REPORT NUMBER: WI/SPR-12-99

PB99-176869

COST EFFECTIVE CONCRETE PAVEMENT CROSS-SECTIONS

INTERIM REPORT

WISCONSIN TECHNOLOGY ADVANCEMENT DEPARTMENT OF TRANSPORTATION

AUGUST 1999

1. Report No. WI/SPR-12-19	2. Government Accession No.	3. Recipients Catalog No.
4. Title and Subtitle Cost Effective Concrete Pavement Cross-Sections		5. Report Date August 15, 1999
		6. Performing Organization Code
7. Author(s) James A. Crovetti		8. Performing Organization Report No. Report #
9. Performing Organization Name and Address Marquette University Center for Highway and Traffic Engineering Department of Civil & Environmental Engineering P.O. Box 1881 Milwaukee, WI 53201-1881		10. Work Units No. (TRAIS)
		11. Contract or Grant No. WI/SPR 95-03
12. Sponsoring Agency Name and Address Wisconsin DOT, Pavements and Research Section 3502 Kinsman Boulevard Madison, WI 53704-2507		13. Type of Report and Period Covered Interim - 9/97 to 8/99
		14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

The present pavement selection policy of the Wisconsin Department of Transportation (WisDOT) 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 outer 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 receive significantly lower Equivalent Single Axle Load (ESAL) applications over the service life of the pavement.

Analysis of PCC pavement design alternatives, including variable slab thickness within and/or across traffic lanes, variable load transfer designs, and alternative base layer drainage designs, were completed. Based on the results of these analyses, four alternative dowel patterns were developed to reduce the number of dowel bars installed across transverse pavement joints. These patterns were developed to be consistent with dowel bar installation equipment currently used within the State of Wisconsin while still providing necessary load transfer mechanisms in the wheelpath areas of both travel lanes. In additional to dowel placement alternates, test sections were constructed using alternative dowel materials, including fiber reinforced polymer (FRP) composite dowels, solid stainless steel dowels, and hollow core - mortar filled stainless steel dowels.

This report presents details relating to the design, construction, and first year performance of concrete pavement test sections constructed in the State of Wisconsin along STH 29 in Clark and Marathon Counties. These test sections were constructed during the Summer of 1997 to validate the constructability and cost-effectiveness of alternative concrete pavement designs incorporating variable dowel strategies and slab thicknesses.

17. Key Words PCC Pavements, Dowel Bars, Stainless Steel Dowels, FRP Composite Dowels, Variable Thickness PCC Slabs	18. Distribution Statement		
19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price

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INTERIM REPORT

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August, 1999

For

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1.0 INTRODUCTION

This report presents details relating to the design, construction, and first year performance of concrete pavement test sections constructed in the State of Wisconsin along STH 29 in Clark and Marathon Counties. These test sections were constructed during the Summer of 1997 to validate the constructability and cost-effectiveness of alternative concrete pavement designs incorporating variable dowel strategies and slab thicknesses. This research is jointly sponsored by the Wisconsin Department of Transportation (WisDOT) and the Federal Highway Administration under the FY 96 Priority Technologies Program (PTP) and the High Performance Rigid Pavements (HPRP) project.

Test sections incorporating alternate dowel patterns and materials were constructed within the Eastbound lanes of STH 29 in Clark County between Owen and Abbotsford, herein referred to as STH 29 West. All dowel bars in the STH 29 West test sections were placed by an automated dowel bar inserter (DBI) mounted to the concrete paver. Test sections incorporating alternative dowel placements and materials as well as variable slab thicknesses were constructed within the Eastbound and Westbound lanes of STH 29 in Marathon County between Hatley and Wittenberg, herein referred to as STH 29 East. Dowel bars in the STH 29 East test sections were placed with conventional dowel bar baskets. Furthermore, all test sections constructed on STH 29 East are designated as Strategic Highway Research Program (SHRP) test sections, as shown in Table 1.1.

1.1 Project Background

The present pavement selection policy of the Wisconsin Department of Transportation (WisDOT) 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 outer 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 receive significantly lower Equivalent Single Axle Load (ESAL) applications over the service life of the pavement.

Pavement design analyses were completed during earlier research, including variable slab thickness within and/or across traffic lanes, variable load transfer designs, and alternative base layer drainage designs. Based on the results of these analyses, four alternative dowel patterns were developed to reduce the number of dowel bars installed across transverse pavement joints. These patterns were developed to be consistent with dowel bar installation equipment currently used within the State of

Wisconsin while still providing necessary load transfer mechanisms in the wheelpath areas of both travel lanes.

The four alternate dowel patterns utilized in the construction of the STH 29 test sections are illustrated in Figure 1.1. Test sections including all four placement alternates were constructed along the Eastbound lanes of STH 29 West. A test section incorporating placement alternate 1 was also constructed along the Westbound lanes of STH 29 East.

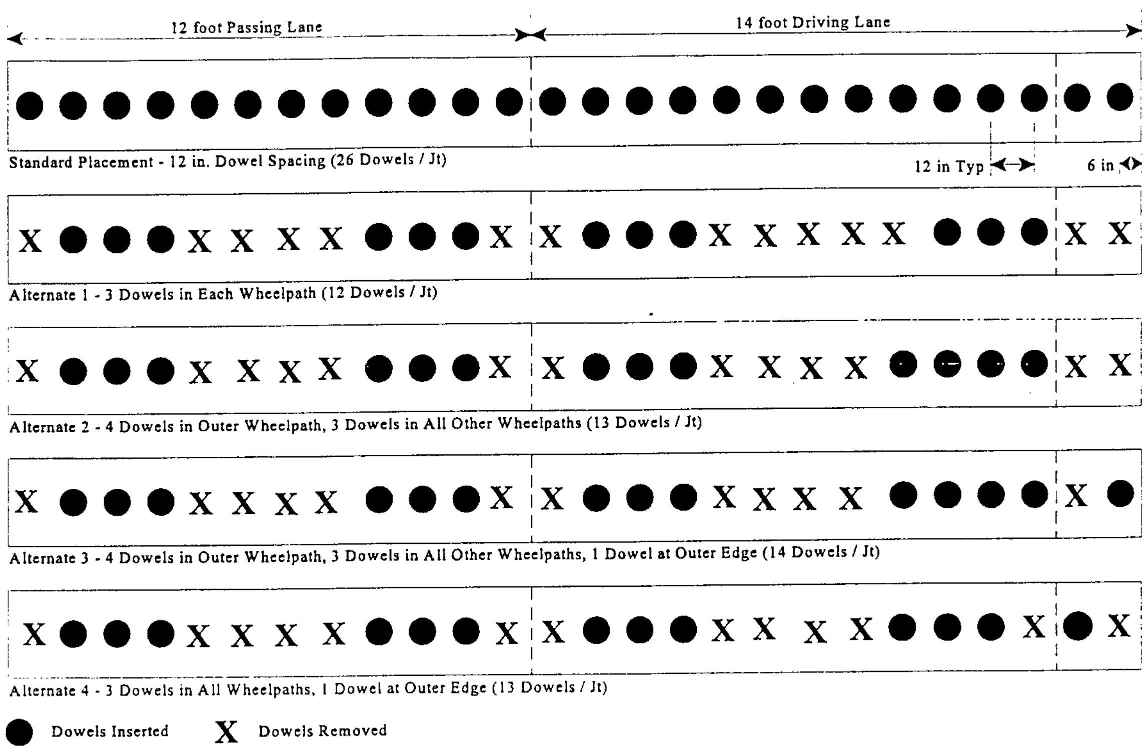

Figure 1: Dowel Bar Placement Alternatives for STH 29 West

In additional to dowel placement alternates, test sections were constructed using alternative dowel materials, including fiber reinforced polymer (FRP) composite dowels, solid stainless steel dowels, and hollow core - mortar filled stainless steel dowels. All alternate dowel materials were utilized within test sections constructed along the Eastbound lanes of STH 29 West. Test sections constructed along the Eastbound lanes of STH 29 East incorporated only FRP composite and solid stainless steel bars.

Two separate control sections were established on STH 29 West and one control section was established on STH 29 East. Descriptions of all test sections, including test section codes utilized in this report as well as SHRP test section designations, where applicable, are provided in Tables 1.1 and 1.2.

Table 1.1 STH 29 East Test Sections

Description	Report Code	SHRP Code
11" (275 mm) PCC, placement alternate 1 (3 standard epoxy coated dowels in each wheelpath, 12 per joint)	1E	550260
11" (275 mm) PCC, standard dowel placement (26 per joint) using FRP composite bars manufactured by MMFG, Glasforms, and Creative Pultrusions	FR	550264A
11" (275 mm) PCC, standard dowel placement (26 per joint) using FRP composite bars manufactured by RJD	RJD	550264B
11" (275 mm) PCC, standard dowel placement (26 per joint) using solid stainless steel dowels manufactured by Slater Steels	SS	550265
8-11" (200-275 mm) variable thickness PCC, standard dowel placement (26 per joint) using standard epoxy coated dowels	TR	550263
11" (275 mm) PCC, standard dowel placement (26 per joint) using standard epoxy coated dowels (Control)	C1	550259

Table 1.2 STH 29 West Test Sections

Description	Report Code
11" (275 mm) PCC, placement alternate 1 (3 standard epoxy coated dowels in each wheelpath, 12 per joint)	1E
11" (275 mm) PCC, placement alternate 2 (4 standard epoxy coated dowels in outer wheelpath of outside lane, 3 in other wheelpaths, 13 per joint)	2E
11" (275 mm) PCC, placement alternate 3 (4 standard epoxy coated dowels in outer wheelpath of outside lane, 3 in other wheelpaths, one at outer slab edge, 14 per joint)	3E
11" (275 mm) PCC, placement alternate 3 (4 solid stainless steel dowels manufactured by Avesta Sheffield in outer wheelpath of outside lane, 3 in other wheelpaths, one at outer slab edge, 14 per joint)	3S
11" (275 mm) PCC, placement alternate 4 (3 standard epoxy coated dowels in each wheelpath, one near outer edge, 13 per joint)	4E
11" (275 mm) PCC, placement alternate 4 (3 solid stainless steel dowels manufactured by Avesta Sheffield in each wheelpath, one near outer edge, 13 per joint)	4S
11" (275 mm) PCC, standard dowel placement (26 per joint) using FRP composite bars manufactured by Creative Pultrusions	CP
11" (275 mm) PCC, standard dowel placement (26 per joint) using FRP composite bars manufactured by Glasforms	GF
11" (275 mm) PCC, standard dowel placement (26 per joint) using FRP composite bars manufactured by RJD	RJD
11" (275 mm) PCC, standard dowel placement (26 per joint) using hollow core-mortar filled stainless steel dowels manufactured by Damascus-Bishop	HF
11" (275 mm) PCC, standard dowel placement (26 per joint) using standard epoxy coated dowels (Control)	C1, C2

2.0 LABORATORY TESTS

2.1 Introduction

Laboratory testing, including joint deflection tests and dowel bar pull-out tests, were conducted at Marquette University with dowels provided by the manufacturers. Lab tests were conducted prior to construction using sample dowels originally proposed for use. Additional tests were conducted on dowels obtained during construction on STH 29 West.

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.** Rectangular test specimens, 12 inches (304 mm) wide x 11 inches (279 mm) deep by 48 inches (1219 mm) long were constructed using paving grade concrete supplied by the Tews Company. Two full-depth joints, each 3/8 inch (9.5 mm) wide, were formed 12 inches (304 mm) from each specimen end using wood inserts. Centered holes on each insert allowed for the placement of 18 inch (457 mm) long dowel bars across each joint. Dowel bars were positioned at the mid-depth of the test specimens. Figure 2.1 provides a schematic illustration of the fabricated specimens.

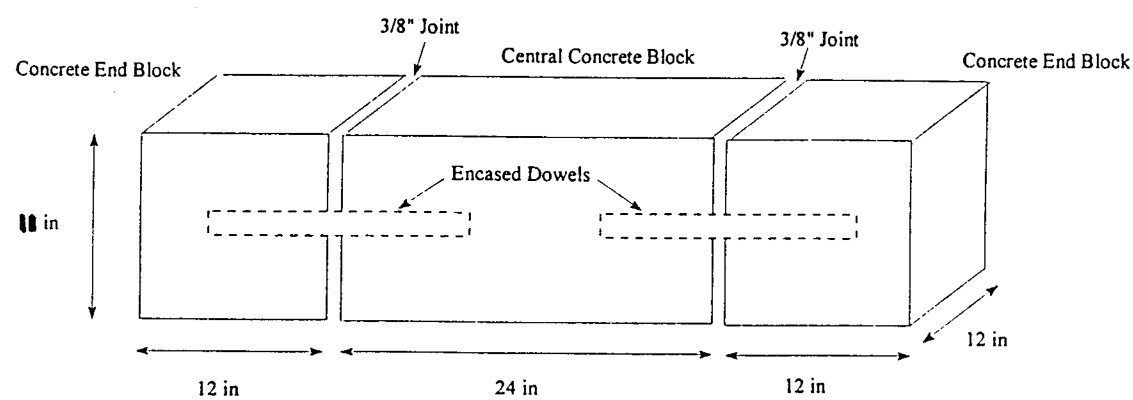

Figure 2.1 Schematic Illustration of Joint Deflection Test Specimens

Test specimens were fabricated with six different types of dowel bars: standard epoxy coated steel dowels (control), polished solid stainless steel dowels, brushed solid stainless steel dowels, and three types of composite dowels as manufactured by MMFG, Creative Pultrusions, and Glasforms. The wooden insert used for fabrication of the test specimen with brushed solid stainless steel dowels failed and resulted in a test specimen which could not be used.

Cast specimens were cured for 21 days prior to the start of testing. The specimen ends were then placed on neopreme capped steel support pedestals and clamped to restrict rotation during loading. The formed joints were positioned about ½ inch (12.7 mm) inwards from the edge of the support pedestals to allow for the placement of Linear Variable Differential Transformers (LVDTs) on the underside of each end to monitor displacement during loading. LVDTs were also positioned across each joint on the underside of the central 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 set at a 2 Hz sampling rate. The load was increased at a rate of 2000 lb/min (8.9 kN/min) until a maximum of 5000 lb (22.2 kN) was obtained. Figure 2.2 illustrates the test set-up during loading.

The maximum relative joint deflections, recorded at a load of 4000 lb (17.8kN) , are provided in Table 2.1. Figures 2.3 through 2.7 illustrate plots of relative deflection vs load for complete test series.

T 253 test protocol allows for a maximum relative joint deflection of 0.010 inch (0.254 mm) at a test load of 4,000 lb (17.8 kN). As shown in Table 2.1 and Figures 2.3 through 2.7, all test results with the exception of one of the joints of the Glasforms' specimen meet this criteria. Furthermore, the composite dowels generally had higher relative joint deflections as compared to the epoxy coated and solid stainless steel dowels.

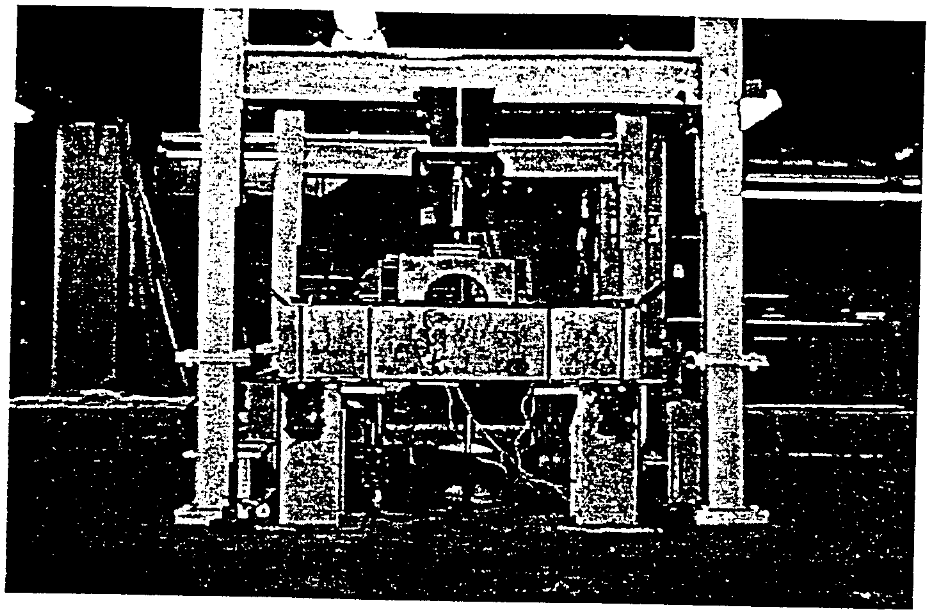

Figure 2.2 Joint Deflection Test Set-up

Table 2.1: Summary of Joint-Deflection Test Results

Dowel Type	Dowel Diameter in (mm)	Relative Joint Deflection, in (mm)		
		Joint 1	Joint 2	Average
Epoxy Coated	1.52 (38.6)	0.0056 (0.1422)	0.0085 (0.2159)	0.00705 (0.17907)
Stainless Steel	1.50 (38.1)	0.0061 (0.1549)	0.0059 (0.1499)	0.00060 (0.01524)
Glasforms	1.50 (38.1)	0.0070 (0.1778)	0.0162 (0.4115)	0.01160 (0.29464)
Creative Pultrusions	1.50 (38.1)	0.0088 (0.2235)	0.0098 (0.2489)	0.00930 (0.23622)
MMFG	1.49 (37.8)	0.0076 (0.1930)	0.0071 (0.1803)	0.00735 (0.18669)

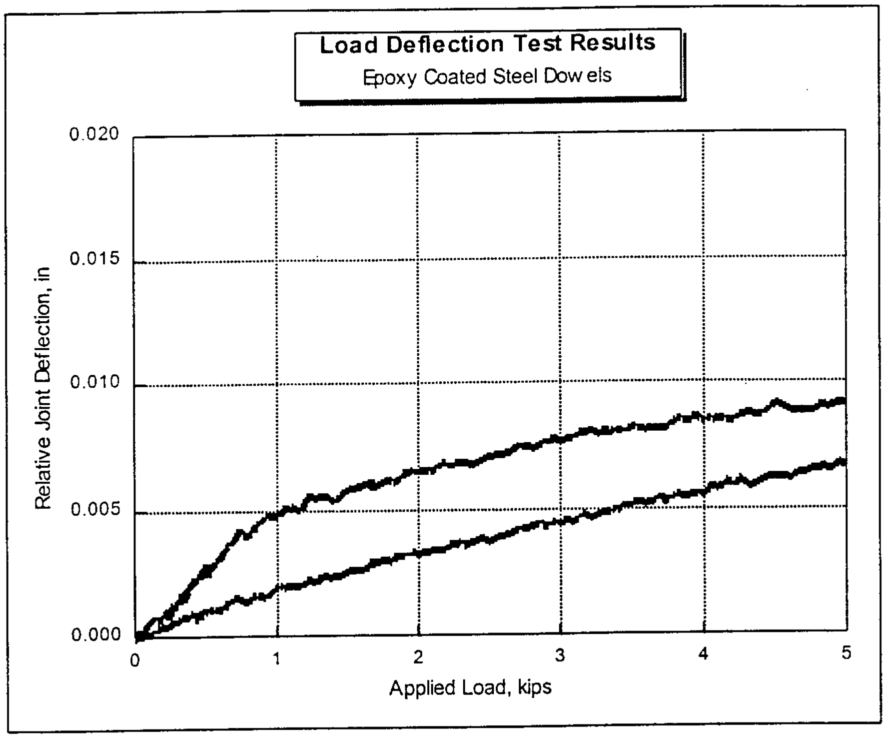

Figure 2.3 Load-Deflection Test Results - Epoxy Coated Dowels

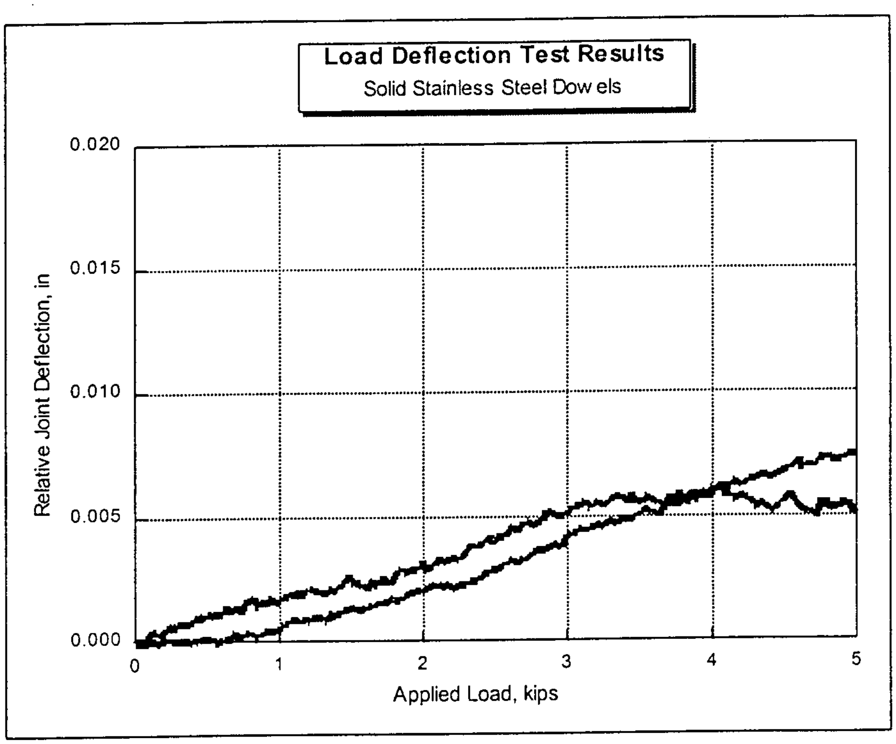

Figure 2.4 Load-Deflection Test Results - Solid Stainless Steel Dowels

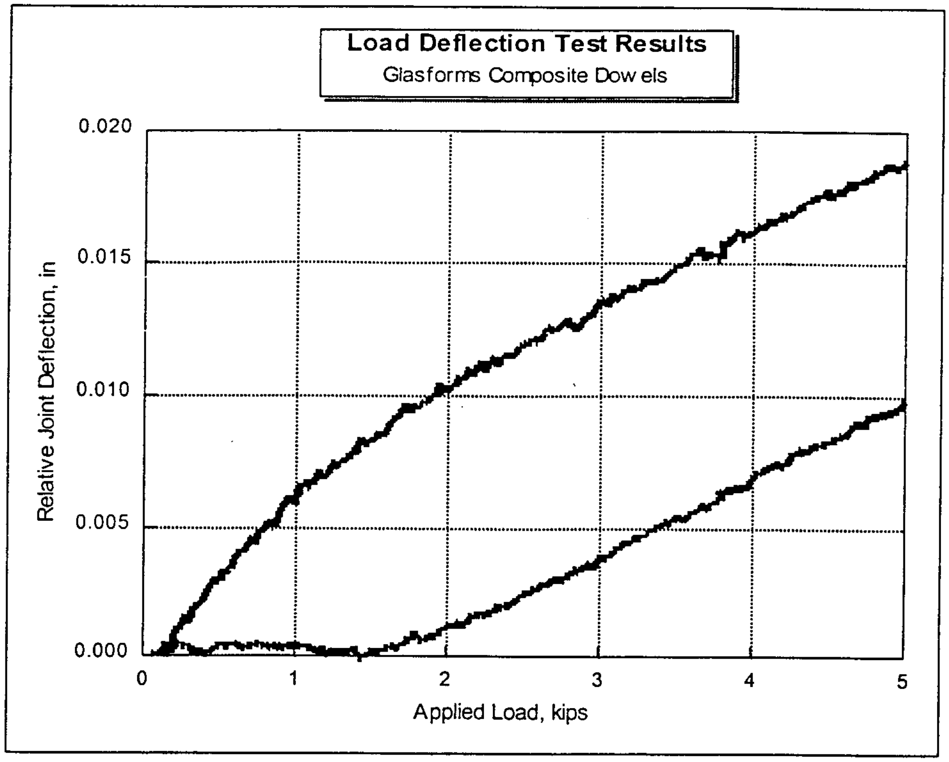

Figure 2.5 Load-Deflection Test Results - Glasforms Dowels

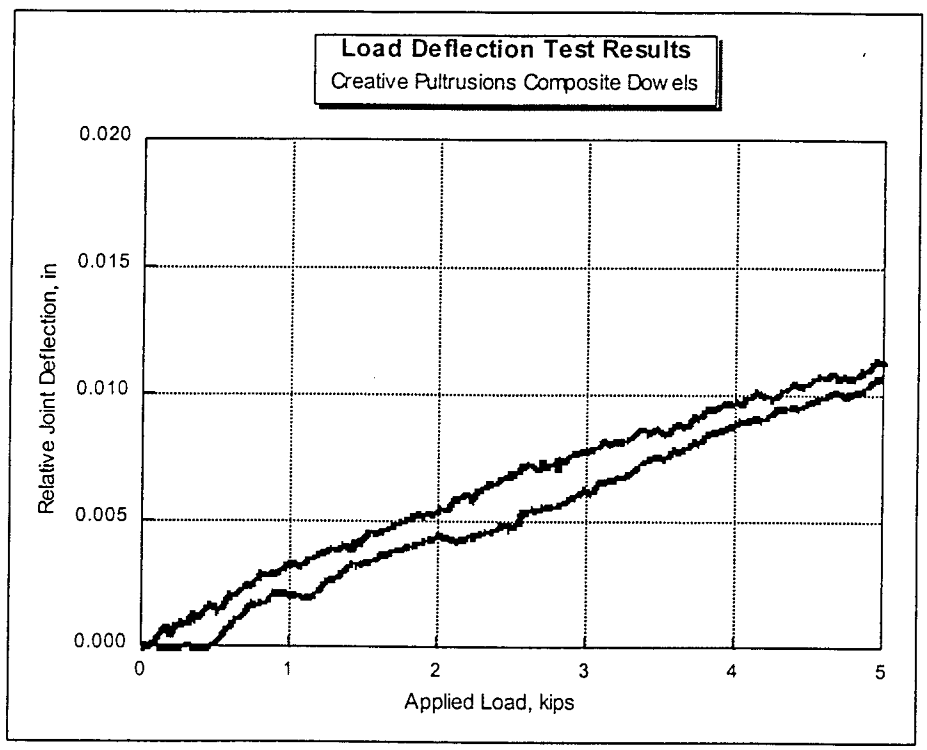

Figure 2.6 Load-Deflection Test Results - Creative Pultrusions Dowels

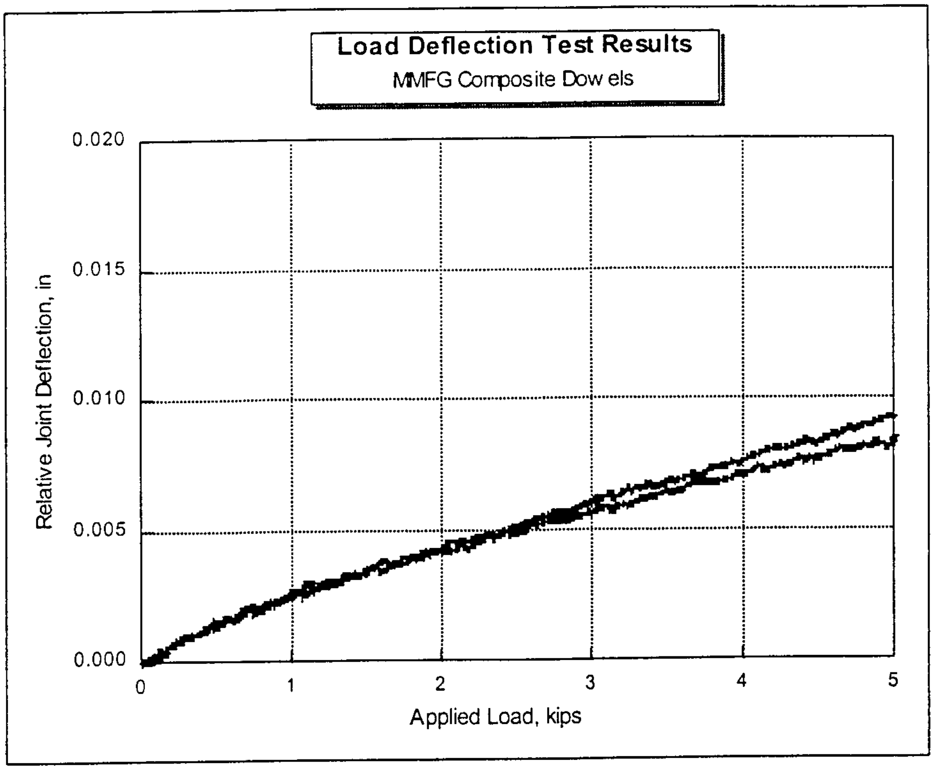

Figure 2.7 Load-Deflection Test Results - MMFG Dowels

2.3 Pull-Out Test - Uncoated 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 x 6 x 12 inch (152 x 152 x 304 mm) were cast in wooden forms using paving grade concrete supplied by the Tews Company. Dowel bars were positioned at the center of the 6 x 6 inch (152 x 152 mm) face, extending approximately 9 inches (228 mm) into the concrete beam. Figure 2.8 provides a schematic illustration of the fabricated specimens.

Pull-out tests were conducted prior to construction with uncoated dowels supplied by the manufacturers, including: standard epoxy coated steel bar (control), a solid polished stainless steel bar manufactured by Slater Steels, a brushed stainless steel bar manufactured by Slater Steels, and three composite dowels 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.

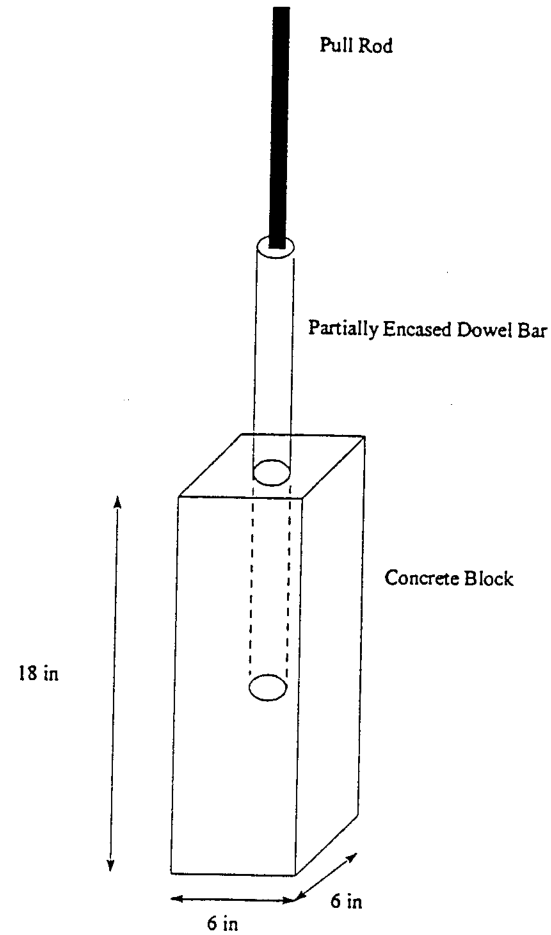

Figure 2.8 Schematic Illustration of Pull-out Test Specimens

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 2.9 illustrates the pull-out test configuration.

Tests were conducted using a crosshead movement rate of 0.03 in/min (0.76 mm/min). Load readings were recorded for every 0.005 inch (0.127 mm) of relative dowel displacements, to a total relative displacement of 0.05 inches (1.27 mm). Additional reading were taken for every 0.05 inch (1.27 mm) of relative displacement to a total relative displacement of 0.5 inches (12.7 mm).

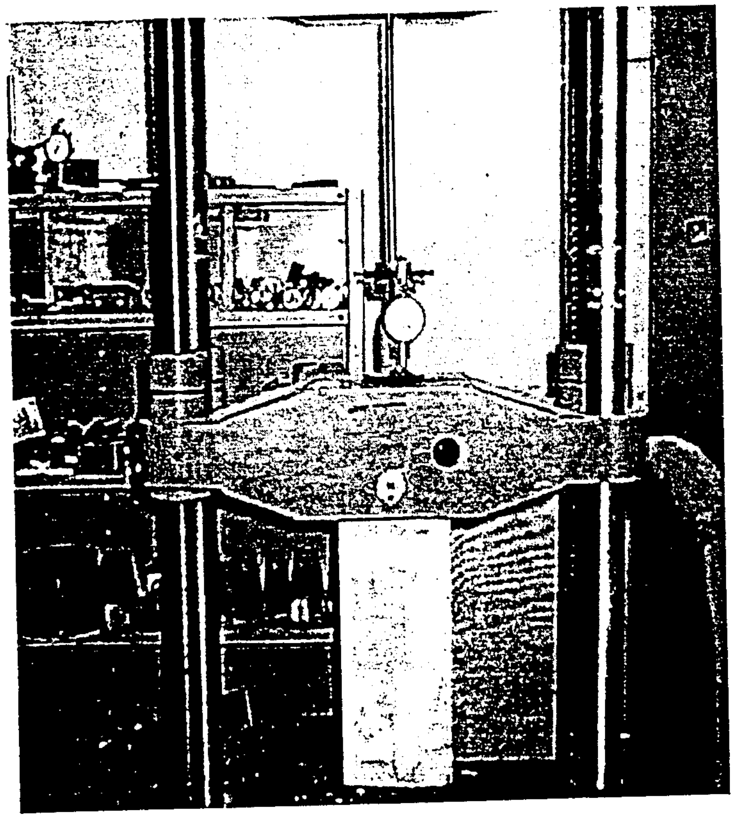

Figure 2.9 Pull-outTest Configuration

The maximum pull-out loads and calculated maximum pull-out stresses are provided in Table 2.2. Maximum pull-out stresses were calculated based on maximum pull-out loads and circumferential contact area between the dowel and the concrete at the start of testing. The maximum pull-out load for the sttel dowels (epoxy coated, brushed stainless steel, polished stainless steel) typically occurs during the initial .05 inches (1.27 mm) of relative displacement and then reduces significantly to a residual load level. The roughened surface on the brushed stainless steel dowel resulted in a maximum pull-out load approximately 50% greater than the epoxy coated dowel whereas the maximum pull-out load for the polished stainless steel dowel was approximately 40% lower than the epoxy coated dowel.

The maximum pull-out load for the composite dowels generally occurs within the initial 0.05 inches (1.27 mm) of relative dowel displacement. Unlike the steel dowels, the residual loads thereafter do not reduce significantly from the maximum value; however, the maximum pull-out loads for all uncoated composite dowels tested are significantly reduced as compared to all steel dowels.

Table 2.2: Summary of Pull-Out Tests on Uncoated Dowels

Dowel Bar Type	Maximum Pull-Out Load lb (kN)	Circumferential Contact Area in^2 (m^2)	Maximum Pull-Out Stress psi (kPa)
Epoxy Coated	4000 (17.8)	43.05 (0.0278)	92.9 (640)
Polished Stainless Steel	2420 (10.8)	42.79 (0.0276)	56.6 (390)
Brushed Stainless Steel	5725 (25.5)	42.74 (0.0276)	134.0 (924)
Glasforms	430 (1.9)	43.26 (0.0279)	9.9 (68)
Creative Pultrusions	155 (0.8)	41.70 (0.0269)	3.7 (26)
MMFG	640 (2.8)	40.76 (0.0263)	15.7 (108)

2.4 Pull-Out Test - Oiled Dowels

Pull-out tests were also conducted using the six different 1.5 inch (38 mm) nominal diameter dowels used during construction on STH 29 West. Obtained dowel bar types include the standard epoxy coated steel dowels (control), solid polished stainless steel manufactured by Avesta Sheffield, hollow-core polished stainless steel (mortar filled) manufactured by Damascus-Bishop, and composite dowels as manufactured by RJD, Creative Pultrusions, and Glasforms. Rectangular test beams, 6 x 6 x 12 inch (152 x 152 x 304 mm) were cast in a specially fabricated steel form using fly ash concrete produced in the Marquette lab and proportioned according to the job mix used during construction on STH 29 West. All dowel bars were oiled and positioned at the center of the 6 x 6 inch (152 x 152 mm) face, extending approximately 9 inches (228 mm) into the concrete beam.

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 them 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. 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 2.10 illustrates the modified test set-up.

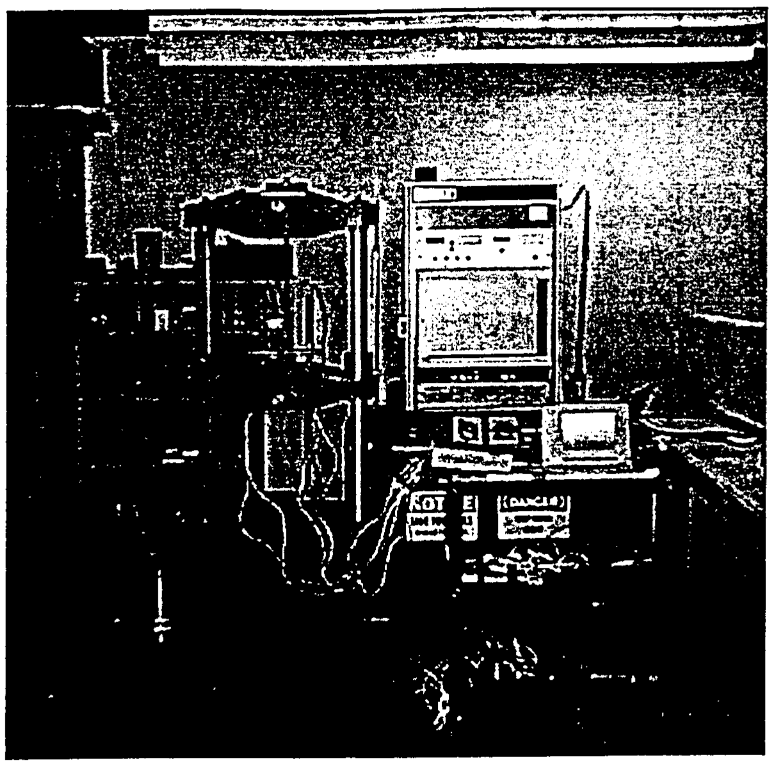

Figure 2.10 Modified Pull-out Test Set-up

The maximum pull-out loads and calculated maximum pull-out stresses are provided in Tables 2.3 and 2.4. Maximum pull-out stresses were calculated based on maximum pull-out loads and circumferential contact area between the dowel and the concrete at the start of testing. Figure 2.11 summarizes the pull-out load results for the complete test series.

Table 2.3: Summary of Pre Freeze-Thaw Pull-Out Tests on Oiled Dowels

Dowel Bar Type	Maximum Pull-Out Load lb (kN)	Circumferential Contact Area in^2 (m^2)	Maximum Pull-Out Stress psi (kPa)
Epoxy Coated	5853 (26.0)	41.79 (0.0270)	140.1 (966)
Polished Stainless Steel	5159 (22.9)	40.33 (0.0260)	127.9 (882)
Hollow-Filled Stainless Steel	4576 (20.4)	43.84 (0.0283)	104.4 (720)
Glasforms	1604 (7.1)	41.25 (0.0266)	38.9 (268)
Creative Pultrusions	1943 (8.6)	41.28 (0.0266)	47.1 (325)
RJD	1694 (7.5)	42.48 (0.0274)	39.9 (275)

Table 2.4: Summary of Post Freeze-Thaw Pull-Out Tests on Oiled Dowels

Dowel Bar Type	Maximum Pull-Out Load lb (kN)	Circumferential Contact Area in^2 (m^2)	Maximum Pull-Out Stress psi (kPa)
Epoxy Coated	8493 (37.8)	39.40 (0.0254)	215.6 (1486)
Polished Stainless Steel	995 (4.4)	37.98 (0.0245)	26.2 (181)
Hollow-Filled Stainless Steel	1716 (7.6)	41.48 (0.0268)	41.4 (285)
Glasforms	2064 (9.2)	38.89 (0.0251)	53.1 (366)
Creative Pultrusions	2630 (11.7)	38.93 (0.0251)	67.6 (466)
RJD	974 (4.3)	40.12 (0.0259)	24.3 (168)

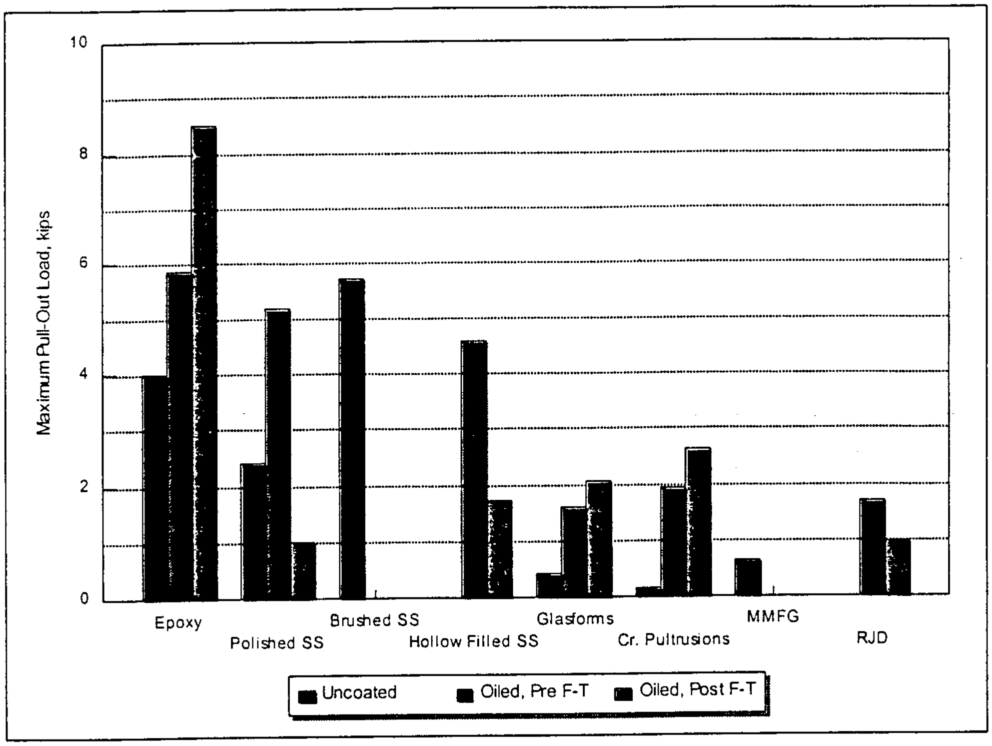

Figure 2.11 Summary of Pull-out Test Results

The maximum pull-out load typically occurred during the initial 0.002 inches (0.05 mm) of dowel displacement after which time the pull-out loads reduced to a significantly lower residual stress level. In all cases, the maximum pull-out stress for the composite dowels were significantly lower than for the control or stainless steel dowels. After freeze-thaw conditioning, the peak pull-out load was approximately equal to the pre freeze-thaw residual load, again occurring during the initial 0.002 inches (0.05 mm) displacement. A notable exception to this trend was the epoxy coated dowel. During initial testing, the peak pull-out stress occurred at approximately 0.05 inches (1.27 mm) displacement and only reduced slightly to a residual stress that remained essentially constant to a displacement of approximately 0.35 inches (8.9 mm). The pull-out stresses then began to increase with increasing displacements for the remaining 0.15 inches (3.8 mm) of displacement. After freeze-thaw conditioning, pull-out loads again continually increased with increasing displacement, with the most significant increase occurring during the initial 0.1 inches (1.27 mm) of displacement.

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. Longitudinal striations were noted on the surfaces of all dowels. Also, the exposed surfaces of the polished stainless steel dowels resembled the brushed stainless steel surfaces of the dowels used during uncoated tests.

3.0 TEST SECTION CONSTRUCTION

3.1 STH 29 West

Paving of the Eastbound lanes on STH 29 West incorporating all test sections was completed by E.J. Streu 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, which was designed as an English project. All paving within the limits of test section construction was completed using a single paver configuration which provided for a 25.6 ft (7.8 m) paved width with repetitive random joint spacings of 17-20-18-19 ft (5.2 - 6.1 - 5.5 - 5.8 m). The dowel bar inserter utilized fixed dowel spacings of 12 inches (304 mm) throughout the central portions of the slabs. The spacing between the outer dowel and the next dowel inwards was reduced to 9 inches (229 mm) on both pavement edges to account for the reduced paving width. Each outer dowel was positioned at 6 inches (152 mm) from the pavement 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 5 minutes and resulted in minimal paving delays. Paving with alternate dowel bar materials progressed with minimal 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. Composite dowels produced by Creative Pultrusions were supplied in wooden crates which resulted in handling problems during transfer to the dowel bar carriage.

Table 3.1 provides a daily summary of the paving operations and related test section construction. Signs denoting the limits of each test section were fabricated and placed by WisDOT staff near the ROW limits on the South edge of the highway. After construction, representative sections of approximately 528 ft (161 m) were selected from within each test section for long-term monitoring. 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. Each test section includes 29 transverse joints with the exception of the hollow filled stainless steel dowels where only 20 joints were constructed. Table 3.2 provides the station limits for each selected section, which are located at the center of each slab directly outside the first and last joints included within the monitoring sections.

Table 3.1 Paving Summary - STH 29 West

Day	Start Station	End Station	Comments
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 solid stainless steel dowels. Paving with Alternate 3 using solid stainless steel dowels.
09-16-97	71+688 71+384	71+384 70+997	Paving with Alternate 3 using solid stainless steel dowels. Paving with Alternate 3 using solid stainless 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* 2292+97 2276+85	Paving with standard placement using epoxy coated dowels. Paving with standard placement using hollow filled stainless steel dowels. Paving with standard placement using composite dowels (RJD). Paving with standard placement using composite dowels (Glasforms). Paving with standard placement using composite dowels (Creative Pultrusions).
09-18-97	2276+85	2264+29	Paving with standard placement using epoxy coated dowels.

* Station change from Metric to English, Sta 70+680 (M) = Sta 2138+89.76 (E)

Table 3.2 - Monitoring Section Locations - STH 29 West

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 Composite Dowels (Creative Pultrusions)
GF	2300+00	2305+32	Standard Placement with Composite Dowels (Glasforms)
RJD	2310+10	2315+43*	Standard Placement with Composite Dowels (RJD)
HF	70+867*	70+979	Standard Placement with Hollow Filled Stainless Steel Dowels
3Ea	71+047	71+210	Alternate 3 with Epoxy Coated Dowels
3SS	71+523	71+681	Alternate 3 with Solid Stainless Steel Dowels
4SS	71+898	72+060	Alternate 4 with 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 2138+89.76 (E)

3.2 STH 29 East

Paving of the Eastbound lanes on STH 29 East incorporating all Eastbound test sections was completed by James Cape & Sons during the period of October 16-17, 1997. 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 shipment of composite dowels produced by RJD was delayed which caused this test section to be placed West of the remaining alternate dowel material test sections. Furthermore, the remaining composite dowels were improperly distributed between the 12-foot (3.6 m) and 14-foot (4.3 m) basket lengths, resulting in all of the Glasforms composite bars being placed in 12-foot (3.6 m) baskets and most of the MMFG composite bars being placed in the 14-foot (4.3 m) baskets. As a result, of the 36 joints located within the composite section, 27 contained mismatches of manufacturers between the inner and outer lanes.

Table 3.3 provides a daily summary of the paving operations related to test section construction observed by Marquette University staff. Table 3.4 provides a listing of the composite dowel placement details between joint Stas. 1194+30 to 1200+60. After construction, representative sections of approximately 528 ft (161 m) were selected from within each 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 3.5 provides the station limits for each selected section, which are located at the center of each slab directly outside the first and last joints included within the monitoring sections.

Table 3.3 Paving Summary - STH 29 East

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 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 composite (RJD) dowels

Table 3.4 - Composite Dowel Placement Details

Joint Station	Outer Lane	Inner Lane
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 3.5 - Monitoring Section Locations - STH 29 East

Eastbound Lanes			
Section Code	Start Station	End Station	Comments
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 Solid Stainless Steel Dowels
Westbound Lanes			
Section Code	Start Station	End Station	Comments
1E	1207+44	1202+20	Alternate 1 with Epoxy Coated Dowels
C1	1200+23	1195+00	Control 1 - Standard Placement with Epoxy Coated Dowels
TR	1193+55	1188+28	Standard Placement with Epoxy Coated Dowels and Trapezoidal Slab Design

3.3 Dowel Bar Location Study - STH 29 West

A dowel bar location study was conducted in December, 1998 using the impact echo technique. The main objectives of this study were to determine the depth of dowel placement, the longitudinal position, and the transverse position of each dowel end on either side of constructed transverse joints. Impact echo tests were conducted by Dr. Al Ghorbanppor over three dowel positions near the outer pavement edge of each joint tested. Tests were conducted directly over the placement position at a distance of 6 inches (152 mm) on either side of the joint. Table 3.6 provides a summary of the measured dowels depths within test section C1, and the three composite dowel test sections. Dowel depth data were inconclusive from within the hollow filled and solid stainless steel sections. Also, the test results could not provide exact longitudinal and transverse positions of each dowel end.

Table 3.6 - Summary of Dowel Bar Location Study - STH 29 West

Test Section	No. of Joints Tested	Average Depth West Side of Joint, in (mm)	Average Depth East Side of Joint, in (mm)	Average Depth Variation in (mm)
C1	1	6.04 (153)	5.86 (149)	0.18 (4.6)
CP	2	6.17 (157)	5.97 (152)	0.21 (5.3)
GF	5	6.12 (155)	6.00 (152)	0.47 (11.9)
RJD	7	6.04 (153)	6.05 (154)	0.20 (5.1)

4.0 NONDESTRUCTIVE DEFLECTION TESTING PROGRAM

4.1 Introduction

Nondestructive deflection testing (NDT) was conducted along STH 29 East and West to provide a measure of the structural response of the pavement system to loads similar in magnitude and duration to moving truck loadings. The NDT program 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 an 11.81 inch (300 mm) segmented load plate. The magnitude of the dynamic load is varied by adjusting the height of fall of the primary weight package. Post-construction deflection tests were conducted immediately prior to opening to public traffic as well as after six and 12 months of traffic. Additionally, deflection testing was conducted along STH 29 West prior to paving operations.

4.2 STH 29 West - Pre-Paving Deflection Testing

Deflection tests were conducted along STH 29 West immediately prior to the paving operations to provide a measure of the strength and uniformity of the foundation materials. Deflection tests were conducted with the Marquette University 2m-FWD at approximately 325 ft (100 m) intervals along the outer traffic lane from stations 79+900 to 69+787 (2289+00). Additional tests were conducted at selected locations along the inner lane at 325 ft (100 m) intervals, staggered 160 ft (50 m) from the outer lane tests, from stations 72+150 to 79+650 The smallest load level of approximately 3,000 lb (13.3 kN) was used to provide top-of-foundation stress levels as close as possible to those which would be induced during post-paving testing at the 9,000 lb (40 kN) load level. It should be noted, however, that this applied top-of-foundation stress level is significantly greater than the 0.5 - 1.0 psi (3.4 - 6.8 kPa) stress level which would be anticipated under the 9,000 lb (40 kN) load after the 11 inch (275 mm) concrete slab is in place. As such, foundation material properties which are derived from these pre-paving surface deflections may be significantly lower than those derived from post-paving deflections due to the stress-dependent behavior of the foundation materials.

The maximum deflection under loading, normalized to a common 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. Table 4.1 provides summary statistics for the maximum deflections, normalized to a 3,000 lb (13.3 kN) load, along the inner and outer lanes within the limits of testing. As shown, all values (mean, standard deviation and coefficient of variation) are higher along the inner lane.

Table 4.1 Summary Statistics of Maximum Pre-Paving Deflections - STH 29 West

Test Lane	Outer	Inner
Overall Mean mils @ 3,000 lb (μm @ 13.3 kN)	21.36 (542.5)	25.52 (648.2)
Std. Deviation mils @ 3,000 lb (μm @ 13.3 kN)	9.88 (251.0)	14.68 (372.9)
Coefficient of Variation, %	46.2	57.5

The maximum deflection as well as deflections away from the center of loading may be used to estimate the elastic moduli of foundation materials. Using single-layer elastic layer theory, an approximation of the combined moduli of the base-subgrade may by obtained from the maximum deflection under loading using the equation:

$$E_{comb} = \frac{1.5\ P}{\pi\ a\ d_0}$$

where: E_{comb} = Combined Elastic Modulus, ksi
P = applied load, lb
a = load radius, in
d_0 = maximum deflection, mils

The subgrade elastic moduli may be approximated following guidelines presented in the 1993 AASHTO Design Guide using deflections away from the center of loading by the equation:

$$E_{sg} = \frac{0.24\ P}{d_r\ r}$$

where: E_{sg} = subgrade elastic modulus, ksi
P = applied load, lb
d_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, E_{sg} values should be computed based on all measured deflections with r > 0 and the minimum computed E_{sg} used to estimate the subgrade elastic modulus.

The breakpoint resilient modulus of the subgrade may be estimated based on surface deflections recorded at 36 inches form the center of loading using the equation:

$$E_{ri} = 24.2289 - 5.7114\ d_{36} + 0.3513\ d_{36}^{2}$$

where: E_{ri} = breakpoint resilient modulus, ksi
d_{36} = surface deflection at 36 inches from load center, mils

Table 4.2 provides summary statistics for estimated moduli values, calculated with the above equations, from deflections collected along the outer lane.

Table 4.2 Summary Statistics For Estimated Moduli Values - STH 29 West

Estimated Parameter	Combined Base/Subgrade Elastic Modulus	AASHTO Subgrade Elastic Modulus	Subgrade Breakpoint Resilient Modulus
Mean Value, ksi (MPa)	13.5 (93)	10.2 (70)	10.7 (74)
Standard Deviation ksi (MPa)	5.4 (37)	4.7 (32)	3.8 (26)
Coefficient of Variation, %	39.8	46.5	35.9

4.3 STH 29 West - Post-Paving Deflection Testing

Deflection tests were conducted within established PDI sections along the outer lane of STH 29 West after completion of paving operations but before opening to public traffic as well as after 6 months and 12 months of trafficking. Pre-opening deflection tests were conducted with the Marquette University 2m-FWD. Tests conducted after six months of trafficking were initially conducted with the Marquette University 2m-FWD, but due to equipment problems the ERI 2m-FWD was used to complete testing. Tests conducted after 12 months of trafficking were conducted using the ERI 2m-FWD. Both 2m-FWD's are of the same make and utilize the similar load plates, load cells, and deflection sensors and thus no significant differences in measured deflection are expected.

Pre-opening deflection tests were conducted within 5 test slabs located at approximately 100 ft (30 m) intervals. Center slab tests were conducted to establish baseline values for foundation k-values and concrete elastic moduli. Transverse joint tests were conducted in the outer wheelpath to establish initial values for deflection load transfer. Additional mid-lane transverse joint tests were conducted in alternate dowel placement test sections as well as within the Control 1 test section to provide within section comparative values.

The foundation k-value was estimated from center slab deflections using deflections recorded at 0, 12, 24 and 36 inches (0, 304, 608, 912 mm) from the center of the 5.91 inch (150 mm) radius circular load. Initially, the deflection basin AREA was computed using the equation:

$$AREA = \frac{6}{d_0}\left(d_0 + 2\,d_{12} + 2\,d_{24} + d_{36}\right)$$

where: AREA = deflection basin AREA, in

d_i = surface deflection measure at i inches from the load center

The calculated AREA value was then used to backcalculate the radius of relative stiffness of the pavement system (dense-liquid foundation model) using the equation:

$$l_k = \left[\frac{\ln\left(\frac{36 - AREA}{1812.279133}\right)}{-2.55934}\right]^{4.387009}$$

where: l_k = dense-liquid radius of relative stiffness, in

The dense-liquid radius of relative stiffness is a combined term which incorporates both slab and subgrade properties and may be computed by the equation:

$$l_k = \sqrt[4]{\frac{E_c\,H_c^{3}}{12\,(1-\mu_c^{2})\,k}}$$

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

The subgrade k-value was then backcalculated using the equation:

$$k = \frac{1000\ P}{d_0\ l_k^2}\left[0.1253 - 0.008\ \frac{a}{l_k} - 0.028\left(\frac{a}{l_k}\right)^2\right]$$

where: P = applied load, lb
d_0 = maximum center slab deflection (0 inches from load center), mils
a = radius of load, in

After backcalculation of the subgrade k-value, the elastic modulus of the concrete slab was estimated from the equation for l_k as follows:

$$E_c = \frac{11.73\ l_k^4\ k}{H_c^3}$$

where: H_c = known or assumed slab thickness, in

The above equations generally provide reasonable estimates for slab and foundation properties provided the effective slab dimensions exceed 5 times the radius of relative stiffness and the slabs are not excessively curled due to temperature gradients. For practical applications, highway slabs with joint spacings exceeding approximately 15 feet do not require adjustments due to slab size effects. However, through slab temperature gradients during the June 1998 testing (top warmer than bottom) produced sufficient downward curling to create zones of non-contact near the slab center. In these cases, incremental analysis using two test load levels was 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 was computed from backcalculated k and lk values using the equation:

$$E_{sg} = 0.00343\ k\ l_k$$

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

The deflection load transfer across the transverse joints was calculated using the equation:

$$LT\% = \frac{d_{12}}{d_0} \times 100\%$$

where: LT% = deflection load transfer efficiency, %
d_{12} = deflection on unloaded slab at 12 inches from load center, mils
d_0 = deflection on loaded slab at 0 inches from load center, mils

For doweled concrete pavements, deflection load transfer efficiencies of approximately 90% or greater are generally expected to indicate properly performing joints. A secondary measure of joint behavior is obtained by determining the total joint deflection under loading. The normalized total joint deflection was computed using the equation:

$$d_t = d_0 + d_{12}$$

where: dt = normalized total joint deflection, mils@9k
d_0 = deflection on loaded slab at 0 inches from load center, mils@9k
d_{12} = deflection on unloaded slab at 12 inches from load center, mils@9k

For equivalent slab systems (equal E_c, H_c, k), the total joint deflection should remain essentially constant, regardless of joint load transfer efficiency. For comparative measures, increasing total joint deflection represents decreasing E_c, H_c, and or k, and vice versa.

Table 4.3 provides summary statistics for the mean dynamic k-values backcalculated within each test section from the three post-construction cycles of deflection testing. Table 4.4 provides summary statistics for the average backcalculated concrete elastic modulus within each test section for the three test series.

The average k-values obtained from each test series indicate general uniformity within all test sections. There is, however, appreciably more variation in the results obtained from the June 1998 test series. This trend is most likely due to slab temperature variations which were not fully compensated for by incremental analysis. There is also a marked decrease in average k-values for both 1998 test series as compared to the 1997 results. The exact reason for this trend is not fully understood. Future testing scheduled for June and November 1999 may help clarify these results.

The average concrete moduli obtained from each test series indicate general uniformity within test sections with the June 1998 data again showing the highest variability. As expected, the mean concrete moduli increase with age. However, it would normally be expected that the most notable increase in moduli would occur during the first six months after paving, with more gradual increases thereafter. This trend may have been delayed due to the last season paving.

Table 4.3 Summary Statistics for Mean k-values - STH 29 West

Test Cycle	Oct 1997	Jun 1998	Nov 1998
Average Test Section Mean psi/in (kPa/mm)	312 (85)	252 (68)	254 (69)
Std. Deviation of Sample Means psi/in (kPa/mm)	27.6 (8)	60.1 (16)	35.0 (10)
Coefficient of Variation of Sample Means, %	8.9	23.9	13.8
Overall Mean psi/in (kPa/mm)	312 (85.7)	255 (69)	254 (69)
Overall Standard Deviation psi/in (kPa/mm)	53.3 (15)	92.6 (25)	52.9 (14)
Overall Coefficient of Variation, %	17.1	36.4	20.8

Table 4.4 Summary Statistics for Mean Concrete Moduli - STH 29 West

Test Cycle	Oct 1997	Jun 1998	Nov 1998
Average Section Mean Mpsi (GPa)	3.56 (24.5)	3.90 (26.9)	4.80 (33.1)
Std. Deviation of Sample Means Mpsi (GPa)	0.25 (1.7)	0.78 (5.4)	0.44 (3.0)
Coefficient of Variation of Sample Means, %	6.9	20.0	9.2
Overall Mean, Mpsi (GPa)	3.56 (24.5)	3.87 (26.7)	4.82 (33.2)
Overall Standard Deviation Mpsi (GPa)	0.78 (5.4)	1.39 (9.6)	0.85 (5.9)
Overall Coefficient of Variation, %	21.9	35.9	17.6

Note: 1 Mpsi = 1,000,000 psi = 4.4 GPa

Figure 4.1 illustrates the calculated average outer wheelpath transverse joint deflection load transfer within each test section for each test series. Late season tests (Oct 1997 and Nov 1998) indicate significantly reduced load transfer efficiencies in the composite dowel sections and Alternate 1 (3 dowels in each wheelpath) as compared to the control sections. There is also a slight reduction in load transfer efficiency noted in the stainless steel sections (solid and hollow filled). Load transfer efficiencies measured during the warmer period (Jun 1998) do not indicate any significant differences within any test section. This is most likely due to the closing of the joints due to thermal expansion of the slabs and the concurrent increase in aggregate interlock across the joints.

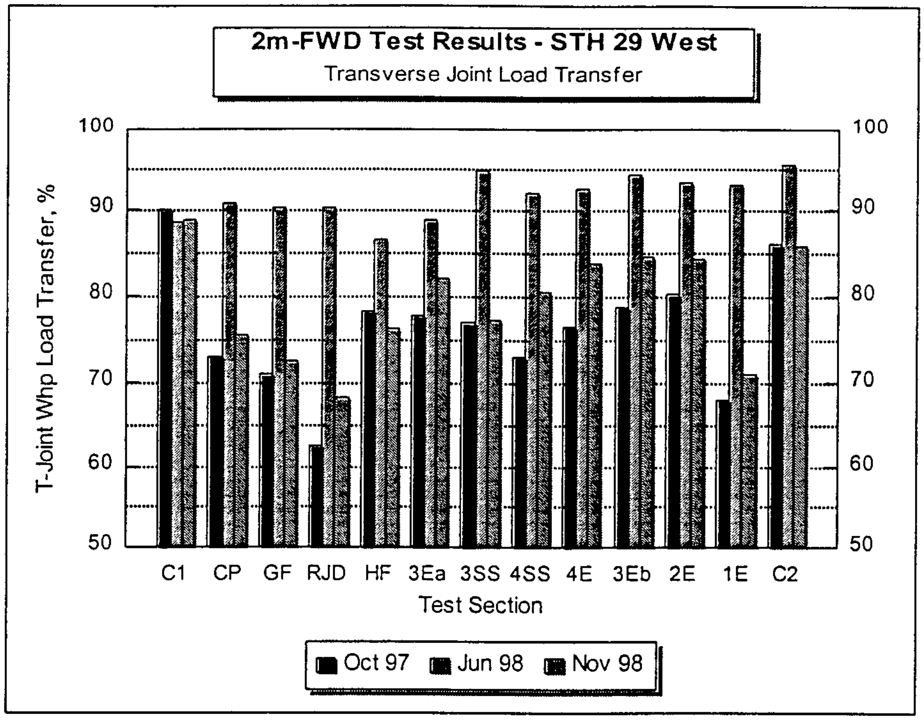

Figure 4.1 Outer Wheelpath Transverse Joint Load Transfer - STH 29 West

Figure 4.2 illustrates the average midlane and wheelpath transverse joint deflection load transfers measured during the October 1997 test series. As shown, the midlane load transfers in all alternate placement test sections, where no dowels are present, are significantly lower than the comparative wheelpath load transfers. Furthermore, the wheelpath load transfer measured in the RJD test section is approximately equal to the midlane load transfer in the alternate test sections. Also, as expected, no significant differences between midlane and wheelpath load transfers are noted in the control section C1.

Figure 4.3 illustrates the average total transverse wheelpath joint deflection measured within each test section for each test series. Table 4.5 provides summary statistics for these results.

The average total joint deflections from each test series indicate general uniformity amongst all sections. The apparent increase in total joint deflection in sections C2 and 4E during Oct 1997 and Nov 1998 testing is most likely due to the fact that these sections were tested at the start of each day when upwards curling is maximum.

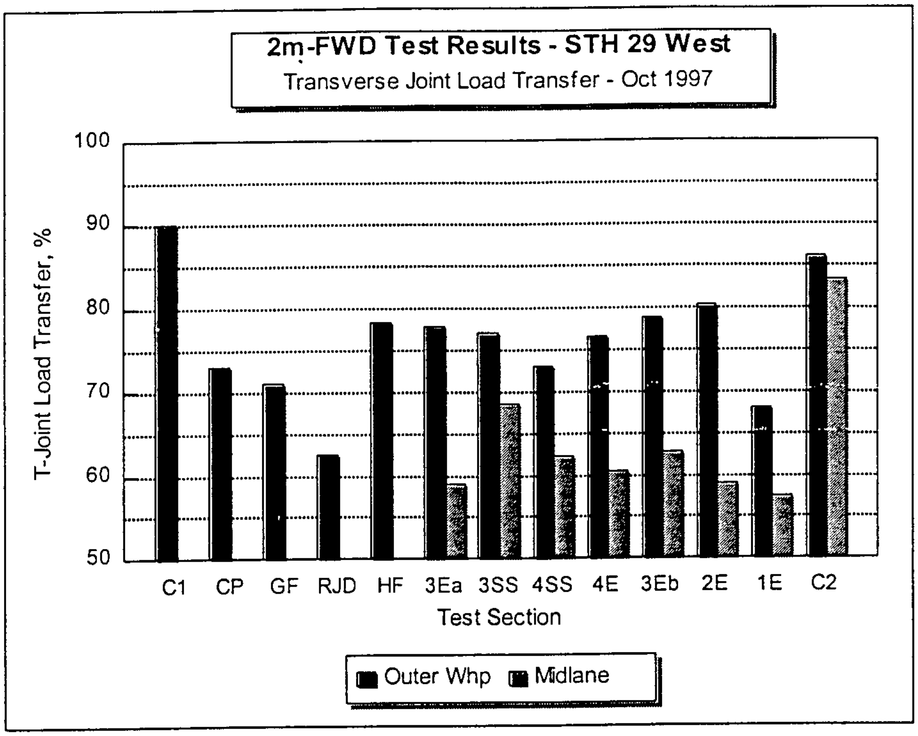

Figure 4.2 Midlane and Outer Wheelpath Joint Load Transfer - STH 29 West

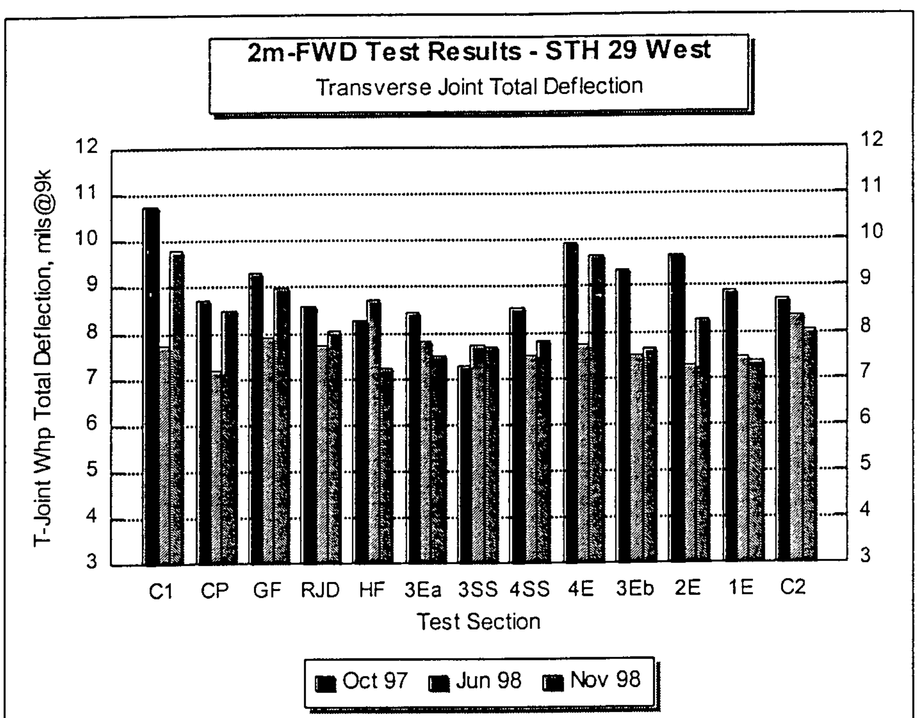

Figure 4.3 Total Wheelpath Transverse Joint Deflection - STH 29 West

Table 4.5 Summary Statistics for Total Joint Deflection - STH 29 West

Test Cycle	Oct 1997	Jun 1998	Nov 1998
Average Section Mean mils @ 9,000 lb (μm @ 40 kN)	8.94 (227)	7.72 (196)	8.18 (208)
Std. Deviation of Sample Means mils @ 9,000 lb (μm @ 40 kN)	0.86 (22)	0.41 (10)	0.83 (21)
Coefficient of Variation of Sample Means, %	9.6	5.3	10.2
Overall Mean mils @ 9,000 lb (μm @ 40 kN)	8.96 (228)	7.77 (197)	8.18 (208)
Overall Standard Deviation mils @ 9,000 lb (μm @ 40 kN)	1.06 (27)	0.93 (24)	0.98 (25)
Overall Coefficient of Variation, %	11.8	11.9	12.0

4.4 STH 29 East - Post-Paving Deflection Testing

Deflection tests were conducted within established PDI sections along the outer lane of STH 29 East after 6 months and 12 months of trafficking. Additionally, deflection tests were conducted within the Westbound test sections after completion of paving operations but before opening to public traffic. Pre-opening deflection tests were conducted with the Marquette University 2m-FWD. Tests conducted after six months of trafficking were conducted with the ERI 2m-FWD. Tests conducted after 12 months of trafficking were conducted using the Marquette University 2m-FWD. Both 2m-FWD's are of the same make and utilize the similar load plates, load cells, and deflection sensors and thus no significant differences in measured deflection are expected.

Pre-opening deflection tests were conducted within 5 test slabs located at approximately 150 ft (50 m) intervals along the Eastbound lanes. Center slab tests were conducted to establish baseline values for foundation k-values and concrete elastic moduli. Transverse joint tests were conducted in the outer wheelpath to establish initial values for deflection load transfer. Mid-lane transverse joint tests were conducted in all Eastbound test sections to provide within section comparative values.

Table 4.6 provides summary statistics for the mean dynamic k-values backcalculated withing each test section from the three post-construction cycles of deflection testing. The average k-values obtained from each test series indicate general uniformity within all test sections for each travel direction. There is, however, appreciably more variation in the results obtained from the June 1998 test series along the Westbound lanes. This trend is most likely due to slab temperature variations which were not fully compensated for by incremental analysis. There is also a marked decrease in average k-values for the Westbound lanes as compared to the Eastbound lanes.

Table 4.6 Summary Statistics for Mean k-values - STH 29 East

Travel Direction	Eastbound			Westbound	
Test Cycle	Oct 1997	Jun 1998	Nov 1998	Jun 1998	Nov 1998
Average Test Section Mean psi/in (kPa/mm)	369 (100)	324 (88)	322 (87)	255 (69)	222 (60)
Std. Deviation of Sample Means psi/in (kPa/mm)	59.7 (16)	72.1 (20)	32.2 (9)	86.5 (23)	38.5 (10)
Coefficient of Variation of Sample Means, %	16.2	22.3	10.0	33.9	17.4
Overall Mean psi/in (kPa/mm)	364 (99)	324 (88)	324 (88)	255 (69)	222 (60)
Overall Standard Deviation psi/in (kPa/mm)	93.2 (25)	91.2 (25)	86.4 (23)	111.8 (30)	52.4 (14)
Overall Coefficient of Variation, %	25.6	28.1	26.7	43.9	23.6

Table 4.7 provides summary statistics for the average backcalculated concrete elastic modulus within each test section for the three test series using an assumed slab thickness of 11.0 inches (275 mm). The average concrete moduli obtained from each test series indicate general uniformity within test sections for each travel direction with the June 1998 data again showing the highest variability.

Table 4.7 Summary Statistics for Mean Concrete Moduli - STH 29 East

Travel Direction	Eastbound			Westbound	
Test Cycle	Oct 1997	Jun 1998	Nov 1998	Jun 1998	Nov 1998
Average Test Section Mean, Mpsi (GPa)	3.95 (17.6)	5.99 (26.6)	6.09 (27.1)	5.31 (23.6)	6.12 (27.2)
Standard Deviation of Sample Means Mpsi	0.44 (2.0)	0.85 (3.8)	0.43 (1.9)	1.20 (5.3)	0.58 (2.6)
Coefficient of Variation of Sample Means, %	11.1	14.2	7.0	22.6	9.5
Overall Mean Mpsi (GPa)	3.97 (17.6)	5.99 (26.6)	6.06 (27.0)	5.29 (23.5)	6.13 (27.3)
Overall Standard Deviation Mpsi (GPa)	1.13 (5.0)	1.53 (6.8)	1.88 (9.4)	2.59 (11.5)	1.20 (5.3)
Overall Coefficient of Variation, %	28.4	25.6	30.9	49.0	19.6

Note: 1 Mpsi = 1,000,000 psi = 4.4 GPa

Figure 4.4 provides an illustration of the calculated average outer wheelpath transverse joint deflection load transfer within each test section for each test series. The final test series (Nov 998) indicates significantly reduced load transfer efficiencies in the composite dowel sections and Alternate 1 (3 dowels in each wheelpath) as compared to the control sections. There is also a slight reduction in load transfer efficiency noted in the stainless steel section. Load transfer efficiencies measured during the warmer period (Jun 1998) do not indicate any significant differences within any test section. This is most likely due to the closing of the joints due to thermal expansion of the slabs and the concurrent increase in aggregate interlock across the joints.

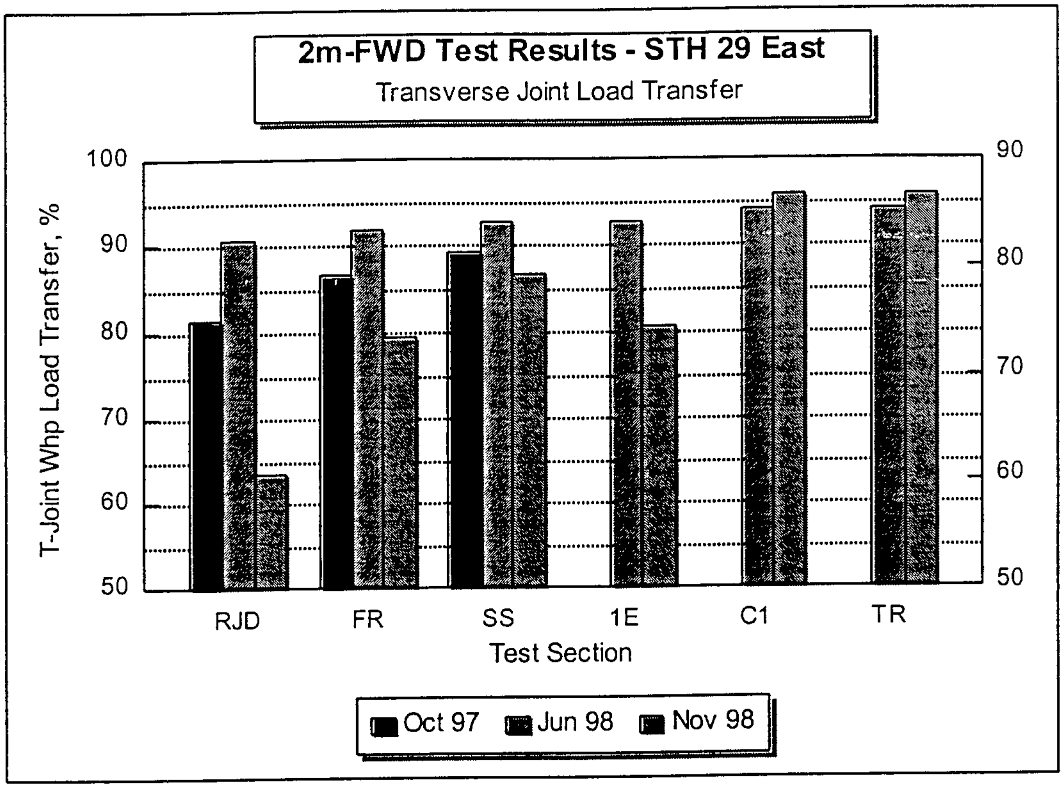

Figure 4.4 Outer Wheelpath Transverse Joint Load Transfer - STH 29 East

Figure 4.5 illustrates the average midlane and wheelpath transverse joint deflection load transfers measured during the October 1997 test series. As shown, no significant differences between midlane and wheelpath load transfers are noted in any test section.

Figure 4.6 illustrates the average total transverse wheelpath joint deflection measured within each test section for each test series. Table 4.8 provides summary statistics for these results.

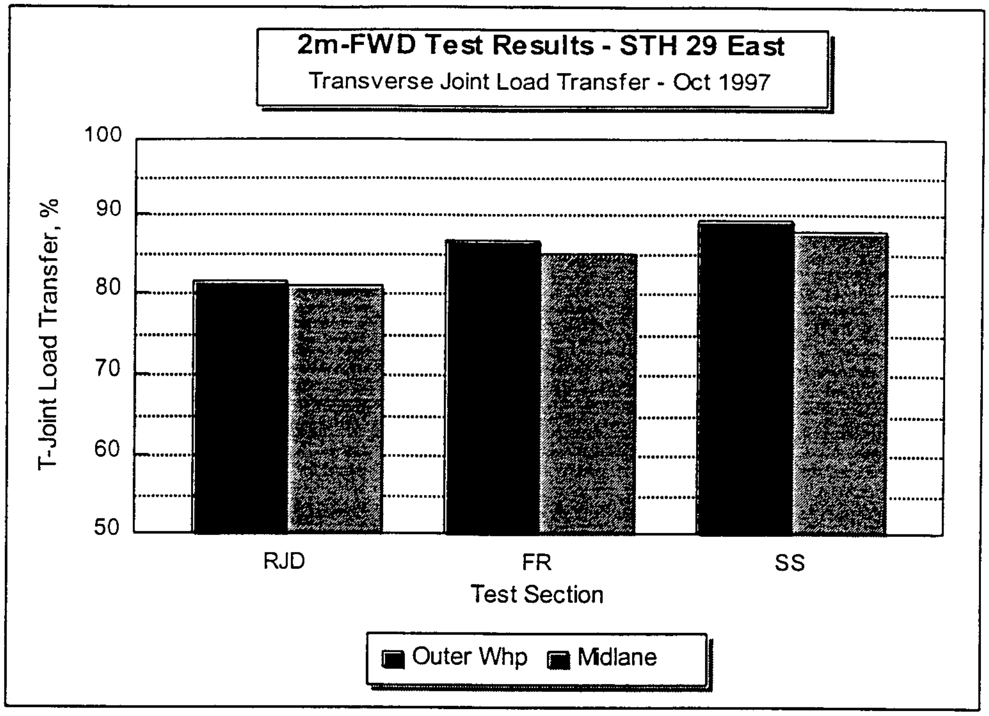

Figure 4.5 Midlane and Outer Wheelpath Joint Load Transfer - STH 29 East

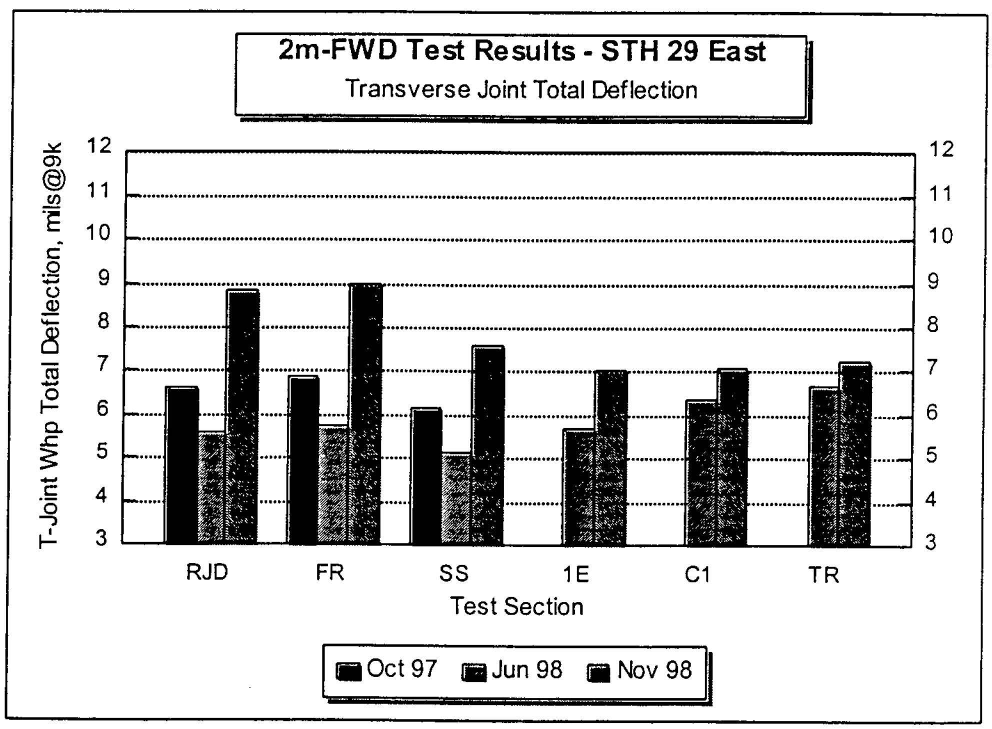

Figure 4.6 Total Outer Wheelpath Transverse Deflection - STH 29 East

Table 4.8 Summary Statistics for Total Joint Deflection - STH 29 East

Travel Direction	Eastbound			Westbound	
Test Cycle	Oct 1997	Jun 1998	Nov 1998	Jun 1998	Nov 1998
Average Test Section Mean mils @ 9,000 lb (μm @ 40 kN)	6.69 (170)	5.56 (141)	8.51 (216)	6.22 (158)	7.11 (181)
Standard Deviation of Sample Means mils @ 9,000 lb (μm @ 40 kN)	0.33 (8)	0.48 (12)	1.61 (41)	0.50 (13)	0.11 (3)
Coefficient of Variation of Sample Means, %	4.9	8.7	18.9	8.0	1.5
Overall Mean mils @ 9,000 lb (μm @ 40 kN)	6.70 (170)	5.56 (141)	8.48 (215)	6.23 (158)	7.11 (181)
Overall Standard Deviation mils @ 9,000 lb (μm @ 40 kN)	0.83 (21)	0.60 (15)	1.85 (47)	0.63 (16)	0.48 (12)
Overall Coefficient of Variation, %	12.4	10.7	21.8	10.1	6.7

The average total joint deflections from each test series indicate general uniformity within all sections. The apparent increase in total joint deflection in sections C1 and the composite dowels (RJD, FR) during Nov 1998 testing is most likely due to the fact that these sections were tested early morning when upwards curling is maximum.

5.0 JOINT DISTRESS SURVEY

5.1 Introduction

Visual joint distress surveys were conducted prior to opening to public traffic and after 6 and 12 months of trafficking. All sections surveyed during each time period with the exception of Westbound test sections along STH 29 East. Surveys within these sections were completed only after 6 and 12 months of trafficking. All 29 joints located within each monitoring section were surveyed for distress. For most test sections, an additional 21 joints located adjacent to each test section were also surveyed to expand the database. Typically 11 joints immediately West of the monitoring section and 10 joints immediately East of the monitoring section were included in the survey. Exceptions to the above include test sections constructed with fewer than 50 joints and STH 29 West test sections 1E and 3Ea. In test section 1E, only 4 additional joints were included West of the monitoring section to avoid a nearby structure and 17 additional joints were added east of the monitoring section. Test section 3Ea is positioned just East of the Hollow Filled (HF) test section and thus only six joints West of the monitoring section were available. The remaining 15 additional joint were selected East of the monitoring section.

During the survey, separate records were maintained for the inner and outer lanes of travel. Any joints with cracking, breaking, chipping or fraying along the slab edges were noted as well as the approximate location of the distress. Distress severity levels were recorded following SHRP/LTPP guidelines.

5.2 Joint Surveys - STH 29 West

All visual joints surveys conducted on STH 29 West indicate good joint performance to date. Observed spalling and chipping was primarily the result of the transverse joint saw cutting operations which dislodged aggregates near the joint faces. Dislodged aggregates were noted along the transverse joints as well as at the outer slab corners. All joints with dislodged aggregates were rated as having low severity joint spalling. This distress, however, has not yet progressed to the point to be considered as low severity distressed joints based on WisDOT Pavement Distress Index (PDI) guidelines. No transverse joint faulting, slab cracking, or other surface distress exists within any test section. All sections along STH 29 West have a current PDI = 0.

Figure 5.1 illustrates the percentage of spalled (spalled, chipped or frayed) joints within each test section recorded during the June 1998 and December 1998 surveys.

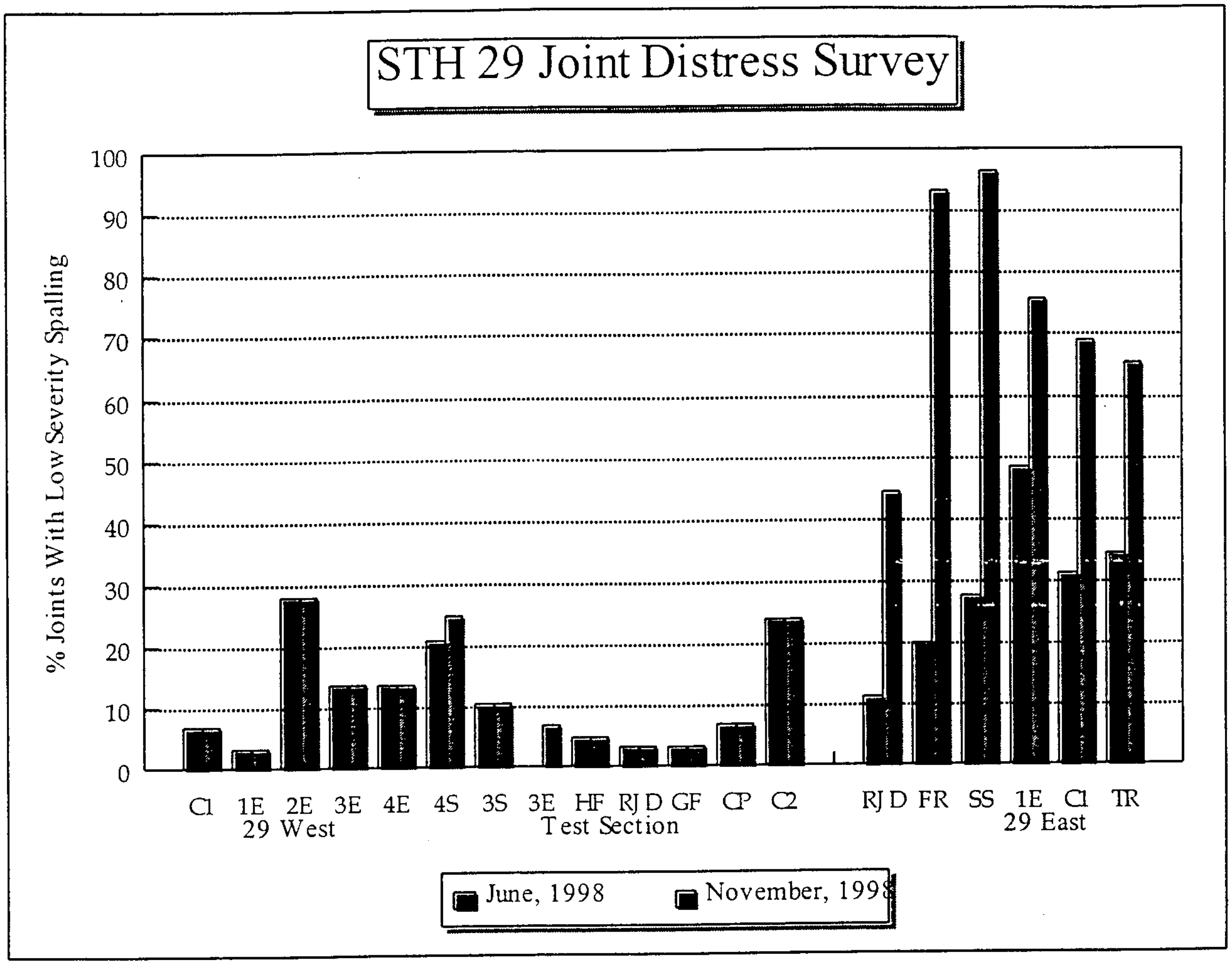

Figure 5.1 Summary of Transverse Joint Spalling

5.3 Joint Surveys - STH 29 East

All visual joints surveys conducted on STH 29 East indicate generally good joint performance to date. Observed spalling, chipping and fraying was primarily the result of the transverse joint saw cutting operations which dislodged aggregates and portions of the tined surface near the joint faces. Dislodged aggregates were noted along the transverse joints as well as at the outer slab corners. All joints with dislodged aggregates, chipped or frayed edges were most currently rated as having low severity joint spalling. Joints with limited frayed edges were not severity rated during the June 1998 survey. During the November 1998 survey these joints were rated as low severity. Transverse joint distress has not yet progressed to the point to be considered as low severity distressed joints based on WisDOT PDI guidelines. No transverse joint faulting, slab cracking, or other surface distress exists within any test section. All sections along STH 29 West have a current PDI = 0.

Figure 5.1illustrates the percentage of spalled (spalled, chipped or frayed) joints within each monitoring section recorded during the June 1998 and December 1998 surveys. The apparent dramatic increase in spalling between June and December 1998 is due to the fact that joints with only limited fraying along the edges were not recorded during the June 1998 survey.

6.0 CONCLUSIONS

This report has presented details relating to the design, construction, and first year performance of concrete pavement test sections constructed in the State of Wisconsin along STH 29 in Clark (STH 29 West) and Marathon (STH 29 East) Counties. These test sections were constructed during the Summer of 1997 to validate the constructability and cost-effectiveness of alternative concrete pavement designs incorporating variable dowel placement strategies, variable dowel materials, and variable slab thicknesses.

Eleven test sections were constructed along STH 29 West using variable dowel placement strategies and dowel materials. A Gomaco paver fitted with an automatic dowel bar inserter (DBI) was used during construction. All dowel materials used during construction were easily accommodated by the DBI. The versatility of the dowel bar inserter was also demonstrated during the construction of the placement alternates 3 and 4. Changes to the DBI, including opening and closing of three insertion ports near the outer pavement edge, were completed in approximately 10 minutes without interruptions to the paving operations. Five test sections were constructed along STH 29 East using variable dowel placement strategies, dowel materials, and slab thicknesses. A Rex paver was used during construction with all dowels placed on dowel baskets. No problems associated with these variable designs were encountered during construction.

Post construction monitoring, including joint distress surveys and deflections testing, has been completed during the first year of service. Observed joint distress, including minor spalling, chipping, and fraying, is predominately attributable to the joint saw cut operations. No transverse joint faulting or slab cracking has been observed to date. Center slab deflection testing conducted to date has indicated general uniformity of foundation support within all test sections. Deflection testing has also been conducted across transverse joints to quantify deflection load transfer efficiency. Joint tests conducted during the Fall of 1997 and 1998 indicate reduced load transfer efficiencies within all test sections as compared to control sections, most notably within the composite dowel test sections and placement alternate 3 (3 dowels in each wheelpath). Tests conducted during the late Spring of 1998 do not indicate any significant differences in any test section. Pavement monitoring will continue for the next four years, with additional joint surveys and deflection tests conducted during the Spring of 1999 and during the Fall of each year.

Additional test sections, including variable slab thickness and drainage layer designs, will be constructed during the Summer of 1999 along STH 29 in Shawano County between Wittenberg and Shawano. A second interim report, detailing the design, construction, and first year performance of these test sections will be prepared by Marquette University staff and submitted by December 2000. This second interim report will also include all monitoring data collected between 1998 and 2000 on the pavement test sections constructed in 1997 and reported herein.