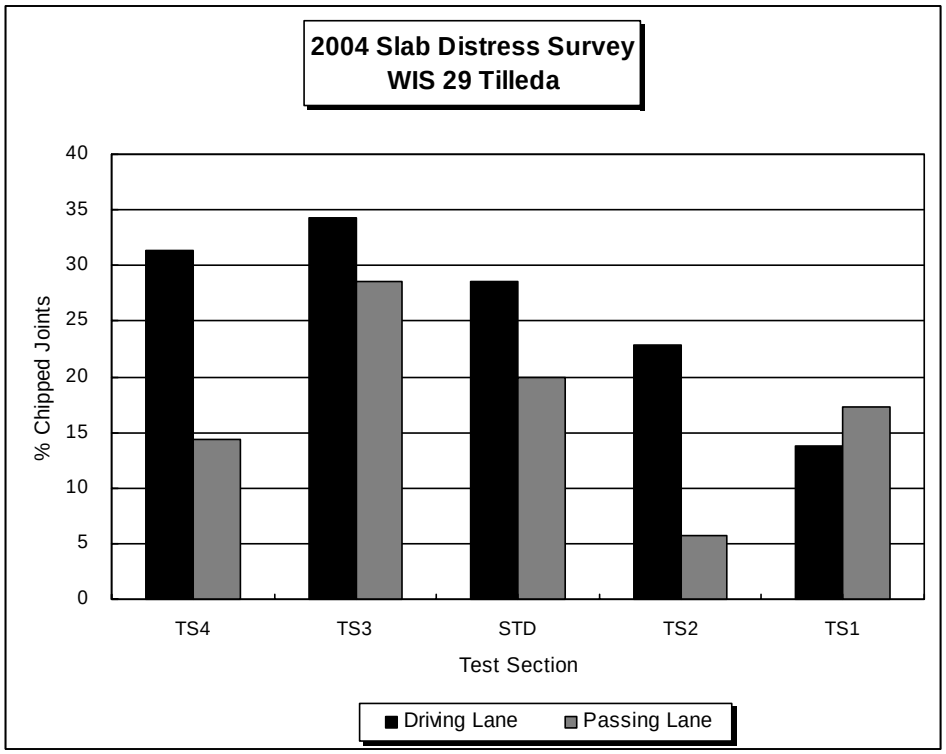


Figure 4.4.10 Chipped Joint Data – WIS 29 Tilleda

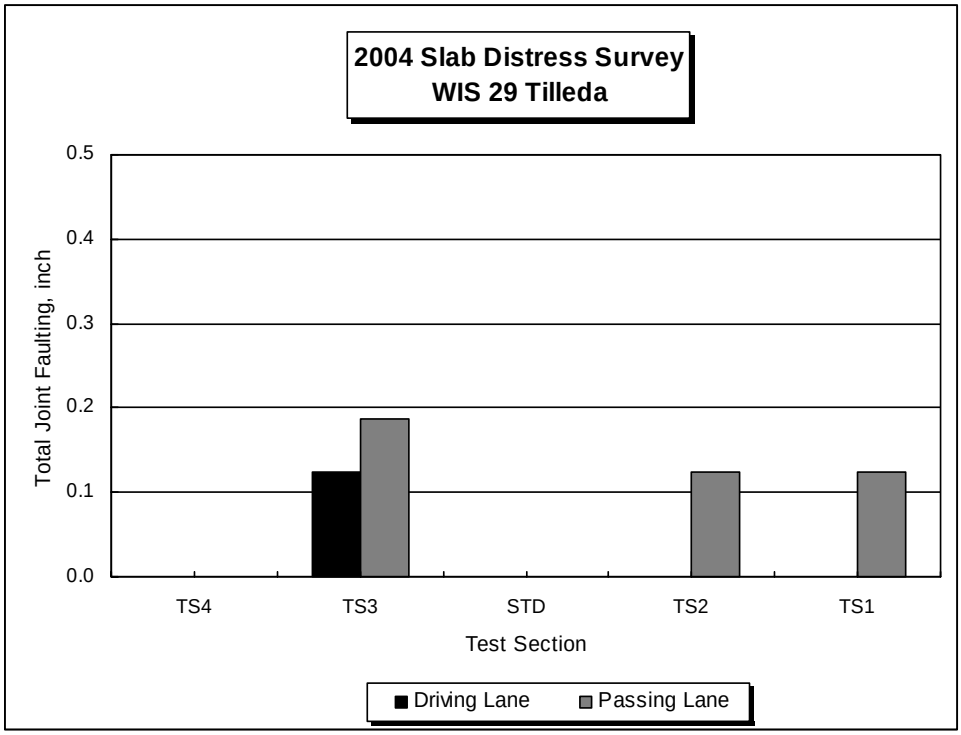


Figure 4.4.11 Joint Faulting Data – WIS 29 Tilleda

CHAPTER 5
CONSTRUCTION COST CONSIDERATIONS

The cost-effectiveness of the various design sections described in this report may be analyzed on the basis of reduced first costs or reduced life-cycle costs. The premise for the design of many of the test sections was to reduce the initial cost of construction without compromising long-term pavement performance. In this respect, any savings in initial construction costs would automatically result in a reduced life-cycle cost and yield a cost-effective concrete pavement design. In contrast, test section designs which would result in an increased construction cost would only be considered as cost-effective if there was a proven increase in the service life of the pavement and/or a reduction in overall maintenance costs.

The performance data collected to date provides some insight into the long-range performance trends for the various test sections, but insufficient clarity in the data makes it difficult to accurately assess the life-cycles costs associated with the various design alternatives. Since ride quality is a factor in long term performance, the collected ride quality data obtained from the three project sites will be used to illustrate the potential cost-effectiveness of each alternate design section.

5.1 WIS 29 Abbotsford

The 2004 IRI values, representing approximately seven years of service life, suggest the following:

- The passing lane is performing better (i.e., lower IRI value) than the driving lane for all constructed sections. This result is as expected due to the increased traffic loadings typically experienced within the driving lane.
- The performance of the FRP sections (RJD, GF, CP) is comparable to the control sections in both the driving and passing lanes, indicating that the ride quality of these sections has not been compromised by the reduction in joint deflection load transfer associated with the FRP bars.

- The passing lane within all alternate placement sections (1E, 2E, 3E, 4E, 3S, 4S) is performing better than the driving lane of the control sections, which suggests that a reduction in the dowel placements within the passing lane may not compromise the long-term performance of the pavement as a whole. In other words, both lanes would be expected to deteriorate at comparable rates.

5.2 WIS 29 – Wittenberg

The 2004 IRI values, representing approximately seven years of service life, suggest the following:

- The passing lane is performing better (i.e., lower IRI value) than the driving lane for all constructed sections except the FRP sections (RJD, FR).
- The performance of the alternate placement section (1E) is comparable to the control sections in both the driving and passing lanes.
- The performance of the alternate placement section (1E) is comparable to the control sections in both the driving and passing lanes.
- The passing lane within the trapezoidal (TR) and alternate placement section (1E) is performing better than the driving lane of the control sections, which suggests that a reduction in slab thickness or in the dowel placements within the passing lane may not compromise the long-term performance of the pavement as a whole.

5.3 WIS 29 – Tilleda

The 2004 IRI values, representing approximately five years of service life, suggest the following:

- The passing lane is performing better (i.e., lower IRI value) than the driving lane for all constructed sections.
- The performance of all sections except Test Section 3 (2-way surface drainage, 1-way base drainage, trapezoidal passing lane) is comparable or better than the control sections in both the driving and passing lanes.
- The one-way surface and one-way base drainage sections (TS1, TS2) are performing better than all other sections.

5.4 Initial Construction Costs

The performance data trends presented in Sections 5.1 to 5.3 suggest potential savings may be realized for the initial construction and/or long-term costs of various test sections. In order to quantify these costs savings, contacts were made with paving contractors, concrete pavement associations, and material suppliers to develop appropriate units costs for the specific paving items which varied amongst the test sections. Table 5.4.1 provides estimates of current unit prices for specific construction items relating to the various test section designs. Table 5.4.2 provides initial construction cost comparisons for each test section, computed on a per-mile basis for the 4-lane divided highway cross section, based on the unit costs provided in Table 5.4.1.

Table 5.4.1 Estimates of Unit Construction Costs

Item	Unit	Unit Price
Epoxy Coated Steel Dowel 1-1/2" x 18"	Loose, Each	$4.00
Polyester FRP Dowel 1-1/2" x 18"	Loose, Each	$6.11
Solid Stainless Steel Dowel 1-1/2" x 18"	Loose, Each	$30.00
Hollow-Filled Stainless Steel Dowel, 1-1/2" x 18"	Loose, Each	$15.00
Paving Grade Concrete	Cubic Yard	$50.00
Longitudinal Drainage System	Foot	$6.50
Aggregate Base, Open Graded No. 2	Cubic Yard	$17.19

Table 5.4.2 Comparative Initial Costs of Test Sections

WIS 29 – Abbotsford	
Test Section	**Initial Construction Cost Comparison $/mile – 4-lane Divided Pavement**
C1, C2	$0
RJD, GF, CP	+ $31,325
HF	+ $163,306
4S	+ $163,306
3S	+ $180,436
4E	- $29,692
3E	- $27,408
2E	- $29,692
1E	- $31,976
WIS 29 – Wittenberg	
Test Section	**Initial Construction Cost Comparison $/mile – 4-lane Divided Pavement**
C1, C2, C3	$0
RJD, FR	+ $32,203
SS	+ $396,812
1E	- $32,872
TR	- $63,600
WIS 29 – Tilleda	
Test Section	**Initial Construction Cost Comparison $/mile – 4-lane Divided Pavement**
STD	$ 0
TS1	- $68,640
TS2	- $60,302
TS3	+ $2,207
TS4	+ $31,138

5.4.1 Alternate Dowel Placements

The performance data indicates that a reduction in dowels bars within the passing lane may be a viable alternative to reduce initial construction costs without compromising the overall pavement performance. The current design procedures for jointed concrete pavements, as detailed in Facilities Development Manual Procedure 14-10-10, requires a transverse contraction joint spacing of 18 feet for pavement thicknesses of 10 inches or greater. Using this benchmark, a total of 3,520 dowels per lane-mile would be required for construction of a standard passing lane with 12 dowels per joint (12 inch c-c spacing). Reducing the passing lane dowel placements to 3 dowels per wheel path (6 dowels per joint), represents a potential cost savings of $14,080 per 4-lane mile using epoxy coated steel dowels.

5.4.2 Trapezoidal Cross Sections

The performance data indicates that the use of a trapezoidal slab may be a viable design alternate to reduce initial construction costs. Based on the full-width trapezoidal slab used for the WIS 29 – Wittenberg section, a potential savings of 636 cubic yards of concrete per 2-lane mile (26-ft width) may be realized, potentially reducing initial construction costs by $63,600 per 4-lane mile. For the trapezoidal sections used for WIS 29 – Tilleda (TS2, TS3, TS4), the increased slab width used for the passing lane results in an *increase* in concrete materials when compared to a typical design cross section (26-ft width). Based on the design slab thickness of 10-inches, additional concrete material requirements per constructed 2-lane mile are 16 cubic yards (TS2) and 244 cubic yards (TS3, TS4), representing *increases* in initial construction costs of $1,600 (TS2) and $24,440 (TS3, TS4) per 4-lane mile.

5.4.3 Alternative Drainage Designs

The alternative drainage designs used for the better performing WIS 29 – Tilleda sections (TS1, TS2, TS4) represent changes to the drainage layer and/or the longitudinal collection system. As compared to the standard design section (4-inch drainage layer, 2-way drainage, 26-ft width), TS2 and TS4 (4-inch drainage layer, 29-ft width) require an

additional 196 cubic yards of open graded aggregate base per mile, representing an *increase* in initial construction costs of $3,369 per mile for both sections. The open graded aggregate material requirements for TS1 (4-inch drainage layer, 26-ft width) are identical to a standard section. For the sections with one-way base drainage (TS1, TS2), the elimination of the median edge longitudinal drainage system represents a potential cost savings of $34,320 per one-way mile. Combining these two cost factors, the initial construction cost variations for each of the better performing WIS 29 – Tilleda sections are:

TS1 - $68,640 savings per 4-lane mile
TS2 - $60,302 savings per 4-lane mile
TS4 - $31,138 ***increase*** per 4-lane mile

5.4.4 Alternative Dowel Materials

The alternative dowel bar materials used for the WIS 29 – Abbotsford and WIS 29 – Wittenberg test sections, including FRP, Solid Stainless Steel, and Hollow-Filled Stainless Steel, are all more costly at present than the standard epoxy coated steel dowels. The performance data collected to date does not indicate that test sections constructed with these alternative dowels are performing substantially better than conventional epoxy coated steel dowels and thus these alternate dowel materials may not be cost-effective. Longer term performance data will be required to better define performance enhancements that may be associated with these alternative dowel materials.

CHAPTER 6
SUMMARY & RECOMMENDATIONS

This report presents details on the design and performance of alternative pavement sections constructed within the State of Wisconsin along portions of WIS 29 in Shawano, Clark and Marathon Counties. These pavement test sections were designed and constructed in order to investigate the feasibility of incorporating design changes which would lower the initial construction costs without compromising pavement performance. These designs, if suitable, may increase the cost effectiveness of concrete pavements and offer pavement designers with alternatives to the standard designs currently used by WisDOT.

6.1 Summary of Study Findings

Pavement test sections incorporating variable dowel positioning and alternative dowel materials were constructed in 1997 along WIS 29, west of Abbotsford. These sections were constructed to investigate the impacts of reduced doweling within the driving lanes and/or alternate dowel materials with enhanced corrosion resistance, including fiber reinforced polymer (FRP) composite and stainless steel dowels. The following summarizes the performance data collected to date within these sections:

- Through the first seven years of service, no significant distress, including joint spalling, joint faulting or slab cracking, is noted within the FRP composite sections. The ride quality of the FRP composite sections, in terms of computed IRI values, is poorer than comparable sections with standard epoxy coated steel dowels.

- The reduced stiffness of the FRP composite dowels, as compared to standard epoxy coated steel dowels, results in markedly lower transverse joint deflection load transfer during periods of cooler weather when joint openings are increased and aggregate interlock is minimal. This reduced deflection load transfer will typically result in higher stresses and deflections within the loaded slab and may lead to a

shortened service life in terms of longitudinal and/or corner cracking and slab faulting.

- Deflection load transfer provided by the solid stainless steel dowels is slightly to markedly lower than conventional epoxy coated steel dowels for WIS 29 Abbotsford placement alternates 3 and 4. For the standard placement on WIS 29 Wittenberg, joint deflection load transfer values are comparable to the epoxy coated steel bars. The hollow-core, mortar-filled stainless steel dowels appear to be providing deflection load transfer roughly equivalent to standard epoxy coated steel dowels.

- Section-average transverse joint load transfer values generally decreases with a reduced number of wheel path dowels. However, sections with four dowels in the outer wheel path are providing deflection load transfer values comparable to the standard full-width dowel placement. Sections with three dowels in the outer wheel path generally provide reduced load transfer compared to other placements. Reduced deflection load transfer within the outer wheel path will typically result in higher stresses and deflections within the loaded slab and may lead to a shortened service life in terms of longitudinal and/or corner cracking and slab faulting.

- Test sections with alternate dowel placements are performing relatively well with limited low severity joint spalling noted within the driving lane in three of the seven sections. Test sections with 3 and 4 epoxy coated steel dowels in the outer wheel path, but no dowel at or near the edge, have 10.3% (1E) and 13.8% (2E) spalled joints, respectively. The remaining test section, with three stainless steel wheel path dowels and 1 dowel at the edge (4SS) has 3.4% spalled joints.

Test sections incorporating alternate dowel materials, placement locations, and slab geometry were constructed within the eastbound and westbound lanes of WIS 29 Wittenberg. The following summarizes the performance data collected to date within these sections:

- Test sections constructed with FRP composite dowels are showing slightly to substantially lower deflection load transfer as compared to standard epoxy coated steel dowels. However, the relative reduction in load transfer capacity varies depending on the period of deflection testing. After 7 years of service, there is minimal joint faulting and spalling within the driving lane of the FRP test sections. Recent profiling data from these sections indicate slightly higher IRI values as compared to control sections values.

- The test section constructed with solid stainless steel dowels is providing deflection load transfer capacity essentially equal to the control sections with epoxy coated steel dowels. After 7 years of service, joint faulting was evident across only 1 joint (0.3 inches) which was coincident with a high early strength slab placement at a driveway location. The profile readings within this section also indicate higher IRI values as compared to the control sections, likely a result of the joint faulting.

- The test section incorporating a reduced number of dowels (three per wheel path) is performing essentially on par with the control section, in terms of IRI, load transfer and joint distress.

- After 7 years of service, the test section incorporating the trapezoidal slab is performing equal to or better than the control section in terms of IRI and joint faulting. Low severity transverse joint spalls are noted at 3 joints within the driving lane but this distress has not affected ride quality measures.

Test sections incorporating alternate slab geometry and drainage designs were constructed within the westbound lanes of WIS 29 Tilleda. The following summarizes the performance data collected to date within these sections:

- After 5 years of service all test sections are exhibiting essentially equal joint load transfer capacities.

- Test sections incorporating one-way surface and one-way base drainage (TS1, TS2) are providing better ride quality (lower IRI) than the standard pavement section in both travel lanes. After 5 years of service, low severity joint faulting (0.06 in) was noted within the driving lane of all sections incorporating one-way base drainage (two to three joints per section).

- Joint spalling is more prevalent within these sections than at other test sites, with driving lane low severity joint spalling ranging from 11% – 20% of the joints and passing lane values ranging from 8% – 14%. Spalling is generally less severe in sections with one-way base drainage.

The early-age performance data collected to date indicates that the FRP composite dowels may not be appropriate for use as direct replacements for standard epoxy coated steel dowels due to their reduced load transfer capacity. After 7 years of service, sections with standard placements of FRP composite dowels show consistently higher IRI values than comparable sections with traditional epoxy coated steel dowels. To enhance the load transfer capacity of joints with FRP dowel, more dowels (i.e., closer spacings) may be required; however, this type of placement strategy was not part of this investigation. Early-age performance data collected to date indicates that the passing lane fitted with FRP composite dowels is performing equal to or poorer than the more heavily trafficked driving lane fitted with epoxy coated steel dowels.

Comparative initial construction cost estimates for the FRP test sections were developed based on transverse joint spacing used on each project. All cost comparisons were referenced to the standard placement of epoxy coated steel dowels (26 dowels per joint). The increased initial construction costs, per 4-lane mile, for standard placements of FRP composite dowels are estimated at $31,325 for WIS 29 Abbotsford (18.5-ft average joint spacing) and $32,203 for WIS 29 Wittenberg (18-ft joint spacing). Short-term ride

quality data collected to date indicates that these may not be cost-effective design alternates. Longer-term data is needed to better define the cost-effectiveness of these alternate dowel materials.

The collected performance data from WIS 29 Abbotsford also suggests that pavement sections with a reduced number of epoxy coated steel dowels are providing better service than stainless steel alternates. The increased initial construction costs for alternate placements of solid stainless steel dowels are estimated at $163,306 and $180,436, respectively, per 4-lane mile (compared to standard placements of expoxy coated steel bars). Short-term ride quality data collected to date indicates that these may not be cost-effective design alternates. In contrast, performance data from WIS 29 Wittenberg indicates the section with standard placements of solid stainless steel dowels is performing as well as the section with traditional epoxy coated steel dowels. The variation in comparative performance between project locations may also be related to the dowel placement techniques used, namely the automatic dowel bar inserter (DBI) at WIS 29 Abbotsford and standard dowel baskets at WIS 29 Wittenberg. The increased initial construction cost for standard placements of solid stainless steel dowels is estimated at $396,812 per 4-lane mile. Longer-term performance data is needed to better define the cost-effectiveness of this alternate dowel material.

The early age performance data from WIS 29 Abbotsford also suggests that reduced dowel placements in the heavily trafficked driving lane will result in a compromised service life. However, for the majority of test sections the performance of the passing lane with reduced dowel placements is equal to or better than the performance of the driving lane. Reducing the number of dowels in the passing lane only is estimated to reduce initial constructions cost by $14,080 per 4-lane mile and would yield a pavement with equivalent lane performance. In this respect, the cost savings associated with reduced dowel placements in the passing lane may be justifiable.

The trapezoidal slab design used on WIS 29 Wittenberg yields an estimated savings of $63,600 per 4-lane mile. Short-term ride quality data collected to date indicates that this design alternate is performing better than all other sections at this location and thus may be a viable, cost-effective design alternate.

The early age performance data from the WIS 29 Tilleda sections constructed with variable slab geometry and drainage designs indicate that one-way surface and one-way base drainage designs (TS1, TS2) are performing as well or better than standard crowned pavements with two-way base drainage. The initial construction cost savings associated with these sections range from $60,302 (TS2) to $68,640 (TS1) per 4-lane mile, indicating both may be viable, cost-effective design alternates. The drainage capacity of the base layer within these sections, constructed with open graded number 1 stone, appears sufficient to handle all infiltrated water. It is unclear if a similarly designed drainage using open graded number 2 stone, which is the current WisDOT standard, would provide comparable performance. It is also unclear if long tangent sections constructed with one-way surface drainage would result in a safety problem due to vehicle drift.

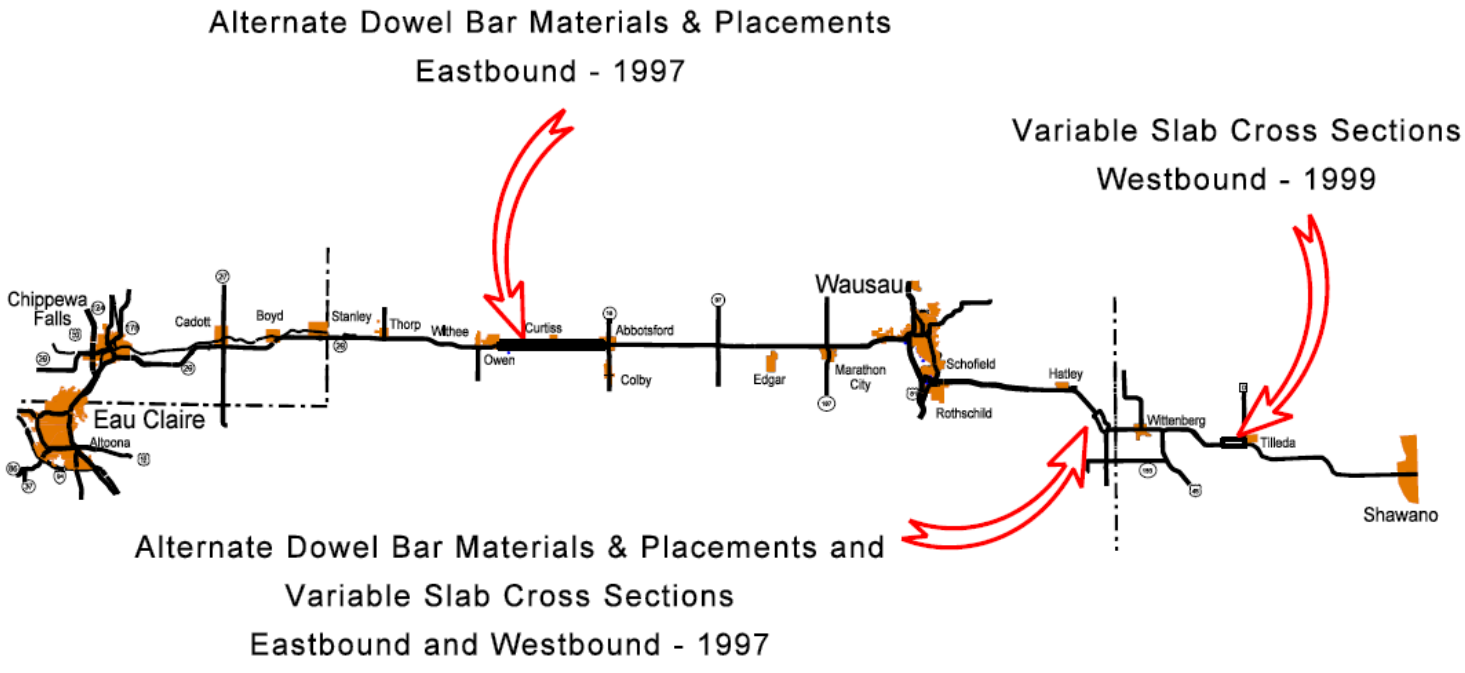
6.2 Recommendations for Further Study

This report presents study results for pavement test sections in service for 5 – 7 years. In the life of well-designed concrete pavement systems, this can be considered as very early-age performance. While estimations of long-term performance may be generated from early-age indicators, sufficient clarity regarding the cost effectiveness of alternate design strategies generally requires an extended period of observation. To this end, it is recommended that all tests sections be continually monitored on a 2-3 year cycle to document their performance. Ride quality and distress information should be gathered on a routine basis following WisDOT protocol for the state-wide pavement survey. Deflection data should be collected at 3-5 year intervals to document the long-term structural performance of all sections.

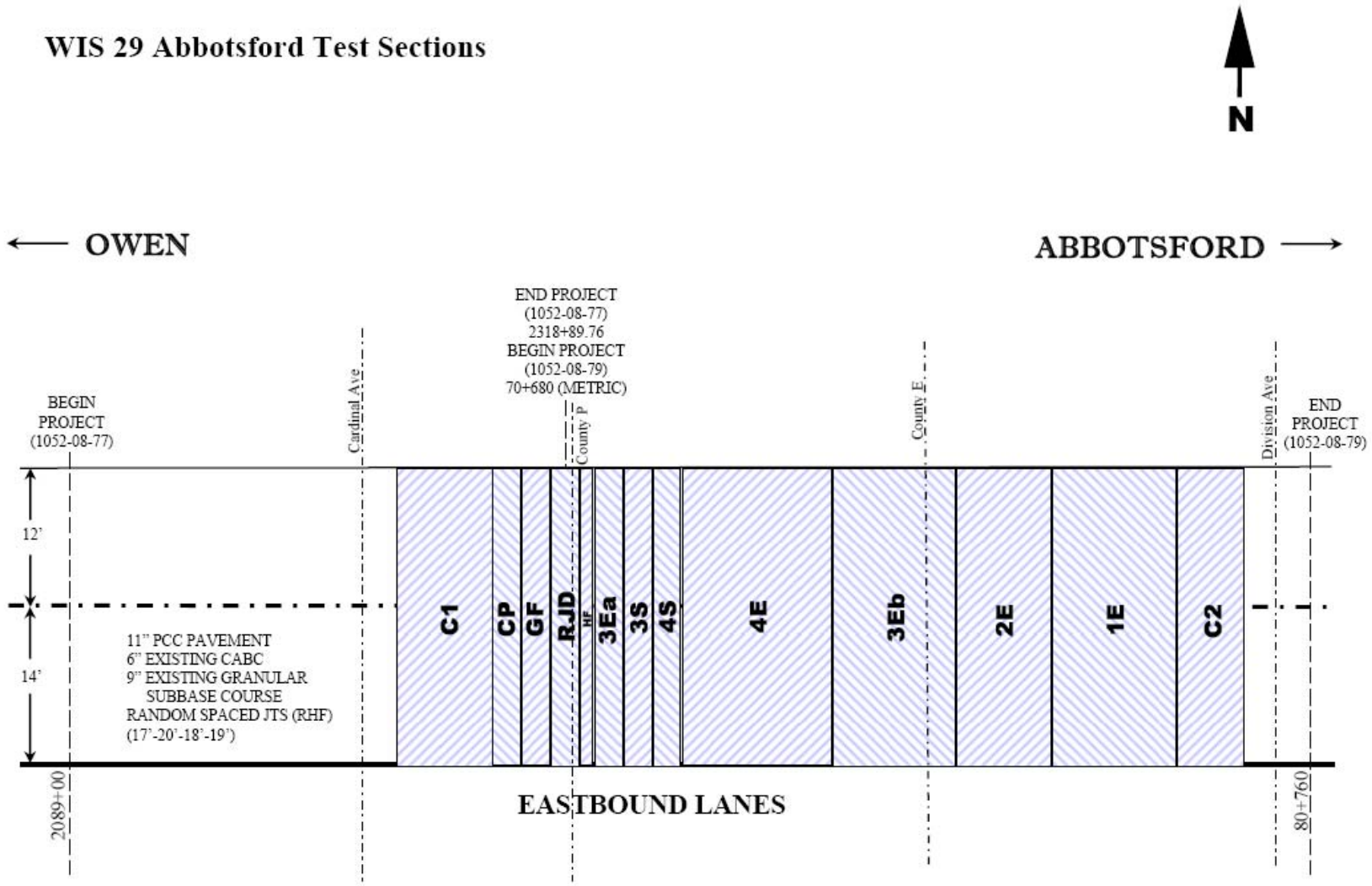
APPENDIX A

Test Section Location Maps

Cost Effective Concrete Pavement Cross Sections
WIS 29 - Research Test Sections

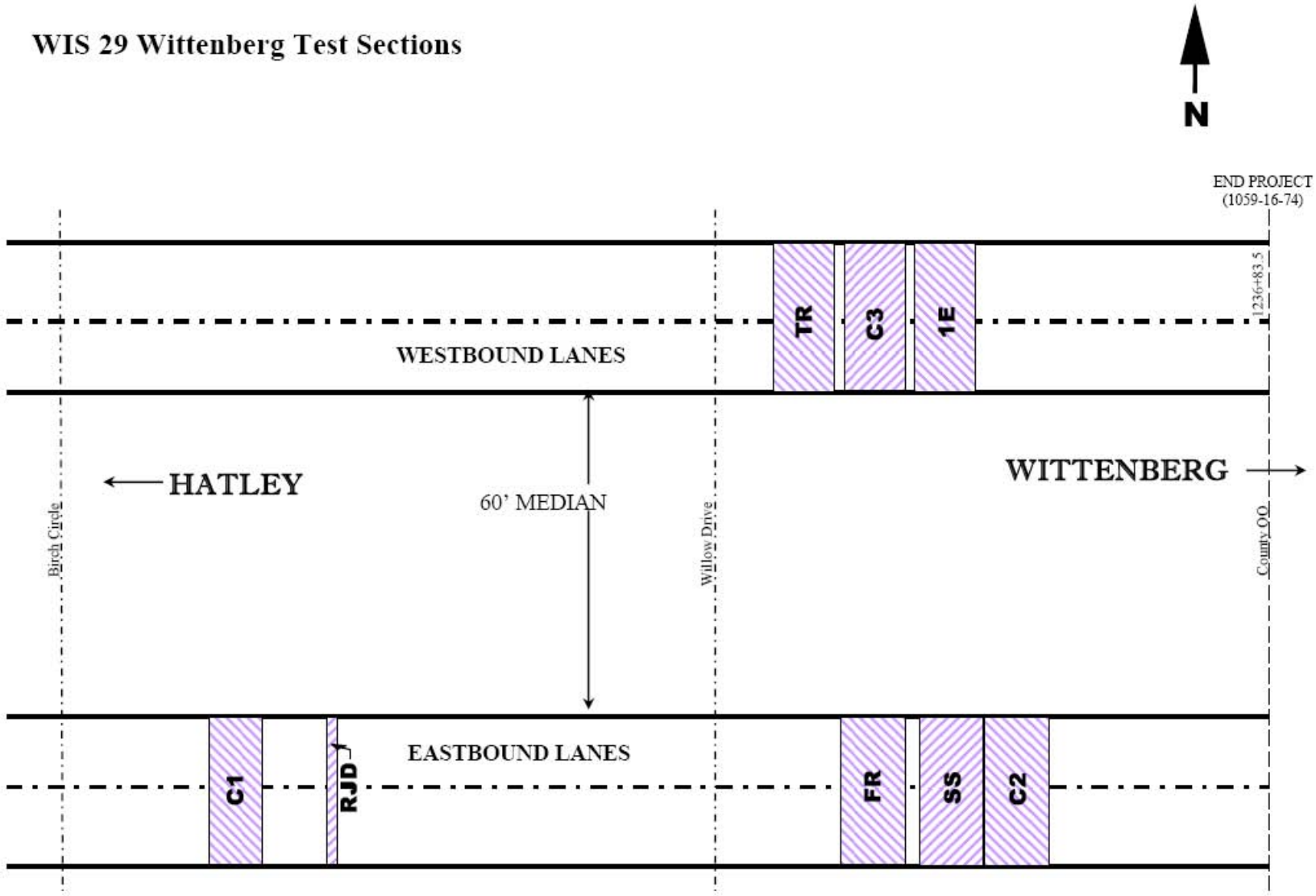


WIS 29 Abbotsford Test Sections



A-2

WIS 29 Wittenberg Test Sections



WIS 29 Tilleda Test Sections