EVALUATION OF ALTERNATIVE DOWEL BAR MATERIALS AND COATINGS

Final Report

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This work was initiated under the May 1988 HITEC Final Evaluation Plan. A detailed summary of that effort was provided in a draft Interim Report dated March 31, 2005.

16. Abstract

This study provided for a continuation of the long-term performance evaluation of 1.5-in (38-mm) diameter FRP dowels and Type 304 stainless steel solid dowels or mortar-filled tubes compared to epoxy-coated dowels. This primarily included an evaluation of load transfer efficiency (LTE) based on FWD testing, but also included a limited evaluation of faulting and ride. In addition, some FWD testing and coring was conducted on older projects (15 to 30+ years old) to evaluate the long-term corrosion protection provided by epoxy coatings.

The 1.5-in (38 mm) FRP dowels with polyester resin and E-glass exhibited generally low LTE values, and are not providing performance comparable to that of the 1.5-in (38-mm) epoxy-coated mild steel dowel bars. The evaluation of alternative stainless steel clad dowels and concrete filled stainless steel tubes or pipes (Type 304 or Type 316) was inconclusive due to the small sample and the relatively short (14 years maximum) evaluation period, but it appears that they will perform satisfactorily in excess of 30 years given the minimal deterioration observed.

Based on the coring of the older pavement projects, the life of the epoxy coating on mild steel dowels evaluated in Ohio and Wisconsin appears to be in the 25 to 30-year range. Many of the epoxy-coated dowels retrieved from in-service pavements revealed that the epoxy coating was debonded from the mild steel dowel and the surface of the mild steel dowel under the coating was pitted and rusted. In most cases, however, there was no significant loss of cross section.

A review of two older projects in Ohio constructed with plastic-coated dowels indicated that those dowels were in excellent condition after 33 years. The overall condition of these projects was also very good, with little if any visible joint deterioration. Because plastic-coated dowels are similar to epoxy-coated dowels in terms of costs, they appear to be a cost-effective alternative to conventional epoxy-coated dowels.

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Acknowledgments

The researchers extend their appreciation to Mr. Roger Green for his guidance and direction in the conduct of this study. The members of the Technical Advisory Panel—Mr. Andy Gisi, Kansas DOT; Ms. Irene Battaglia and Mr. Barry Paye, Wisconsin DOT, and Mr. Mark Gawedzinski, Illinois DOT—are acknowledged for their contributions to this project, along with Dr. Max Porter, Iowa State University, Dr. Seung-Kyoung Lee, FHWA, and Dr. Paul Virmani, FHWA, all of whom participated in meetings and conference calls and provided valuable input.

Thanks are also due to the field testing crews of the Ohio and Wisconsin Departments of Transportation for collecting the additional field testing data required by the project and providing it in a format useful for inclusion under this study; Mr. Roger Green and Mr. Barry Paye are again recognized for their roles in overseeing these activities in their respective agencies. Thanks are also given to Dr. Jim Crovetti, Marquette University, for the information and documentation that he provided on the projects in Wisconsin.

Gratitude is also expressed to a number of other colleagues and professionals who provided valuable information for use in the study: Dr. John Harvey, University of California, Davis, who provided results of their laboratory testing of alternative dowel bars; Mr. Steve Tritsch and Mr. Kevin Ruesch, Commercial Metals Corporation, who provided information on the use and specifications for plastic-coated dowels in Louisiana; Mr. Denis Thebeau, Quebec Ministry of Transport, who provided information on the University of Sherbrooke laboratory study of FRP dowels; and Mr. Doug Gremel, Hughes Brothers, who provided references and comments on desirable RFP specification requirements. These and many other individuals all willingly shared a significant amount of information relevant to this project and their assistance is greatly appreciated.

Preface

Under a contract for the Highway Innovative Technology Evaluation Center (HITEC), a draft *Interim Report* on alternative dowel bars was prepared in March 2005 that briefly documented some of the early experience with alternative dowel bars and proposed a field evaluation program for existing alternative dowel bar installations. However, the HITEC program was terminated shortly thereafter, and the Ohio Department of Transportation (ODOT) led a pooled-fund effort to continue the alternative dowel bar work initiated by HITEC. A new contract (ODOT State Job Number 134411, Transportation Pooled Fund [TPF] Project 5(188)) provided for an extended evaluation of the original HITEC alternative dowel bar material projects constructed in 1996-1998 with additional monitoring in 2009, 2010, and 2011. This *Final Report* documents the result of that expanded evaluation.

Mr. Roger Green of the Ohio Department of Transportation (DOT) served as Chair of the TPF-5(188) Technical Advisory Panel, and was joined on the panel by Mr. Andy Gisi, Kansas DOT; Mr. Barry Paye, Wisconsin DOT; and Mr. Mark Gawedzinski, Illinois DOT. In addition, Dr. Max Porter, Iowa State University, and Dr. Paul Virmani were corresponding members of the advisory panel. Ms. Irene Battaglia, formerly of the Wisconsin DOT, and Dr. Seung-Kyoung Lee, formerly of FHWA, also served as members of the Technical Advisory Panel. Mr. Roger Larson and Mr. Kurt Smith of Applied Pavement Technology, Inc. were the project researchers.

The start date for this contract was October 17, 2008. The Project Start-Up Meeting and initial panel teleconference was held on November 24, 2009, followed by an initial panel web conference on February 25, 2009. The revised *Interim Report* reflected comments provided by the panel at the initial teleconference as well as from the February 25, 2009 web conference. Based on the results of those teleconferences, the adopted revised evaluation plan addressed consideration of the following two issues:

1. Compare the performance and service life costs of 1.5-in fiber-reinforced polymer (FRP) and Type 304 solid stainless steel or concrete-filled pipes or tubes with epoxy coated mild steel for use in dowel bars on projects constructed in IA, IL, OH, and WI in 1996-1998.
2. Evaluate the performance of epoxy coated mild steel dowels on other projects that are 15-30 years or more old so the cost effectiveness of the more expensive alternative materials can be better evaluated.

These served as the primary objectives for the subject project.

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CHAPTER 1. INTRODUCTION

Background

Dowel bars are placed across transverse joints in jointed concrete pavements (JCP) to maintain vertical and horizontal alignment and to provide effective load transfer across those joints. Load transfer refers to the ability of a joint to transmit traffic loading from one slab to another, and is commonly defined as the deflection of the unloaded side of the joint as a percentage of the deflection of the loaded side of the joint (see Figure 1). The use of dowel bars strongly contributes to higher load transfer efficiency (LTE) values, and significantly reduces critical distresses such as pumping, faulting, and corner breaks.

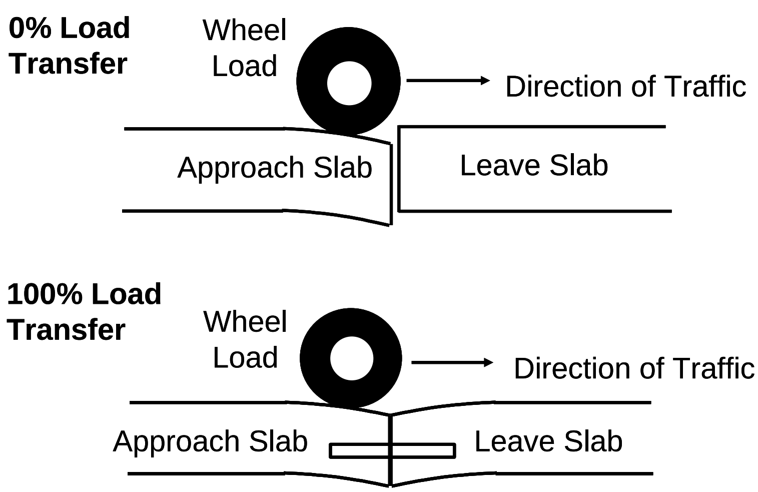

Figure 1. Illustration of deflection joint load transfer efficiency.

Most dowel bars used in highway pavement construction are smooth, round, solid steel bars conforming to ASTM A615 or AASHTO M31. These bars commonly have a fusion-bonded epoxy coating that provides corrosion protection by acting as a barrier against moisture and chloride intrusion. However, there have been some concerns in recent years regarding the long-term performance of epoxy-coated dowel bars and, as a result, the evaluation of alternative dowel bars has been investigated by a number of agencies. For example, the Highway Innovative Technology Evaluation Center (HITEC), which was established by the American Society of Civil Engineers (ASCE) and other stakeholders to facilitate the introduction of new or innovative materials and products into the highway market, initiated a project in the mid-1990s to evaluate a number of alternative dowel bars, namely fiber reinforced polymer (FRP) bars (sometimes referred to as glass fiber reinforced polymer [GFRP] bars), stainless steel bars and pipes, and conventional epoxy-coated dowel bars. Pavement projects incorporating these alternative dowel bars were constructed as early as 1996, and an evaluation plan was subsequently prepared that details procedures for evaluating the constructed projects (HITEC 1998). The principal thrust of that plan was on the observation and testing of field installations completed or planned by various state highway agencies (SHAs). The plan was later revised to emphasize the monitoring of the field performance over a longer period (5 years or longer) and to eliminate materials testing of full-length field samples after the initial 5-year evaluation period. There were a number of reasons for this change, including:

- No highway agency (except for the Ohio Department of Transportation [DOT] in 1983 and 1985 projects) has taken full-length field samples.
- There are few standard test protocols, particularly for the FRP materials.
- There is not a universally acceptable model that is capable of predicting expected performance from variations in the material properties obtained during testing.
- Previous coring of dowel specimens in Ohio and Iowa indicated little deterioration due to corrosion during the 5-year field evaluation period, making any significant findings unlikely. At that early age, socketing in the concrete around the dowel or delaminations in the concrete at the dowel bar level is more likely to be the important performance indicators.
- Significant test results are available from other sources to help characterize the range of materials properties of interest, including data from Ohio, Iowa, University of Manitoba, the University of West Virginia, the University of California at Davis, and the University of Sherbrooke, Quebec, Canada.

Although the HITEC program focused only on FRP dowel bars and stainless steel bars and pipes, there are a number of different types of alternative dowel bars, and several subsets within each type. Table 1 provides a listing of some of the alternative dowel bar types that have been installed on various highway pavement projects, along with advantages, disadvantages, and nominal cost information.

History of HITEC Evaluation Plan for Alternative Dowels

Under the HITEC program, initial field installations of FRP and stainless steel dowels began in 1996 in conjunction with the Federal Highway Administration's (FHWA's) High Performance Concrete Pavement (HPCP) program (also referred to Test and Evaluation Project 30 [TE-30]). At about the same time, a document titled *Preliminary Assessment of Alternative Materials for Concrete Highway Pavement Joints* was prepared (Porter and Braun 1997). That report consisted of a literature review and the results of a HITEC survey that included 36 responses from state highway agencies. The intent of that report was to provide HITEC with information to determine whether or not the use of alternative materials for concrete highway joints was worth a more thorough and rigorous evaluation. Both the Composites Institute and the Specialty Steel Industry of North America sponsored the original non-proprietary evaluation program. A Technical Evaluation Panel was established to help guide the evaluation effort.

A concurrent part of the field installation program was an evaluation of two older projects in Ohio featuring alternative dowel bars: 1) State Route 7 in Belmont County, constructed in 1983 with FRP dowels, and 2) I-77 in Guernsey County, constructed in 1985 with FRP and stainless steel dowels included in concrete joint repairs. In addition to condition surveys and deflection testing, cores and full-length dowels were cut from the Ohio pavements and used in additional laboratory evaluations. The results of this effort are documented in the report *Fiber-Reinforced Polymer (FRP) Composite Dowel Bars …a 15-year durability study* by the Composites Institute (MDA 1999). Also, RJD Industries, Inc. developed a 2-page summary *Long Term Field Performance of GFRP Pavement Dowels* and a report *FRP Dowel Bars, Analysis of Fiber Reinforced Polymer Dowels Removed From Active Roadways* (McCallion 1999).

Table 1. Summary of alternative dowel bar materials (Smith 2002b).

Material Type	Description	Advantages	Disadvantages	Nominal Cost
FRP Composite Bars	A solid bar made up of a composite material consisting of a matrix binder (such as polyester, vinyl ester, or epoxy), a reinforcing element (such as fiberglass or carbon fiber), and fillers.	• Not susceptible to corrosion • Durable • High tensile strength • Light weight /easy to handle • Closer in relative stiffness to PCC than steel bars, which reduces damage at dowel interface	• More expensive than epoxy-coated steel bars • Lower modulus of elasticity and shear strength than epoxy-coated steel bars • Low specific gravity (bar may float to surface during vibration if not secured)	• $6.61 to $8.81 per kg ($3 to $4 per lb) • $4 to $9 per dowel *(depends on diameter and material type)*
FRP Composite Tubes Filled with Cement Grout	An FRP composite tube filled with a high-strength cement grout for strength and deformation resistance.	• Not susceptible to corrosion • Durable • Less expensive than solid FRP composite bar • Closer in relative stiffness to PCC than steel bars, which reduces damage at dowel interface	• More expensive than epoxy-coated steel bars • Lower modulus of elasticity and shear strength than epoxy-coated steel bars	• $4 to $9 per dowel *(depends on diameter)*
Plastic-Coated Dowel Bars	A carbon steel bar containing a thin layer (about 0.5 mm [0.020 in]) of plastic coating, such as polyethylene.	• Corrosion resistance • Relatively moderate cost • Does not bond to PCC (may not require bond breaker coating) • Maintains low pull-out resistance	• Potential for damage during construction handling • Greater relative stiffness of bar compared to PCC may cause damage at dowel interface	• $3 to $6 per dowel *(depends on diameter)*
Solid Stainless Steel Bars	Low carbon steels (less than 1 percent) that contain at least 10.5 percent chromium by weight for corrosion resistance. Type 316 is commonly used for dowel bars.	• Strong corrosion resistance • Durable • High tensile strength • Long service lives (50–75 years) • Fully recyclable • No special handling requirements	• More expensive than epoxy-coated steel bars • More difficult to handle than FRP bars • Higher relative stiffness than FRP bars • Greater relative stiffness of bar compared to PCC may cause damage at dowel interface	• $4.40 to $5.28 per kg ($2 to $2.40 per lb) • $18 to $20 per dowel *(depends on diameter)*
Stainless Steel Clad Bars	Stainless steel cladding (commonly Type 316 and between about 1.8 to 2.3 mm [0.07 to 0.09 in] thick) metallurgically bonded to a conventional carbon steel core.	• Strong corrosion resistance • Durable • High tensile strength • Long service lives (50–75 years) • Cheaper than either FRP or solid stainless steel bars • No special handling requirements	• More expensive than epoxy-coated steel bars (but not as expensive as solid stainless steel bars) • More difficult to handle than FRP bars • Higher relative stiffness than FRP bars	• $1.10 to $1.65 per kg ($0.50 to $0.75 per lb) • $6 to $11 per dowel *(depends on diameter)*
Stainless Steel Tubes Filled with Cement Grout	A stainless steel tube filled with a high-strength cement grout for strength and deformation resistance.	• Strong corrosion resistance • Durable • High tensile strength • Long service lives (50–75 years) • Cheaper than either FRP or solid stainless steel bars • No special handling requirements	• More expensive than epoxy-coated steel bars (but not as expensive as solid stainless steel bars) • More difficult to handle than FRP bars • Higher relative stiffness than FRP bars	• $5 to $10 per dowel *(depends on diameter)*
Epoxy-Coated Steel Bars	A carbon steel bar containing a fusion-bonded epoxy coating (commonly between 0.2 to 0.3 mm [0.008 to 0.012 in] thick) which acts as a barrier system against moisture and chlorides.	• Resistance to corrosion • High tensile strength • Cheapest of all corrosion-resistant bars	• Long-term effectiveness of corrosion protection may be an issue • Coating can easily be nicked or scratched during construction handling • Greater relative stiffness of bar compared to PCC may cause damage at dowel interface	• $0.66 to 0.77 per kg ($0.30 to $0.35 per lb) • $2.50 to $5.00 per dowel *(depends on diameter)*

The final part of the field program was to be the removal and laboratory evaluation, at the conclusion of the 5-year observation period, of sample cores and full-length dowels from the alternative materials dowel joints placed as a part of this experiment. That part of the program is what has been undertaken in the current study under the Federal Highway Administration's (FHWA's) pooled fund program as project TPF-5(188). A revised and updated field evaluation and testing plan was presented in the interim report containing the approved change to eliminate the retrieval and testing of the full-length dowel samples (except as an allowable option) and instead focus on the field performance of the installations.

Problem Statement and Project Objectives

As described above, given the limitations of the original HITEC project, and the desire to evaluate the long-term performance of various alternative dowel bar materials, the subject project was established under the FHWA pooled-fund program. The focus of this project was narrowed to evaluate the long-term performance of 1.5-in (38-mm) diameter FRP bars; 1.5-in (38-mm) diameter, Type 304 stainless steel solid or clad bars; 1.5-in (38-mm) diameter, Type 304 stainless steel, concrete-filled tubes or pipe; and 1.5-in (38-mm) diameter, epoxy-coated mild steel smooth round dowels (as the control) based on over 10 years of service. In addition, it was also desired to assess the long-term performance and condition of conventional epoxy-coated dowel bars to determine the potential need or necessity for alternative dowel bars materials. Thus, the two major objectives of this pooled-fund project may be summarized as:

1. Evaluate the expected long-term performance of 1.5-in (38-mm) diameter FRP bars and 1.5-in (38-mm) Type 304 stainless steel solid or clad bars or concrete filled tubes and cost effectiveness of these materials as alternative dowel bar materials. The focus of this evaluation is limited to seven projects sites in four states.
2. Based on the evaluation of epoxy-coated mild steel smooth round dowels used as control and on FWD testing and coring of other existing projects after 15 to 30+ years of service, determine the expected service life on which to base the cost-effectiveness of the use of higher priced alternative materials.

Research Approach

The achievement of the goals outlined above required the collection of field performance data from a number of in-service concrete pavement projects (both the original alternative dowel bar projects listed in the HITEC evaluation plan and the older, epoxy-coated dowel bar projects introduced in this study). The overall intent was to determine how the pavements were performing (and by extension how the dowel bars were performing) in terms of load transfer efficiency, faulting, and roughness, as well as through coring of dowel bars to assess chloride contents at the depth of the dowel and overall dowel conditions. The participating SHAs assumed responsibility for conducting FWD testing, coring, and roughness testing on the original HITEC projects after 10 or more years of service. Moreover, participating SHAs were asked to select additional "older" projects (15 to 30^{+} years old) with epoxy-coated dowels to evaluate their overall conditions. The collected information could then be analyzed to help evaluate long-term performance and identify overall trends. However, due to funding constraints, only the Ohio and Wisconsin DOTs were able to perform the field testing outlined in this revised evaluation plan, which limits the results and recommendations that can be made.

Overview of Project Report

This report has been prepared to document the findings and information that have been collected to date under the original HITEC program and carried forward under the current pooled-fund study. The report consists of seven chapters (in addition to this one) and five supporting appendixes. Chapter 2 provides a review of literature on the use of alternative dowel bars, primarily in terms of their performance issues and overall applicability. Chapter 3 summarizes the construction and early performance of the alternative dowel bar projects included in the original HITEC program in Ohio, Iowa, Illinois, and Wisconsin. Chapter 4 presents the evaluation plan that was developed under the current project for the evaluation of the alternative dowel bar projects, including the proposed evaluation of older epoxy-coated dowel bar projects. Chapter 5 provides the recent performance evaluations of the alternative dowel bar projects in Ohio and Wisconsin, followed by Chapter 6 which presents the performance evaluations of older epoxy-coated dowel bars in those same states. Finally, Chapter 7 presents overall summary and conclusions, while Chapter 8 provides a recommended plan for implementing the results of the research.

Appendix A and Appendix B provide a summary of the field data collected on the alternative dowel bar projects in Ohio and Wisconsin, respectively. Appendix C presents photos of cores of alternative dowel bars in Ohio, along with FWD data collected on these projects over several years. Appendix D presents background summary information on key alternative dowel bar projects from Illinois, Iowa, Ohio, and Wisconsin as provided in an earlier-published FHWA report. Finally, Appendix E is a reproduction of the summary report documenting the field data collection activities and results conducted in Wisconsin.

CHAPTER 2. BRIEF ANNOTATED LITERATURE REVIEW

Introduction

This chapter provides a very brief summary of literature on the use of alternative dowel bars. It is not the intent of this chapter to provide an exhaustive look at all uses and experiences with alternative dowel bars, but rather to highlight some of the important past installations and current initiatives. A general overview is first presented in this section, followed by an in-depth look at a major FRP dowel bar study being conducted in Canada and a review of the use of alternative dowel bars in a rehabilitation setting. This is followed by a discussion on some of the performance issues associated with alternative dowel bars.

General Overview

Since the original project was established under HITEC in the mid-1990s, there have been a number of continuing studies on the use and application of alternative dowel bars. First, a number of studies continue to document the importance of dowel bars to the performance of jointed concrete pavements. For example, the report *Load Transfer Design and Benefits for Portland Cement Concrete Pavements* (ERES 1996) provides information on the history and benefits of dowel bar load transfer in jointed concrete pavements. The beneficial effect of dowels is also documented in the report *Key Findings from LTPP Analysis 2000-2003* (FHWA 2004). Data from the LTPP program clearly demonstrate that dowels significantly reduce roughness due to faulting and significantly increase the transverse joint load transfer efficiency.

One major effort in the use of alternative dowel bars was conducted under FHWA's High Performance Concrete Pavement Program, which was launched in 1996 as a way of exploring innovative design and construction concepts. The program features over 16 dowel bar-related projects constructed under a range of design variables, and those projects have been documented in various FHWA reports (Smith 2002a; QES 2004; FHWA 2006). Portions of the 2004 document that describe projects included in this pooled-fund program are included in Appendix D for information.

There are also a number of other research studies, either recently completed or ongoing, being performed on the use of alternative dowel bars at a number of venues, including Iowa State University (Cable, Porter, and Guinn 2003; Porter 2009), the University of Manitoba (Murison 2004; Murison, Shalaby, and Mufti 2005), the University of California at Davis (Bian 2003; Mancio et al. 2007; Bian, Kohler, and Harvey 2007), West Virginia University (Li 2004; Gupta 2004; Vijay, GangaRao, and Li 2009), and University of Sherbrooke (Benmokrane 2011; Montaigu, Robert and Benmokrane 2011). Additionally, there have also been a number of accelerated load testing studies of alternative dowel bar size, spacing, and materials that can provide additional insight into expected performance. For example, a study using the Heavy Vehicle Simulator (HVS) was completed in California (Bian, Harvey, and Ali 2008), and an accelerated testing study in Kansas (Melhem 1999) is also available. Two reports evaluating alternative materials for retrofit dowel applications were published by the University of Minnesota (Odden, Snyder, and Schultz 2003; Popehn, Schultz, and Snyder 2003), and a study using the Minne-ALF to evaluate Type 316 stainless steel Schedule 40 unfilled structural pipe (1.66-in [42-mm] outside diameter and 0.14-in [3.6 mm] wall thickness) was also conducted (Yut et al. 2005).

Since its earliest use on two projects in Ohio in the mid-1980s, FRP dowel bars have been installed by a number of other states, including Illinois, Wisconsin, Iowa, Minnesota, and Kansas, among others. In addition, the Ohio DOT constructed a test project in 2005 featuring epoxy-coated dowels, MMFX dowels (3 joints), zinc-plated dowels (3 joints), and FRP dowels on a section of U.S. 30 in Wayne County; the one FRP joint exhibited LTE less than 30 percent by 2009 (Kim et al. 2010). Recently, the Sigma Development Group produced a material specification overview for their MateenDowel bar that is modeled after AASHTO requirements. Their epoxy back boned vinyl ester resin and E-CR Glass product is being evaluated in a number of states and was recently installed on a 16 mile (26 km) roadway on I-84 in Boise, Idaho. There are several other manufacturers that also produce FRP dowel bars (for example, Hughes Brothers currently produces the Aslan 600 FRP dowel bar and RJD Industries produces the FiberDowel™ FRP dowel bar).

In addition to alternative dowel bar materials, some research work has been conducted in the area of alternative dowel bar shapes and configurations. For example, the Iowa DOT has constructed several concrete pavement projects featuring elliptical dowel bars, both steel and FRP (Cable, Porter, and Guinn 2003; Cable, Totman, and Pierson 2006). Preliminary performance data for these elliptical dowels is promising. In addition, the use of plate dowels (which have had widespread use in warehouse floor applications) are beginning to see some use in low-volume roads and streets (ACPA 2010). A summary of available research on alternative dowel bars is available from the Wisconsin Department of Transportation (CTC 2007).

Summary of University of Sherbrooke FRP Research

The University of Sherbrooke undertook this study to characterize and assess the performance of FRP dowels as an alternative to epoxy-coated mild steel bars currently used by the Ministry of Transport of Quebec (MTQ) (Benmokrane 2011). The characterization of the FRP dowels was achieved through mechanical and physical testing trials to assess the sustainability of products beyond the expected service life, as well as through structural tests on reduced scale jointed plain concrete pavement (JPCP) slabs to validate the proposed design method. The tests are performed on FRP dowels manufactured by Pultrall Inc., and include both vinyl ester and polyester resins with 80+ percent Type E glass (Montaigu, Robert and Benmokrane 2011).

The first phase of the research helped characterize the mechanical and physical properties of dowels needed by the manufacturer, with the MTQ providing the necessary design data. It was apparent from this phase that the dowels met the only structural requirement of 22,000 lbf/in^2 (150 MPa) in direct shear strength (values from 22,000 to 29,000 lbf/in^2 [150 to 200 MPa] were obtained).

For all the mechanical tests, dowels of polyester resin resulted in strengths 20 to 30 percent less than that of vinyl ester, with poorer performance at the joint interface. Manufactured with vinyl ester resin, the dowels have high durability physical properties (D1 test given by the CSA S807 code). Apart from the rate of water absorption, dowels of polyester meet the criterion of sustainability D2 of polyester resins.

The second phase of the project evaluated the sustainability of the dowels under different conditions simulating the pavement service conditions. The 300 cycles of freezing and thawing affected the performance of the interface (shear interlayer) in the order of 15 percent for the polyester resin dowels, whereas the vinyl ester resin dowels were not affected by the test. Chemical resistance tests have shown that only an extremely alkaline environment with high diffusivity of the hydroxide ions could lead to hydrolysis of the matrix, causing a degradation of the interface and the loss of significant physical properties.

No effect of diameter was found on mechanical losses incurred. Accelerated aging tests proved the good long-term integrity of vinyl ester dowels that exhibit stability of more than 90 percent for a period of service extrapolated to 200 years. Physical properties are equivalent to the end of the period of aging. The polyester resin dowels suffered losses of mechanical and physical properties of 60 percent bending integrity and 80 percent direct shear at the end of a period of 30 years of service.

Finally, the structural component of the study developed a method of design and assessment of the structural performance of JPCP slabs with epoxy-coated steel dowels and FRP dowels. The results obtained after 1 million cycles of loading/unloading with 11,200 lb (50 kN) help to ensure the structural performance of JPCP with FRP dowels. The 1.37-in (34.9-mm) diameter FRP dowels provide performance equal to those of 1.13-in (28.6-mm) diameter steel dowels and limit development beyond the regulatory load cracks. Thus, based on the equivalence of developed stresses in the concrete around the dowel, the following alternatives were proposed for field applications (Benmokrane 2011):

- 1.13-in (28.6 mm) diameter steel dowel bars = > 1.37-in (34.9-mm) FRP bars.
- 1.25-in (31.8 mm) diameter steel dowel bars = > 1.50-in (38.1-mm) FRP bars.
- 1.37-in (34.9 mm) diameter steel dowel bars = > 1.63-in (41.3-mm) FRP bars.

It should be noted that these alternatives are proposed for a given set of design parameters, and would require some modification if different design parameters are selected.

Based on the results of the MTQ research, the following recommendations were provided (Benmokrane 2011):

1. It is recommended that the MTQ consider the widespread use of FRP dowels of vinyl ester resin in concrete pavement roads and highways of the province.
2. While the specimens of pavement slabs tested resulted in excellent results under conditions of long-term service evaluated and remained non-cracked, a continuation of this research project should be undertaken to optimize the design of JPCP slabs using new FRP vinyl ester-based dowels. In particular, the study of parameters such as the type of soil, concrete, the spacing and diameter of the dowels, and the thickness of the slab would develop new correction factors or equations to design optimized and efficient JPCP slabs of concrete pavements using FRP dowels.

In sum, the Ministry of Transport of Quebec believes that the use of these new FRP vinyl ester-based dowels and the optimization of the design will allow the development of a new generation of safe and economical jointed concrete pavements with increased sustainability.

Rehabilitation Applications of Alternative Dowel Bars

The focus of most of the studies on alternative dowel bars has been on new concrete pavement construction. However, some of the accelerated testing research has been performed on rehabilitated sections including load transfer restoration by dowel bar retrofit. Of particular note, the original 1985 Ohio sections included an evaluation of dowel specimens from full-depth patches. Also, one of the original FRP dowelled joints (removed for evaluation) in the 1983 SR 7 (Belmont County) project was replaced in 1998 with a full-depth patch using 1.5-in (38-mm) diameter polyester resin and E-glass dowels placed on 12-in (305-mm) spacings.

In other studies, Caltrans investigated FRP dowels and stainless steel pipes in retrofit applications under HVS loading, and noted that the vertical deflections for steel dowel bars in a four-dowel-per-wheelpath configuration were less than either the FRP or stainless steel pipes in a similar configuration (Bian, Harvey, and Ali 2008). Similarly, the Minnesota DOT looked at FRP dowel bars and stainless steel pipes in an accelerated loading situation (Odden, Snyder, and Schultz 2003). The results of the study indicated that the FRP dowel bars exhibited lower LTE than similarly sized epoxy-coated bars, while the stainless steel pipes exhibited similar (but slightly lower) LTE values than epoxy-coated bars. The LTE of the stainless steel pipes was later noted to drop off rapidly after about 10 million load cycles (Odden, Snyder, and Schultz 2003). Increasing the diameter of the FRP dowel bars from 1.50 in (38 mm) to 1.75 in (44 mm) resulted in LTE values and differential deflections most closely matching those obtained from grouted stainless steel tubes (Popehn, Schultz, and Snyder 2003).

Performance Issues of Alternative and Conventional Dowel Bars

FRP Dowel Bars

Regarding the use of FRP dowel bars, the major performance issue identified so far relates to the significantly lower LTE values of the 1.5-in (38-mm) diameter FRP dowels after only a few years and under relatively low accumulated equivalent single-axle load (ESAL) applications. This statement is based on the performance of the FRP dowels compared to alternative materials at the same locations during falling weight deflectometer (FWD) testing in the spring or fall of the year when the joints are not locked up (Smith 2002a; Smith 2002b). As expected for the short performance period being evaluated, all the pavements sections were reported to be generally in very good condition at the end of the 5-year evaluation period. An excerpt from the 5-year evaluation report on the Wisconsin projects notes that (Crovetti 2006):

> *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 indicated reduced load transfer capacities and increased IRI 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 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.*

Laboratory test results and the results of limited field evaluations raise concern about the long-term performance of these FRP materials. There appears to be a need for a consensus on what is considered acceptable load transfer performance for both the short term (5- to 10-year evaluation period) and the long term (30 years or longer).

Recent laboratory testing results bear out this concern about the long-term performance capabilities of FRP dowels. For example, research at Iowa State University showed lower load transfer efficiencies for 1.5-in (38-mm) diameter FRP dowels, with the recommendation for increasing dowel size or decreasing dowel spacing to improve LTE (Cable and Porter 2003). Research from the West Virginia University provides considerable information on these options based on a combination of laboratory testing and field evaluation studies (Li 2004; Vijay, GangaRao, and Li 2009). Similarly, a study by the University of Manitoba also looked at larger FRP tubes (2- or 2.5-in [50 or 64-mm] diameter) filled with mortar due to concerns about the performance of 1.5-in (38-mm) solid FRP dowels (including lower load transfer efficiencies and higher bearing stresses in the concrete at the joint face than the 1.5-in [38-mm] diameter epoxy-coated steel dowel used as a control) (Murison 2004; Murison, Shalaby, and Mufti 2004). Moreover, laboratory work performed by the University of Minnesota suggests that 2-in (51-mm) diameter FRP dowels are expected to have similar performance as 1.5-in (38-mm) epoxy-coated steel dowels (Odden, Snyder, and Schultz 2003). Also, the University of Minnesota researchers concluded that the differential deflection at the joint (maximum of 5 mils [0.005 in, or 0.13 mm] under a 9,000-lb [40-kN] load), in addition to load transfer efficiency, is an important failure criterion. It was also recommended that the partial failure criterion of 70 percent or less LTE be tightened to 85 percent or less to allow for more useful early comparisons between the details being evaluated (Popehn, Schultz, and Snyder 2003). Caution is necessary when evaluating load transfer efficiencies if the maximum deflection is very low so this factor also needs to be considered. Conversely, if the maximum deflection is very high (10 mils [0.25 mm] or higher, under a 9,000-lb [40-kN] load), it indicates poor base/subbase/subgrade support, which can be a significant problem on some projects (particularly those with unstabilized permeable bases).

In 1997, the Ohio DOT constructed a project on the eastbound lanes of U.S. 50 near Athens, Ohio, that features various types of alternative dowel bars. That project, sometimes referred to as the OH 2 project or the ATH-50 project, has been the subject of extensive testing and evaluation since its construction. FWD data are available from 1997, 1999, 2001, 2003, 2004, 2005, 2006, and 2008 (see Appendix A), with the load transfer data from 2006 and later now showing LTE values less than 40 percent for the three different types of polyester resin FRP dowels. However, 4-in (102-mm) diameter cores taken of the FRP dowel bar materials in November 2004 showed the FRP bars to be in good condition, whereas the cores of the epoxy-coated dowels showed some corrosion on the epoxy-coated bars. Appendix C contains photos of the cores taken from the U.S. 50 project.

This raises some additional questions about the long-term effectiveness of the epoxy-coated steel bars. HIPERPAV II may be helpful in evaluating the early age stresses on the OH 2 project, stresses that may have contributed to the delaminations in the concrete near the dowel bars. This updated version of the model used earlier information from the instrumented dowels on the OH 2 project to evaluate the expected short-term performance of jointed concrete pavement. However, it is likely that the poor support from the New Jersey unstabilized permeable base on the OH 2 project is a major cause of the distress in the concrete near the more rigid epoxy-coated steel dowels (and on concrete-filled Type 304L stainless steel tubes or pipe as well). A Michigan research report *Qualify Transverse Cracking in PCC from Loss of Slab-Base Contact* evaluates this factor in more detail (Hansen, Peng, and Smiley 2004). At the same time, an ongoing multi-state joint deterioration study is currently underway (pooled-fund study TPF-5(224)) that suggests that poor concrete quality may be a significant issue as well.

Coring was also conducted on a concrete pavement project in Iowa that is evaluating different types of alternative dowel bars (U.S. 65 near Des Moines, constructed in 1997). The 4-in (102-mm) diameter cores of the FRP dowels showed no distress, but no cores were taken of the Type 316 solid stainless steel dowels (Cable and Porter 2003). To facilitate identifying the location of the FRP bars, researchers taped a nail to the FRP dowel bar during construction. The minimum load transfer efficiency of all dowels (including FRP) exceeded 79 percent in Iowa, which is higher than reported on projects in the three other states. Additional research in Iowa is underway to evaluate elliptical FRP and elliptical epoxy-coated steel dowels (Cable, Totman, and Pierson 2006; Porter 2009).

Absorptivity of the FRP composite material is another concern. Several research studies (at the University of California, Davis [Bian 2003; Mancio et al. 2007] and at the West Virginia University [Gupta 2004; Vijay, GangaRao, and Li, 2009]) have evaluated this issue. The research at the University of Sherbrooke (Benmokrane 2011; Montaigu, Robert, and Benmokrane 2011) recommends only vinyl ester resin due to the poor performance of polyester resin caused by moisture absorption. The MateenDowel Material Spec Overview has suggested minimum material specifications for FRP dowels to help ensure product performance and longevity.

Conventional Epoxy-Coated Dowel Bars

One of the key questions regarding the use of conventional epoxy-coated dowel bars is whether corrosion is compromising their long-term performance. Unfortunately, there are very limited data available documenting the extent of the problem. Nevertheless, the interest in the use of alternative dowel bar suggests that there is at least the perception of a significant problem. A December 27, 2005 survey of use of epoxy-coated dowels by the Kentucky Department of Highways resulted in 33 responses in which (KTC 2005):

- Twenty-six of thirty-three respondents reported the use of epoxy-coated smooth dowels.
- Thirteen of the thirty-three respondents indicated that they had recently performed dowel bar excavations; seven of those agencies reported the dowel bars to be in good condition, five reported the dowels to be corroding, and one did not know the condition of the dowels.
- Six of the thirty-three respondents reported rusting problems with epoxy-coated dowels while twenty-one did not.

Until better nationwide data are available, each state will have to evaluate their concrete pavement performance to determine if dowel corrosion is a significant issue and if so, whether or not the use of alternative dowel materials is cost-effective for their specific design conditions (traffic, climate, deicing applications, and so on). The need for long-lasting, durable dowel bars becomes particularly acute as more agencies adopt long-life concrete pavement designs for high-volume roadways (FHWA 2007).

Load Transfer Efficiencies of Other Dowel Materials

It should be noted that reviews of monitoring data from other HPCP projects raise similar concerns about low LTE values for a number of different dowel bar materials. For example, in an experimental project in Michigan (I-75, Detroit, also referred to as the MI 1 project), both the European section (variably spaced 1.25-in [32-mm], plastic-coated dowels) and the control section (1.25-in [32-mm] epoxy-coated mild steel dowels) exhibited LTEs less than 70 percent (Weinfurter, Smiley, and Till 1994; Buch, Lyles, and Becker 2000). Similarly, an experimental project in Kansas (Highway K-96, Haven, also referred to as the KS 1 project) has a number of epoxy-coated steel dowel sections exhibiting LTE values of 70 percent (Wojakowski 1998). Further, an analysis of LTPP data indicated several projects with 1.5-in (38-mm) epoxy-coated dowel bars exhibiting LTE values of 40 percent or less (FHWA 2004). Potential reasons for these low LTE values could be socketing of the dowel bars due to poor consolidation, high initial curling/warping, poor support, and/or heavy overloads. Another cause of the poor joint performance could be the early deterioration of the concrete at the joints, an issue that is currently being evaluated under TPF-5(224).

Summary

This chapter briefly summarizes a review of the literature regarding the performance of alternative dowel bars. The purpose of this review is to present some of the current experiences and highlight some of the issues that have arisen regarding the use of alternative dowel bars. Taken as a whole, this information helps provide valuable background information into the work that was performed under this study.

CHAPTER 3. ALTERNATIVE DOWEL BAR PROJECTS INCLUDED IN ORIGINAL HITEC PROGRAM

This chapter describes the projects and early performance of the alternative dowel bar installations included in the original HITEC program. These projects feature the use of 1.5-in (38-mm) diameter FRP bars, 1.5-in (38-mm) diameter Type 304 solid stainless steel bars, 1.5-in (38-mm) diameter Type 304 stainless steel clad bars and tubes, and conventional 1.5-in (38-mm) diameter epoxy-coated dowel bars, and are located in Ohio, Iowa, Illinois, and Wisconsin. Table 2 summarizes all HPCP projects incorporating alternative dowel bars, with additional information provided in Appendix D and elsewhere (Smith 2002a; QES 2004; FHWA 2006). Updated performance reports on the experimental sections on U.S. 50 in Ohio (Ohio 2) and on State Route 29 in Wisconsin (WI 2) are presented in Chapter 5.

Ohio

US 50, Athens Project

In 1997-1998, the Ohio Department of Transportation constructed three high performance concrete pavement projects, all located on U.S. 50 near Athens. Common to the pavement test sections was a 10-in (254-mm) thick JRCP design (0.14 percent steel) and 21-ft (6.4-m) transverse joint spacing. One of the projects evaluates the use of alternative dowel bars, including conventional epoxy-coated steel dowel bars, type 304 stainless steel tubes filled with cement grout, and FRP composite dowel bars, all of which are located in the eastbound lanes. Several of these dowel bars were instrumented to allow investigation of dowel response under a variety of loading and environmental conditions and to compare the measured responses of different types of dowel bars, but the stainless steel tubes were not instrumented because the thin tube thickness did not permit the machining of a flat surface to attach lead wires (Sargand 2001). In 1998, Type 316 stainless steel clad dowels were included in the adjacent westbound roadway.

The instrumented dowels were monitored under both environmental and dynamic loading for the first few months after paving. An analysis of the strains in the FRP composite and conventional epoxy-coated steel bars revealed the following (Sargand 2001):

- Environmental forces (thermal curling and/or moisture warping) produced greater bending moments in both the steel and FRP composite dowel bars than dynamic loading forces. The dynamic bending stresses induced by a 12,800-lb (56.8 kN) load were considerably less than the environmental bending stresses induced by a 5.4 ^{o}F (3 ^{o}C) temperature gradient.
- Significant stresses were induced by the steel dowel bars early in the life of this pavement as it cured late in the construction season under minimal temperature and thermal gradients in the slab. PCC pavements paved in the summer under more severe conditions may reveal even larger environmental stresses.
- Steel dowel bars induced greater environmental bending moments than FRP bars.
- Both types of dowel bars induced a permanent bending moment in the PCC slabs during curing, the magnitude of which is a function of bar stiffness.
- Curling and warping during the first few days after PCC placement can result in large bearing stresses being applied to the PCC around the dowels. This stress may exceed the strength of the concrete at that early age and result in socketing around the bars.

Table 2. FHWA HPCP projects evaluating alternative dowel bar materials (Smith 2002a).

Project/ Location	Date Built	Type of Load Transfer Devices	Dowel Diameter
Illinois 1 I-55 SB, Williamsville	1996	Epoxy-coated dowels	38 mm (1.5 in)
		FRP composite dowels (*RJD Industries, Inc.*)	38 mm (1.5 in)
Illinois 2 Route 59, Naperville	1997	Epoxy-coated dowels	38 mm (1.5 in)
		FRP composite dowels (*RJD Industries, Inc.*)	38 mm (1.5 in) 44 mm (1.75 in)
		FRP composite dowels (*Corrosion Proof Products, Inc.*)	38 mm (1.5 in)
		FRP composite dowels (*Glasforms, Inc.*)	38 mm (1.5 in)
Illinois 3 U.S. 67 WB, Jacksonville	1999	Epoxy-coated dowels	38 mm (1.5 in)
		FRP composite dowels (*RJD Industries, Inc.*)	38 mm (1.5 in)
		FRP composite dowels (*Strongwell Corporation*)	38 mm (1.5 in)
		FRP composite dowels (*Creative Pultrusions, Inc.*)	38 mm (1.5 in)
		FRP composite tubes filled with cement grout (*Concrete Systems, Inc.*)	51 mm (2 in)
		Type 316L stainless steel clad dowels (*Stelax Industries, Inc.*)	38 mm (1.5 in)
Illinois 4 Route 2 NB, Dixon	2000	FRP composite tubes filled with cement grout (*Concrete Systems, Inc.*)	51 mm (2 in)
		Type 316L stainless steel tubes filled with cement grout	38 mm (1.5 in) 44 mm (1.75 in)
		Type 316L stainless steel clad dowels (*Stelax Industries, Inc.*)	38 mm (1.5 in) 44 mm (1.75 in)
Iowa 2 U.S. Route 65, Des Moines	1997	Epoxy-coated dowels	38 mm (1.5 in)
		FRP composite dowels (*Hughes Brothers, Inc.*) (203- and 305-mm [8- and 12-in] spacings)	48 mm (1.88 in)
		FRP composite dowels (*RJD Industries, Inc.*) (203- and 305-mm [8- and 12-in] spacings)	38 mm (1.5 in)
		Solid stainless steel dowels (203- and 305-mm [8- and 12-in] spacings)	38 mm (1.5 in)
Kansas 1 K-96, Haven	1997	Epoxy-coated dowels	32 mm (1.25 in)
		FRP composite tubes filled with cement grout (*Concrete Systems, Inc.*)	51 mm (2 in)
		X-Flex™ Device (*Kansas State University*)	—
Michigan 1 I-75, Detroit	1993	Plastic-coated dowels	32 mm (1.25 in)
		Epoxy-coated dowels	32 mm (1.25 in)
Minnesota 1 I-35W, Richfield	2000	Epoxy-coated dowels	38 mm (1.5 in)
		Type 316L stainless steel clad dowels (*Stelax Industries, Inc.*)	38 mm (1.5 in) 44 mm (1.75 in)
		Type 316 solid stainless steel dowels (various manufacturers)	38 mm (1.5 in)
		Plastic-coated dowels (PCC shoulders only)	38 mm (1.5 in)
Minnesota 2 Mn/Road Low Volume Road Facility, Albertville	2000	Epoxy-coated dowels	25 mm (1.0 in) 32 mm (1.25 in)
		FRP composite dowels	32 mm (1.25 in) 38 mm (1.5 in)
Ohio 2 U.S. Route 50, Athens	1997/1998	Epoxy-coated dowels	38 mm (1.5 in)
		FRP composite dowels (*RJD Industries, Inc.*)	38 mm (1.5 in)
		Stainless steel (type 304) tubes filled with cement grout	38 mm (1.5 in)
Wisconsin 2 WI 29, Owen	1997	Epoxy-coated dowels (5 layout configurations)	38 mm (1.5 in)
		FRP composite dowels (*RJD Industries, Inc.*)	38 mm (1.5 in)
		FRP composite dowels (*Creative Pultrusions, Inc.*)	38 mm (1.5 in)
		FRP composite dowels (*Glasforms, Inc.*)	38 mm (1.5 in)
		Type 304L solid stainless steel dowels (*Avesta Sheffield, Inc.*) (2 layout configurations)	38 mm (1.5 in)
		Type 304L stainless steel tubes filled with cement grout (*Damascus Bishop Tube Company*)	38 mm (1.5 in)
Wisconsin 3 WI 29, Hatley	1997	Epoxy-coated dowels (2 configurations)	38 mm (1.5 in)
		FRP composite dowels (*Strongwell Corporation*)	38 mm (1.5 in)
		FRP composite dowels (*Glasforms, Inc.*)	38 mm (1.5 in)
		FRP composite dowels (*Creative Pultrusions, Inc.*)	38 mm (1.5 in)
		FRP composite dowels (*RJD Industries, Inc.*)	38 mm (1.5 in)
		Type 304L solid stainless steel dowels (*Slater Steels, Inc.*)	38 mm (1.5 in)

It was also noted that steel dowel bars transferred greater dynamic bending moments and vertical shear stresses across transverse joints than FRP composite bars of the same size.

The load transfer efficiencies on the OH 2 project from 2001 are shown in Figure 2. The Type 318 stainless steel clad dowels installed in 1998 on US 50 westbound were not included in this evaluation.

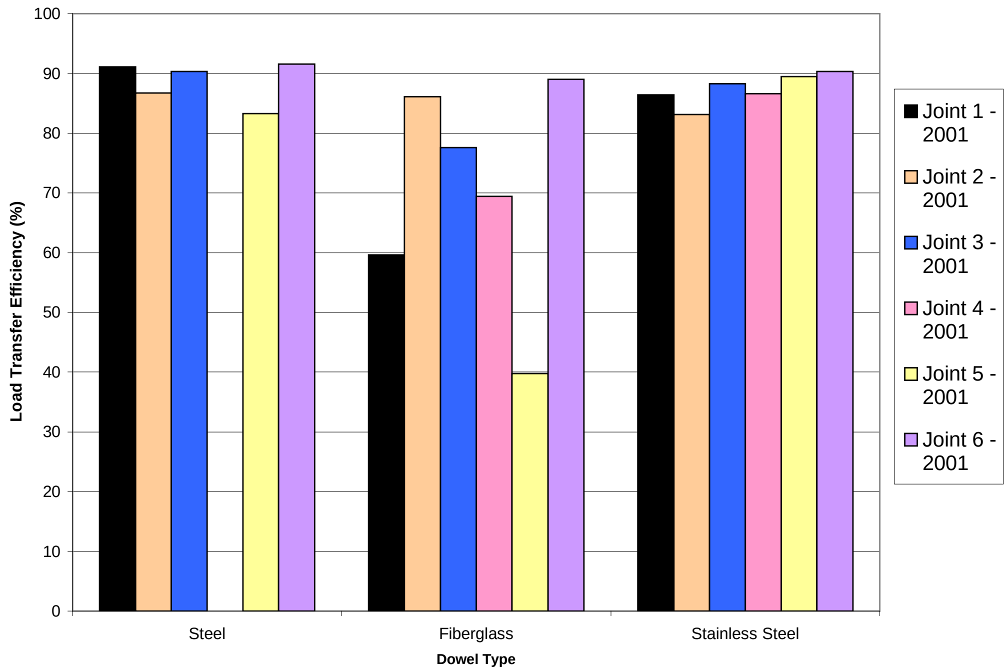

Figure 2. LTE measurements for OH 2 project.

Additional FWD testing and coring was performed on this project in 2004. The 4-in (102-mm) diameter cores (selected photos of which are included in Appendix C) of the epoxy-coated dowels and of the concrete-filled Type 304 stainless steel showed delaminations of the concrete at the dowel bar level. As corrosion of the stainless steel dowel at this age is unlikely, the cause of the cracking is most likely due to the high early environmental stresses noted during construction and/or the poor support provided by the New Jersey unstabilized permeable base combined with the more rigid steel dowel bar properties.

HITEC Evaluation of Older Ohio Experimental Projects

Results from evaluation of removed field samples (Part 2 of HITEC's 1998 Evaluation Plan) are available in the Composites Institute Report (MDA 1999). A review of the Dynaflect deflection data showed LTEs in the 40s for both the epoxy-coated and FRP dowels during cooler weather (McCallion 1999). The good performance of the joints despite the low LTE values will require additional investigation to determine the reason for this apparent discrepancy. The different deflection equipment (FWD and Dynaflect), different testing temperatures, different testing procedures (location of the load plate, number of drops, whether load history information was gathered on the last drop), and different analysis procedures significantly confound the testing results. However, all recent deflection testing of research sections has been with the FWD.

Iowa

The Iowa Department of Transportation and Iowa State University have conducted a significant amount of dowel bar research including the evaluation of alternative materials (Porter and Guinn 2002; Cable and Porter 2003). The following summaries and conclusions have been reached based on the data collected during one field study evaluating alternative dowel bars (Cable and Porter 2003):

- All dowel materials tested are performing equally in terms of load transfer, joint movement, and faulting over the 5-year evaluation period.
- Stainless steel dowels do provide load transfer performance equal to or greater than epoxy-coated dowels in this study on the average over 5 years.
- FRP dowels of the sizes tested in this research should be spaced no greater than 8-in (203-mm) spacings to gain load transfer performance at the same level as epoxy-coated steel dowels at 12-in (305-mm) spacing.
- No deterioration due to road deicers was found on any of the dowel materials retrieved in the 2002 coring operation. (note: the Type 316L solid stainless steel dowels were not cored).

The following items should be considered for future research in the area of alternative dowel materials (Cable and McDaniel 1998):

- Future research is needed on the methods of securing FRP dowels into basket assemblies for construction.
- Efforts must be made to reduce the cost of FRP and stainless steel solid dowels to make them cost competitive with epoxy-coated steel dowels if they are to be included in highway work.
- Laboratory work in the area of consideration of shape, spacing, and chemical composition of the FRP dowels is essential for future specification development.

Additionally, it was noted that the FRP tie bars tended to “float” during their insertion. It appears there would be a similar problem with FRP dowels if a dowel bar inserter were used. However, this was not reported to be a problem in Wisconsin. Also, the problem of locating FRP or stainless steel dowels (in baskets or with an inserter) needs to be evaluated.

In the Iowa field demonstration study, the FRP dowels exhibited 79 percent LTE compared to 84 percent with the solid stainless steel or 90 percent with the epoxy-coated mild steel (Cable and McDaniel 1998; Cable and Porter 2003). Still, Cable and McDaniel (1998) conclude that “From the test data it appears that a longer period of time (10 to 20 years) would be necessary to draw any conclusions on the relative performance of the material types.”

Iowa State University prepared a report, *Assessment of Dowel Bar Research,* which summarizes major dowel projects and investigations since 1990 (Porter and Guinn 2002). This document identifies critical gaps in the current knowledge base and advocates the development of universal testing procedures for both laboratory and field evaluations of dowel bars so that consistent, meaningful comparisons can be made.

Iowa has also performed significant research on the use of elliptical FRP and epoxy-coated mild steel dowels (Cable, Porter, and Guinn 2003; Cable, Totman, and Pierson 2006). A comprehensive listing of Iowa's research into alternative dowel bars is available (Porter 2009).

Illinois

Illinois has four projects evaluating the use of alternative dowel bars (some in conjunction with sealed or unsealed joints). The oldest was built in 1996 on a weigh station ramp on I-55 near Williamsville; it was soon followed by a project on Route 59 near Naperville in 1997, a project on U.S. 67 near Jacksonville in 1999, and a project on Route 2 in Dixon in 2000 (Gawedzinski 1997; Gawedzinski 2000; Gawedzinski 2004). Dowel bar types evaluated in the various projects include FRP composite dowels, cement grout-filled FRP tubes, type 316L stainless steel clad dowels, type 316 stainless steel tubes filled with cement grout, and conventional epoxy-coated dowel bars.

The Illinois DOT has been regularly monitoring the performance of these sections, including the measurement of load transfer efficiencies. Test sites are monitored with an FWD on a monthly, semi-annual, or annual basis, depending upon test schedules. After up to 4 years of service, all of these sections were performing well (Gawedzinski 2000). In general, the LTE values for the sections containing FRP dowels are lower and more variable than those for those sections containing conventional epoxy-coated steel dowel bars.

The 1996 project, IL 1, included 1.5-in (38-mm) diameter, FRP dowels in four contraction joints on an entrance ramp to I-55 from a truck weigh station. At an age of 7.5 years and over 10.1 million ESALs the joints show little damage or distress (Gawedzinski 2004). However, initial testing in 1998 showed all FRP dowels with less than 75 percent LTEs. A bituminous aggregate mixture subbase (BAM) was used.

The 1997 project, IL 2, consisted of five different FRP sections and the epoxy-coated dowel bar control section. A plot of the LTE measurements is provided in Figure 3. This shows that all five FRP sections had LTEs less than 85 percent soon after construction. Overall performance of the FRP joints (range 65 to 80 percent LTE after 6 years and 1.3 million ESALs) appears to be very close to the behavior of the epoxy-coated steel control set (minimum of 83 percent LTE after 6 years). This project had a granular subbase.

One construction issue that arose on the IL 2 project was that the fiber composite bars were loose and only partially attached to the upper support wire of the basket (Gawedzinski 1997). A special metal spring clip was devised to secure the dowel bars to the basket so they did not move when the PCC was placed.

The 1999 project, IL 3, consisted of five alternative dowel sections (three different solid 1.5-in [38-mm] diameter FRP composite dowels, one FRP tube filled with hydraulic cement grout, and one Type 316 stainless steel clad dowel) and two epoxy-coated steel dowel control sections, one with sealed joints and the other with unsealed joints. This project had a cement aggregate mixture subbase (CAM2 with a minimum cement content of 200 lbs/yd^3 [119 kg/m^3]). The control section with epoxy-coated dowels, the epoxy-coated dowel section with unsealed joints, the stainless steel clad carbon steel dowel section, and the fibrillated wound fiber composite bars exhibited better load transfer and lower joint deflections than the pultruded fiber composite bars.

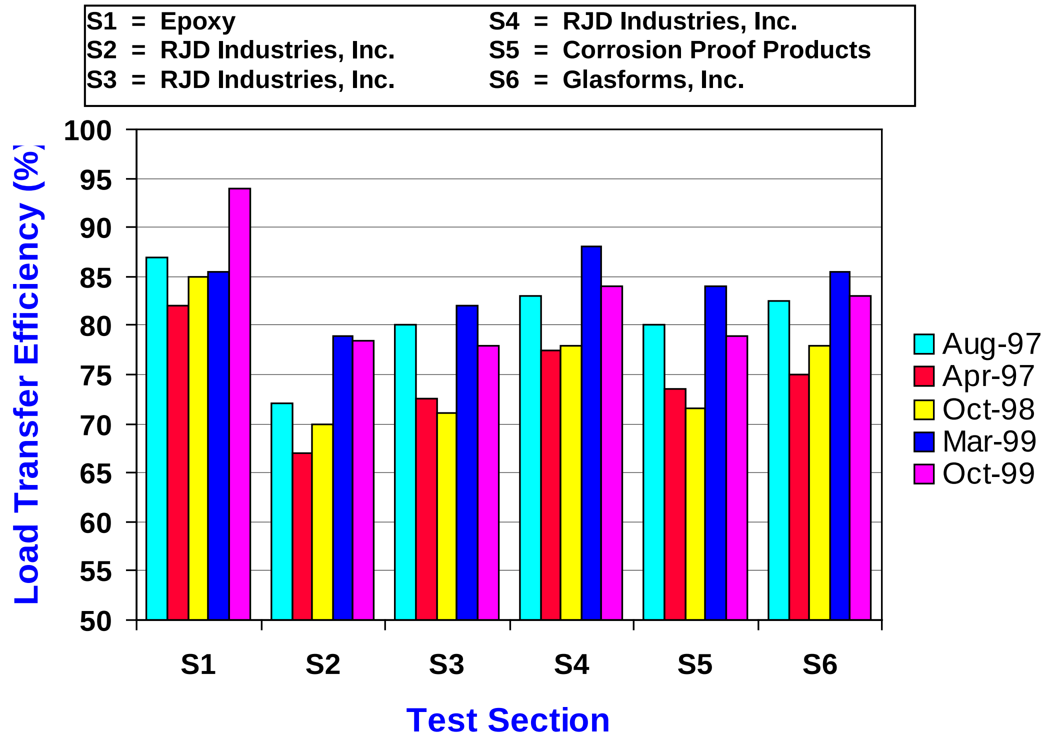

Figure 3. LTE measurements on IL 2 project (Gawedzinski 2000).

The 2000 project, IL 4, included stainless steel tubes filled with cement grout, Type 316L stainless steel clad carbon steel tubes, and fiber composite tubes filled with cement grout. Two different diameters, 1.5 and 1.75 in (38 and 44 mm), were used for the stainless steel tubes and for the stainless steel clad dowels. The fiber composite tubes were formed using a pultrusion process and had a diameter of 2 in (51 mm). The pultrusion process produced a much smoother bar, compared to the first generation, fibrillated bars. All joints were unsealed. On this project all test sections had LTEs greater than 85 percent in 2003 after only about 130,000 ESALs.

Presently all four test sites appear to be performing well, without any signs of spalling, faulting, or other pavement distress. It is too soon to tell what effect the generally lower LTEs on the FRP composite dowel sections will have on long-term performance.

Unfortunately, due to manpower limitations and traffic control/safety concerns, the IL 2 project located on IL 59 near Naperville will no longer be evaluated with FWD. In order to gather all nine testing locations (including the outer wheelpath, inner wheelpath, and center of the lane), two of the three lanes had to be closed for testing, which is no longer possible given the urban location and high traffic volumes.

Wisconsin

The Wisconsin DOT constructed three experimental PCC projects under FHWA's HPCP program, two in the summer of 1997 and one in the summer of 2002. The two older projects (both located on Highway 29, one between Owen and Abbotsford and one between Hatley and Wittenberg) were constructed to evaluate the use of alternative dowel bars, alternative dowel bar spacings, and variable pavement cross sections (Crovetti 1999). The common pavement design was an 11-in (279-mm) thick JPCP with skewed transverse joints variably spaced at 17-20-18-

19-ft (5.2-6.1-5.5-5.8-m) intervals. The dowel bars included in the study are standard epoxy-coated steel dowel bars, type 304L solid stainless steel dowel bars, FRP composite dowel bars, and type 304L stainless steel tubes filled with cement grout. All were placed in standard dowel configurations with 12-in (305-mm) spacings with the exception of some of the solid stainless steel dowel bars, which were placed in configurations clustering three and four dowel bars in the wheelpath of the outer lane (Crovetti 1999).

These sections were performing well after only a few years of service. FWD testing of transverse joint load transfer has been conducted on the projects, with the results for the outer lane wheelpaths of WI 2 and WI 3 shown in Figures 4 and 5. Generally speaking, the late season tests (October 1997 and November 1998) indicate significantly reduced LTE for the FRP composite dowels, although the LTE measurements in the summer do not indicate any significant differences within the test sections, probably because of the increased aggregate interlock brought about by the closing of the joints due to the warmer temperatures (Crovetti 1999; Smith 2002b). The use of impact echo testing to determine dowel bar locations on WI 2 was inconclusive for the solid stainless steel dowels and the Type 304L stainless steel tubes filled with cement grout.

The more recent HPCP project (WI 4) was constructed in September 2002 on I-90 near Tomah with a design life of 50-years (QES 2004). Three test sections were constructed, one with a tied concrete shoulder and epoxy-coated dowels, one with a tied concrete shoulder and Type 316L solid stainless steel dowels, and one with an asphalt shoulder and epoxy-coated dowel bars. The section with the solid stainless steel dowel bars also used stainless steel tie bars to tie the concrete shoulder. One problem noted during the construction of these sections was the flexibility of the baskets made with 0.125-in (3.2-mm) diameter wire, and as a result 0.19-in (4.8-mm) diameter wire will be specified on future projects. The DOT is monitoring the performance of these test sections but has not yet reported any preliminary performance data.

The performance evaluation of WI 2 and WI 3 reported by Crovetti (2006) included the results of laboratory testing, joint deflection tests, and dowel bar pull-out tests (AASHTO T 253-76). A summary of the average transverse joint load transfer based on FWD testing revealed the following:

- 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 joint load transfer for the FRP, HF and epoxy coated steel dowels was 69 percent, 78 percent, and 88 percent, 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 percent and from 76 to 79 percent, respectively.

Deflection test results are strongly dependent upon the season of the year and temperature gradients causing downward curling during field testing. The negative effects are more pronounced as the stiffness of the subgrade layer increases.

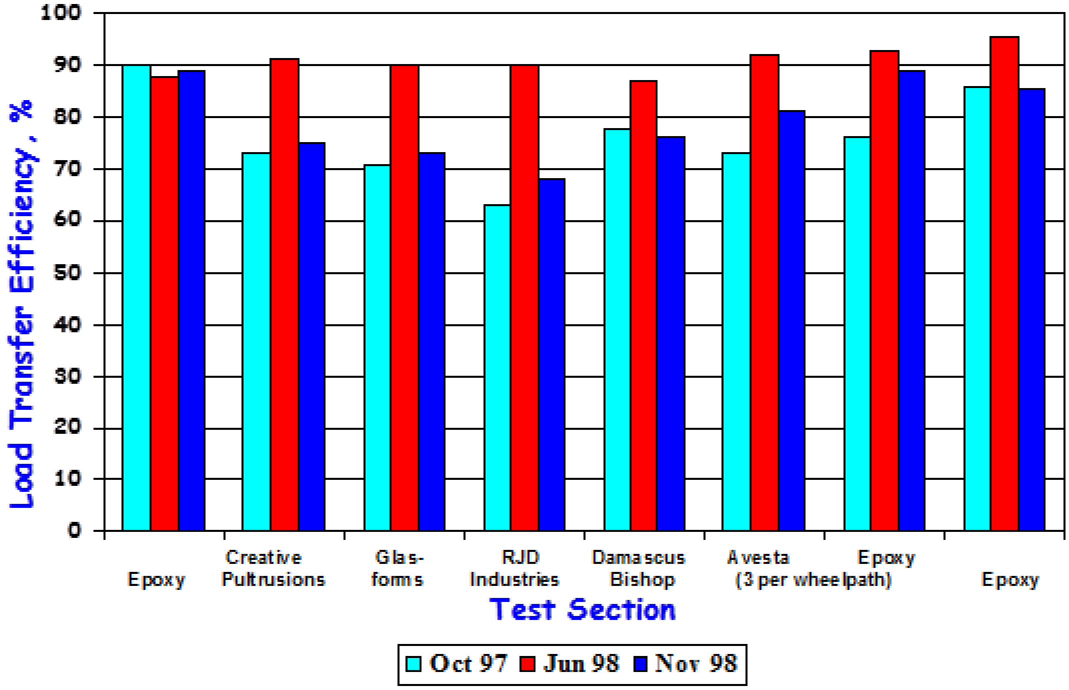

Figure 4. LTE measurements for WI 2 project (Smith 2002b).

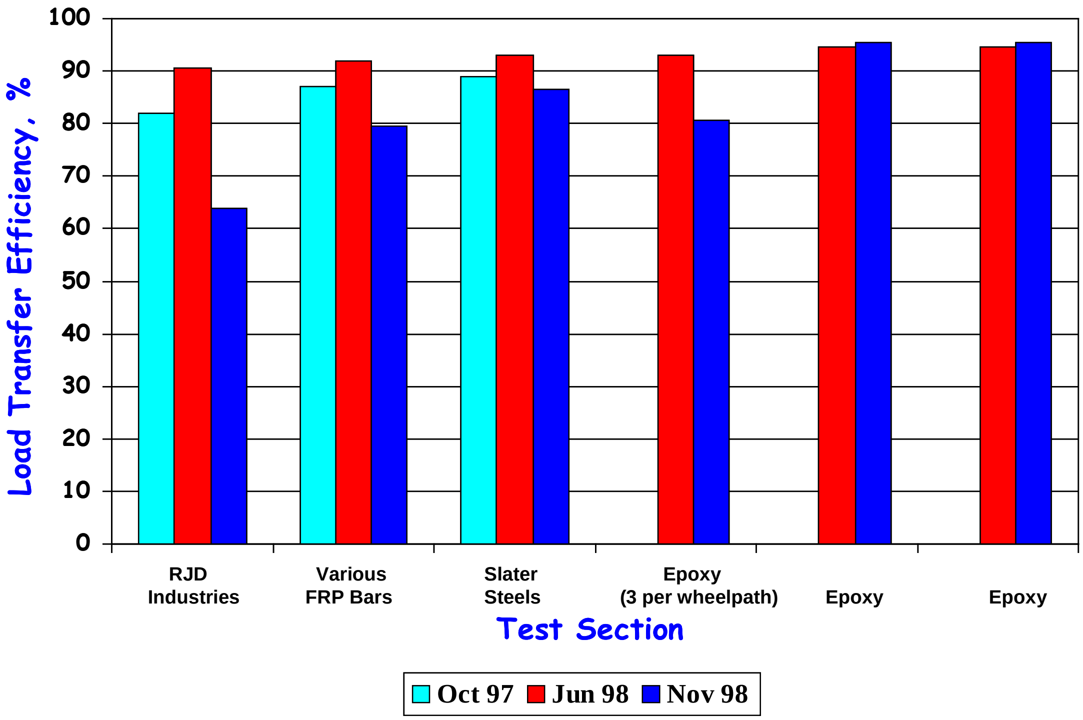

Figure 5. LTE measurements for WI 3 project (Smith 2002b).

CHAPTER 4. REVISED EVALUATION PLAN FOR TPF-5(188) (APRIL 15, 2009)

Introduction

This chapter presents the revised evaluation plan for the alternative dowel bar projects included under the TPF-5(188) pooled-fund study. This plan, which was originally submitted in an interim report in April 2009 and approved by the project advisory panel (APTech 2009), provides the framework for the data collection and analysis activities that were conducted under the project.

Objective

As previously indicated, the main objective of this effort is to evaluate the performance of 1.5-in (38-mm) diameter, 18-in 457-mm) long (all on 12-in [305-mm] centers) FRP composite and Type 304 stainless steel solid dowels or concrete-filled tubes compared to that of conventional, epoxy-coated steel dowels (used as the control) after at least 10 years of service. Recommendations for the use of alternative dowel bars will be made based on this study and other related research findings.

To help evaluate the cost effectiveness of these newer materials, a secondary objective is to document the performance of eight to twelve projects (minimum of two projects and a maximum of three projects in each age category) in each state where the epoxy-coated steel dowels (used in this effort as the control material) have been subjected to 15 to 30^+ years of deicing materials and traffic. It is hoped that observations on cores of the epoxy-coated steel dowels removed from these older projects would help verify the extent of the corrosion problem and help justify the use of more expensive alternative dowel bars in order to minimize the problem, particularly in long-life JPCP designs.

Proposed Evaluation of Original HITEC Projects

To complete the 10-year performance evaluation of the subject projects, the following field testing is proposed:

FWD Deflection Testing, Dowel Bar Removal, Chloride Analysis, and Roughness

Testing should be conducted on projects IL 3, IA 2, OH 2, and WI 2 and WI 3. Due to the limited number of joints, it is recommended that only FWD testing be conducted on IL 1. Traffic volumes will not allow FWD testing and coring on IL 2; instead it is recommended that consideration be given to evaluating the roughness of the joints with the various FRP materials using a high-speed profilometer similar to the 2004 evaluation of WI 2 and WI 3 (Crovetti 2006). It is recommended that FWD testing also be conducted on SR 7 (constructed in 1983) and on I-77 (constructed in 1987) in Ohio, which are the oldest known projects containing FRP dowel bars but were not previously included in the HITEC program. Based on the higher pavement roughness at the FRP joints on the WI 2 and WI 3 projects, it is recommended that roughness be evaluated for the different material types on all the joints being evaluated as part of the HITEC project continuation. It is suggested that the ProVAL software be used for this analysis.

FWD Deflection Testing

It is recommended that the SHA's continue their deflection testing studies in both 2009 and 2010 to complete this 10-year evaluation of performance. In addition, it is recommended that deflection/load history data on the last drop be stored in an Access database so they can be exported into Excel to facilitate the production of graphs and charts. It is especially important that LTE, differential joint deflection, and total joint deflection data be available for both approach and leave slab positions and particularly on the joints where the dowels will be cored or removed for evaluation.

Coring of Alternative Dowel Bar Materials

It is recommended that 6-in (152-mm) diameter cores be taken through the dowels at the joints containing alternative dowel bar materials that are the subject of this study (that is, only 1.5-in [38-mm] diameter, 18-in [457-mm] long dowels on 12-in [305-mm] spacings consisting of FRP composite, Type 304 stainless steel [solid or tube], or epoxy-coated steel). Cores of other materials, sizes, or spacings may be retrieved at the option of the respective SHA. Note: The removal of three full-length dowels of each type at one joint for each project for additional laboratory testing as recommended in the May 1998 HITEC Evaluation Plan is not considered warranted at this time; however, it is allowed as an option if the SHA elects to do so (see the alternative testing procedures section below).

Number of Cores

For projects where no more than two FRP composite materials are used, it is recommended that cores be taken at the first and eighth dowel (as measured from the outside shoulder edge of the outer traffic lane) for two joints each for the FRP composite dowels, the Type 304 solid stainless steel dowels or Type 304 concrete filled stainless steel tubes, and the epoxy-coated steel dowel control joints. If a defect is noted during the coring of the joint, it is recommended that an additional 6-in (152-mm) core be taken at the sixth dowel in the joint. If three or more types of FRP composite dowels are used on the project, only one joint (2 or 3 cores) of each FRP composite dowel need to be sampled. If only one joint is sampled preference should be given to joints with lower LTEs and/or higher differential slab deflections. If two joints are selected, one joint with the lower LTE and/or higher differential slab deflections and one with average LTE and differential joint deflection should be cored.

As a minimum, one joint per material type (except two joints for the epoxy-coated steel dowel used as the control) should be taken with cores at the first (outer) dowel and the eighth dowel and a core taken at the sixth dowel only if some defect was noted in the first or eighth dowel core. This would reduce the impact on performance of the dowels removed while still having samples of the dowel material to verify conditions. For this minimum sampling option, it is suggested a joint with the lowest load transfer efficiency and/or highest differential slab deflection be taken.

Testing of Cores

The cores taken should be photographed and given a detailed visual examination for signs of defects (e.g., socketing around dowel, corrosion of dowel at concrete/dowel interface, abrasion of the dowel surface at the crack face, and so on). The core hole should be visibly examined and any defect of the dowel/concrete slab interface or base material noted. The cores should be tagged and wrapped before transporting to the laboratory.

In the laboratory, the cores should be split at the joint and the dowel specimens removed for visual inspection and photographing. No lab testing of any dowel specimens is anticipated at this time. However, for the cores of Type 304 stainless steel dowels or tubes and of epoxy-coated steel dowels, the concrete in the core face above the dowel should be sampled for chloride content as per ASTM C 1152, *Test Method for Acid-Soluble Residue in Mortar and Concrete* or AASHTO T 260. The purpose of the test is to 1) determine the chloride content in the concrete at the dowel bar level, and 2) relate any occurrence of corrosion with the chloride content. For 6-in (152-mm) diameter cores, one sample could be taken on both sides within 1.5 in (38 mm) of the joint crack and one sample from the 1.5 in (38 mm) of concrete on the outer edges of the core. Chloride testing is not necessary on cores with FRP composite dowels or Type 316 stainless steel dowels. Data on the amount and type of deicing chemicals used should be obtained, if available. Note: It is not expected that there will be significant corrosion of either the Type 304 stainless steel dowel or tube or the epoxy-coated steel dowel within the 10-year evaluation period. However, it is considered necessary to verify the chloride content to predict future risk of corrosion and to verify that there are not any signs of significant corrosion within this minimum 10-year evaluation period.

Alternate Testing Procedures

Alternate testing procedures conforming to their standard practices may be used by highways agencies, but similar procedures should be used to evaluate the HITEC projects and the 15-year and older epoxy-coated dowel bar projects. A full-length bar could be removed by making 2-in (51-mm) cores on the end of each dowel to just below the dowel, saw cutting at edges of core holes below level of dowel, and using a jackhammer to remove the concrete and the dowel bar. If the full length dowel is removed, a new dowel bar would be inserted on a chair and the hole patched similar to the dowel bar retrofit process. This technique has the advantage of restoring full load transfer at the joint.

Chloride ion testing could also be done in a similar fashion as for bridge decks. A hollow stem carbide bit and vacuum system is used to collect concrete at 0.75-in (19 mm) intervals. The concrete dust is collected on a filter paper and the filter paper is treated with silver nitrate titrate to determine the chloride content in lb/ft^3. This is essentially a nondestructive test procedure. Corrosion potential of the dowel bars could also be measured using the method outlined in NCHRP Synthesis 57 (NCHRP 1979) using a potentiometer half-cell and copper sulfide probe. This could be accomplished in conjunction with the chloride sampling. Several of the bars at a joint could be tested with minimally invasive practices, i.e., a hole to attach a probe on the dowel. Note: The purpose of this test is to help evaluate the potential corrosion on the epoxy coated dowels and Type 304 stainless steel solid dowels or tubes. If available, the amount and type of deicing salts applied would be valuable.

Evaluation of Epoxy-Coated Dowel Projects after 15 Years or More of Traffic

As a means of determining the extent of the dowel bar corrosion problem, it is proposed that each state pull cores (or full length dowels) from older projects (15 to 30^+ years) and assess their overall condition; this will help determine if more corrosion resistant dowels are warranted and, ultimately, whether they are cost-effective. It is currently recommended that epoxy-coated steel dowels remain the standard corrosion protection for routine projects (other than long-life pavements). To ensure cost-effective performance (compared to black steel dowels which should not be used), the current performance of epoxy-coated steel dowels must be evaluated and documented. There currently is very limited documentation of the long-term performance of epoxy-coated dowels on regular construction projects.

It is recommended that each state select a minimum of 2 (maximum of 3) projects with epoxy-coated dowels in each of the following age groups:

- 15-19 years.
- 20-24 years.
- 25-29 years.
- 30 years or more.

Candidate projects for each of these categories can include HMA overlays of existing jointed concrete pavements. Removal of the dowels and chloride testing should be performed in the same manner as that used to evaluate the 10-year performance of the HITEC projects.

It is recommended that one epoxy-coated steel dowel at a minimum of three and maximum of five consecutive joints (to minimize effect on joint load transfer capability at each joint sampled) at two randomly determined locations on each project be selected for taking 6-in (152-mm) diameter cores. The cores would be visually examined and the dowels removed and examined as recommended above. The concrete face above the dowels would be sampled (two locations) in accordance with ASTM C 1152 or AASHTO T 260 as noted above, any corrosion of the dowels noted and the dowels photographed for future reference. It would be desirable, but not required, to have FWD data on the dowels selected for testing. For this evaluation:

Total Number of Cores	=	2 locations/project *x* minimum 3 cores/location *x* 2 projects/age category *x* 4 categories
	=	48 cores minimum
Total Number of Chloride Tests	=	2 locations/project *x* 3 cores/location *x* 2 samples per core *x* 2 projects/age category *x* 4 categories
	=	96 ASTM C 1152 chloride tests recommended

It is estimated that the ASTM C 1152 testing costs would be $80 to $120 per core or using $100 per core average *x* 96 cores = $9,600, which is in addition to the coring and traffic control costs. Each SHA would have to fund the testing as funds are not currently available from the pooled funds project. This is considered to be a very reasonable cost to help verify whether or not corrosion resistant dowels are currently being provided so that full-depth repairs to joints caused by dowel bar corrosion in the future are avoided/minimized or that more corrosion resistant dowels are warranted.

Summary

This chapter presents recommended evaluation procedures for the field investigation of alternative dowel bars. It is based on the original HITEC 1998 evaluation plan, but has been modified to collect what are considered to be more meaningful data. An additional component has been added, in which agencies are encouraged to evaluate their existing epoxy-coated dowel practices to determine the extent that dowel corrosion may be affecting pavement performance.

CHAPTER 5. RECENT FIELD EVALUATIONS OF ALTERNATIVE DOWEL BARS IN OHIO AND WISCONSIN

Introduction

Following the field evaluation plan presented in Chapter 4, in 2010 and 2011 the Ohio and Wisconsin DOTs performed additional testing on the their projects featuring 1.5-in (38 mm) diameter FRP composite, stainless steel, and epoxy-coated dowel bars. This field testing included primarily the collection of additional FWD data, coring analysis of dowel bars, chloride content testing at the depth of the dowel bars, and overall roughness and faulting measurements. The availability of this data from these two states does limit the extent of the evaluation that can be made, but some overall trends are still apparent. Moreover, it should be noted that several of the cores (particularly in Wisconsin) showed what appeared to be very poor quality concrete, which may have affected the overall joint performance. The concrete quality issue is being evaluated under a separate pooled-fund study, TPF-5(224).

In addition, several other alternative dowel bar projects were evaluated in Ohio that were not part of the original HITEC evaluation plan. This includes the two older projects from the mid-1980s that contain FRP bars, as well as several older projects that feature plastic-coated dowel bars.

Evaluation of Ohio 2 Project (U.S. 50, Athens)

General information on this project was provided previously in Chapter 3, with more detailed information presented in Appendix D. The Ohio DOT has performed regular testing of the test sections on this roadway since their construction in 1997-1998. Table A-1 (Appendix A) presents the tabularized historical FWD data results taken on the OH 2 project from 1997 to 2010, with an overall summary of the load transfer results (broken out by approach slab and leave slab) provided in Table 3. For the OH 2 experimental sections, only one joint failed by having a normalized deflection greater than 10 mils (0.25 mm), even though the support from the unstabilized New Jersey permeable base was low and there were some problems with concrete durability at the joints.

A summary of the evaluation of the different types of dowel bars included in the project is provided below.

Evaluation of Polyester FRP Dowels

The summary data provided in Table 3 are presented graphically in Figures 6a and 6b. These figures clearly show that the polyester resin FRP dowels evaluated in the OH 2 project exhibit far lower LTE values than their counterparts. The three types of polyester resin FRP dowels (two types furnished by RJD Industries and one type furnished by CEF, Inc.) all have LTE values averaging less than 40 percent since 2006. The most recent data indicate that LTE values obtained on 10/30/09 averaged 31.3 percent on the approach joint and 28.5 percent on the leave joint, whereas the LTE values measured on 11/03/10 averaged 33.1 percent on the approach joint and 39.9 percent on the leave joint. In general, a deflection-based LTE of 70 percent or less represents less than 50 percent stress load transfer and it is considered a critical threshold level.

Table 3. Summary of historical approach and leave LTE averages for OH 2.

	Epoxy-Coated Dowels (Control 1)		**Grout Filled Stainless Steel (Type 304) Tubes**		**FRP Dowels**		**Epoxy Coated Dowel (Control 2)**	
	LTE	Max Deflection, mils*	LTE	Max Deflection, mils*	LTE	Max Deflection, mils*	LTE	Max Deflection, mils*
Approach Averages								
11/6/1997	87.1	3.6	80.6	4.0	71.9	4.9		
11/15/1999	94.5	14.0	90.2	15.3	74.3	15.3		
8/2/2001			88.6	5.0	70.3	7.7	87.4	4.9
12/8/2003	89.5	5.7	87.4	5.0	74.8	5.9	62.7	8.8
5/24/2004	90.3	4.1			83.7	4.0	76.8	5.0
1/13/2005	84.4	6.1	76.5	4.2	75.8	5.0		
11/7/2006	88.1	6.5	84.5	8.6	30.8	12.5		
10/29/2008	91.3	7.0	78.6	8.61	28.3	10.1		
10/30/2009	92.6	5.0	80.5	5.1	31.3	7.0		
11/3/2010	95.4	9.5	96.9	14.9	33.1	15.0	92.1	6.5
Leave Averages								
11/6/1997	89.8	3.5	77.2	4.0	70.2	5.1		
11/15/1999	98.12	13.6	96.5	15.1	85.6	14.3		
8/2/2001			90.7	4.8	80.3	6.8	90.8	4.5
12/8/2003	87.7	5.8	84.1	5.2	75.4	6.5	63.3	9.3
5/24/2004	87.4	4.1			81.4	4.0	72.0	5.4
1/13/2005	88.4	5.1	86.0	2.6	83.4	2.8		
11/7/2006	90.9	6.2	88.0	8.3	35.2	12.3		
10/29/2008	90.7	7.0	87.9	8.1	32.1	10.5		
10/30/2009	89.4	5.0	80.8	5.1	28.5	7.5		
11/3/2010	95.5	9.4	94.9	14.8	39.9	15.3	93.3	6.7

*Normalized to 9,000 lb (40 kN) load.

The poor performance of the polyester resin FRP dowels might be related to its increased moisture adsorption capacity, as noted previously. In addition, there are other factors that may also be contributing to the performance of these sections, including the poor stability of the unstabilized New Jersey permeable base and the poor quality of the concrete (cores revealed some deterioration at the depth of the dowel bar).

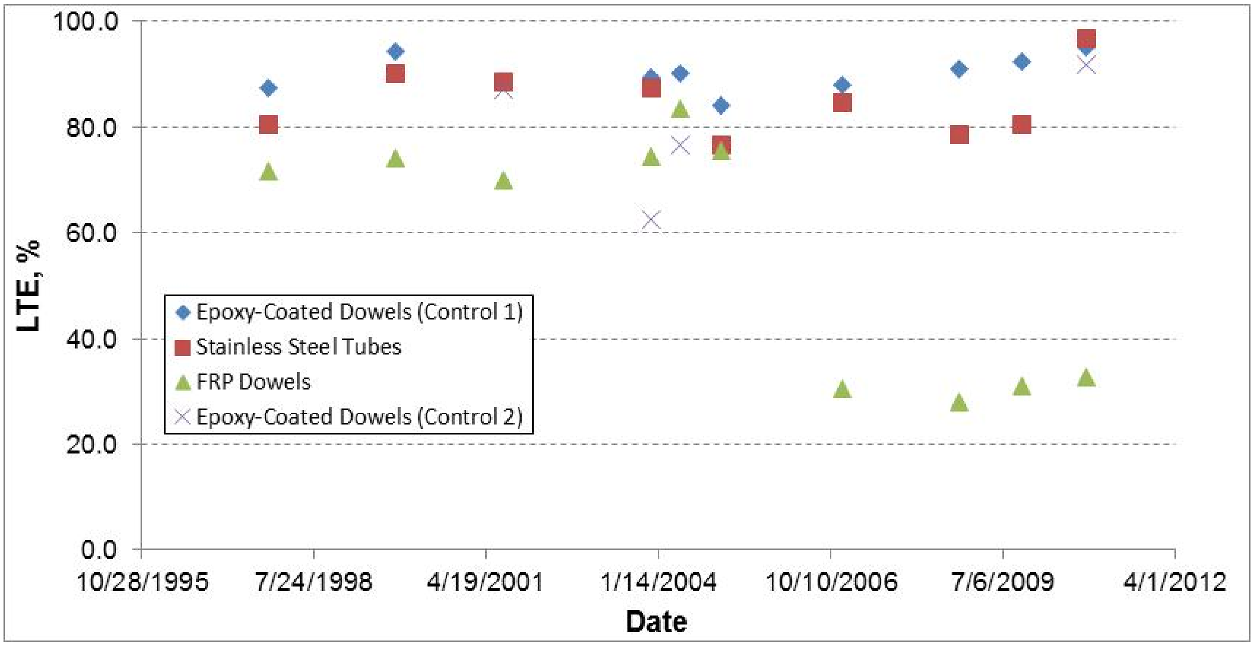

Figure 6a. Summary of historical load transfer efficiency data for OH 2 EB (approach joint).

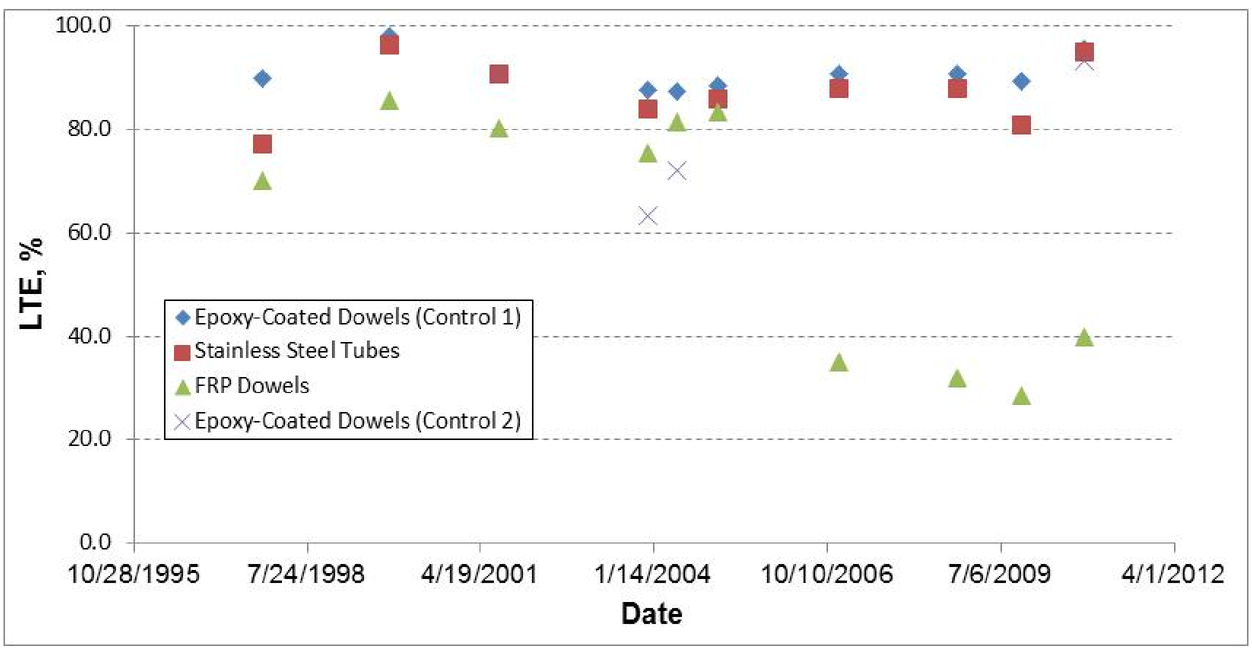

Figure 6b. Summary of historical load transfer efficiency data for OH 2 EB (leave joint).

Even though the sections with FRP bars are not performing well (as evidenced by the extremely low LTEs), the sections with the other two dowel bar types are still exhibiting satisfactory LTEs after nearly 14 years of service. The Type 304 stainless steel tubes (mortar filled) average 78.6 percent on the approach joint and 87.9 percent on the leave joint in 2008. Similarly, the epoxy-coated mild steel dowels used as a control averaged 91.3 percent on the approach joint and 90.7 percent on the leave joint. The 1.5-in (38-mm) FRP polyester resin dowels appear to be much more susceptible to poor support (concrete quality and/or base/subbase/subgrade) than the conventional and alternative steel dowels. This suggests that for poor support conditions, vinyl ester resin FRP dowels of diameter greater than 1.5-in (38-mm) will be required. The percentage and type of glass fibers is also a critical design variable in addition to the resin type.

The three different types of polyester resin FRP bars used on this project (two manufactured by RJD Industries and one by CEF, Inc.) exhibited similar LTE trends over the 16-year monitoring period (see Figure 7). All FRP dowels maintained relatively good levels of LTE until 2006, at which time the LTE values dropped off considerably. The CEF bars are exhibiting slightly higher LTE values than the RJD varieties.

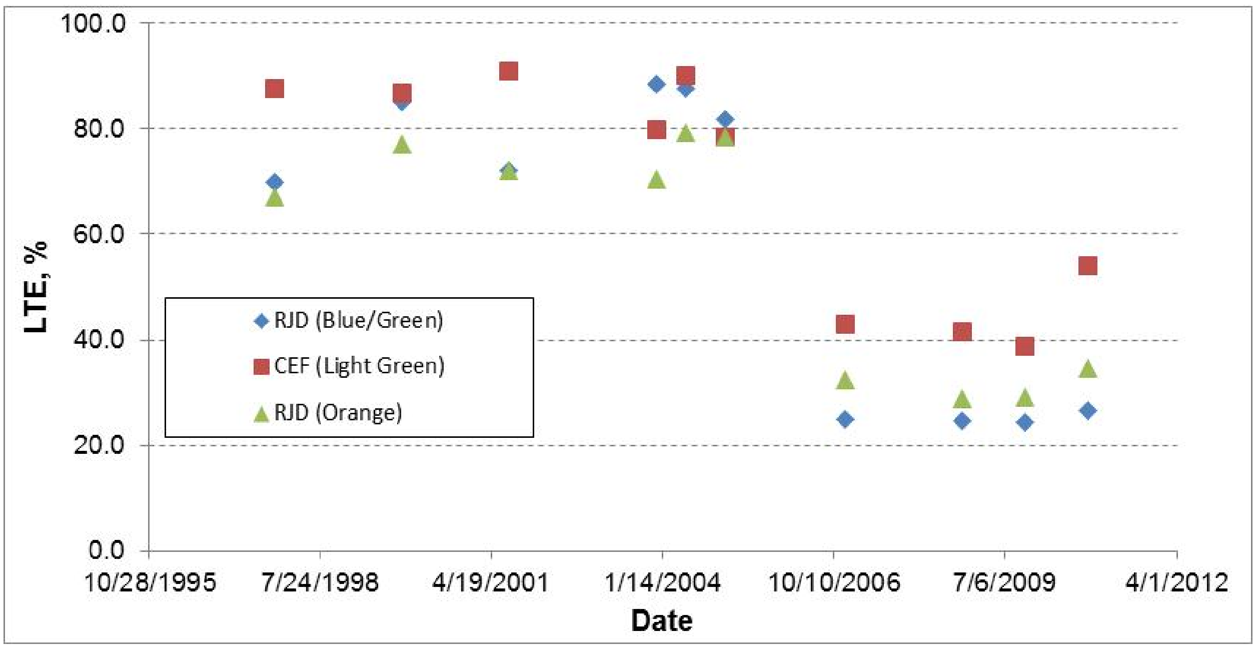

Figure 7. Average LTE (approach and leave) for FRP bars on OH 2.

Evaluation of Type 304 Stainless Steel Tubes (Mortar filled)

As shown in Table A-1 (Appendix A), the LTE values for the sections containing these dowels are slightly less than 80 percent on the approach joint and around 85 percent on the leave joint for the years prior to 2009. This raises concerns about their long-term performance as joints with LTEs of 70 percent or less are candidates for load transfer restoration (dowel bar retrofit). However, FWD testing in 2009 or 2010 shows approach LTE values in the 97 to 98 percent range, and leave LTE values in the 95 to 97 percent range, both of which are considered excellent. There was some rust staining at the joint but no deterioration of the dowel surface. The average joint LTE values from 1997-2010 for the Type 304 stainless steel tubes are shown in Figures 6a and 6b.

Evaluation of Epoxy-Coated Mild Steel Dowel Bars (Control)

Table A-1 (Appendix A) shows that the LTE values for the conventional epoxy-coated dowel bars have all averaged more than 84 percent from 1997-2010, with most over 90 percent. These data are presented graphically in Figures 6a and 6b. However, cores of the epoxy-coated dowel bars revealed that, in most cases, the epoxy coating could be easily removed, exposing an area of rusted steel; however, at this time (after approximately 14 years of service), there was no significant loss of dowel cross section.

Evaluation of Stainless (Type 316) Steel Clad Dowels (ATH50 Westbound)

Stainless steel clad dowels were installed at eight joints in the westbound lanes of the U.S 50 project; these were placed in 1998 (the dowels were not received in time to be included in the 1997 construction of the eastbound lanes). These dowel bars were tested for deflection and then cored. Table A-2 (Appendix A) summarizes the deflection data on the sections constructed in the westbound lanes containing stainless steel clad dowel bars and conventional epoxy-coated dowel bars. At one joint, the measured LTE values averaged 88 percent on the approach joint and 95 percent on the leave joint. At the second joint, the LTEs on the approach averaged 40 percent and the leave slab 65 percent due to a gap around the bar on the approach side assumed to be due to poor consolidation of the concrete. Minor pitting of the stainless steel was observed at the joint face.

Coring and Chloride Analysis of OH 2 Sections

Table C-1 in Appendix C provides a summary of the cores taken through the various dowel bars on U.S. 50. In general, the FRP bars are relatively free of deterioration (some minor pitting was observed at the end of the bars), as would be expected. The stainless steel tubes are also in relatively good condition, with a rust stain observed on one sample and another sample exhibiting a dull, lusterless appearance. For the stainless steel clad bars located in the westbound direction), it was noted that the coating was generally in excellent condition with a few observations of minor corrosion or rust stains. Finally, the epoxy-coated dowel bars exhibited highly variable conditions, with a few bars showing no deterioration, others showing some minor corrosion or pitting, and still others exhibiting debonding of the epoxy coating and corrosion beneath the coating.

As described in Chapter 4, chloride content testing was performed to determine the level of chlorides at the depth of the dowels, to see if it might have any bearing on the condition or performance of the dowel bars. Testing was performed at the joint face and at the outside edge of the retrieved core. This information is also presented in Table C-1 in Appendix C, but a cursory examination of the chloride contents with the dowel deterioration does not provide any discernable trends. The chloride contents were also plotted against the measured LTE values (see Figures 8 and 9), but there is considerable scatter in the data.

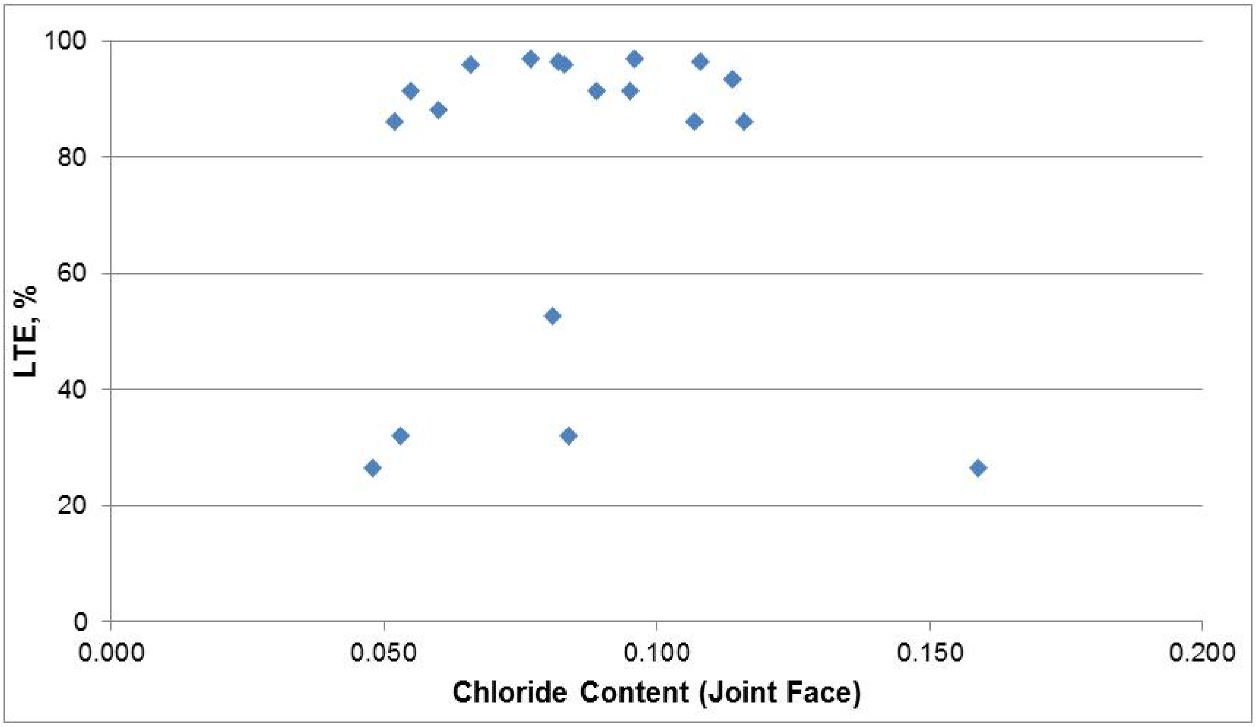

Figure 8. Chloride content (joint face) vs. LTE for OH 2 (ATH-50).

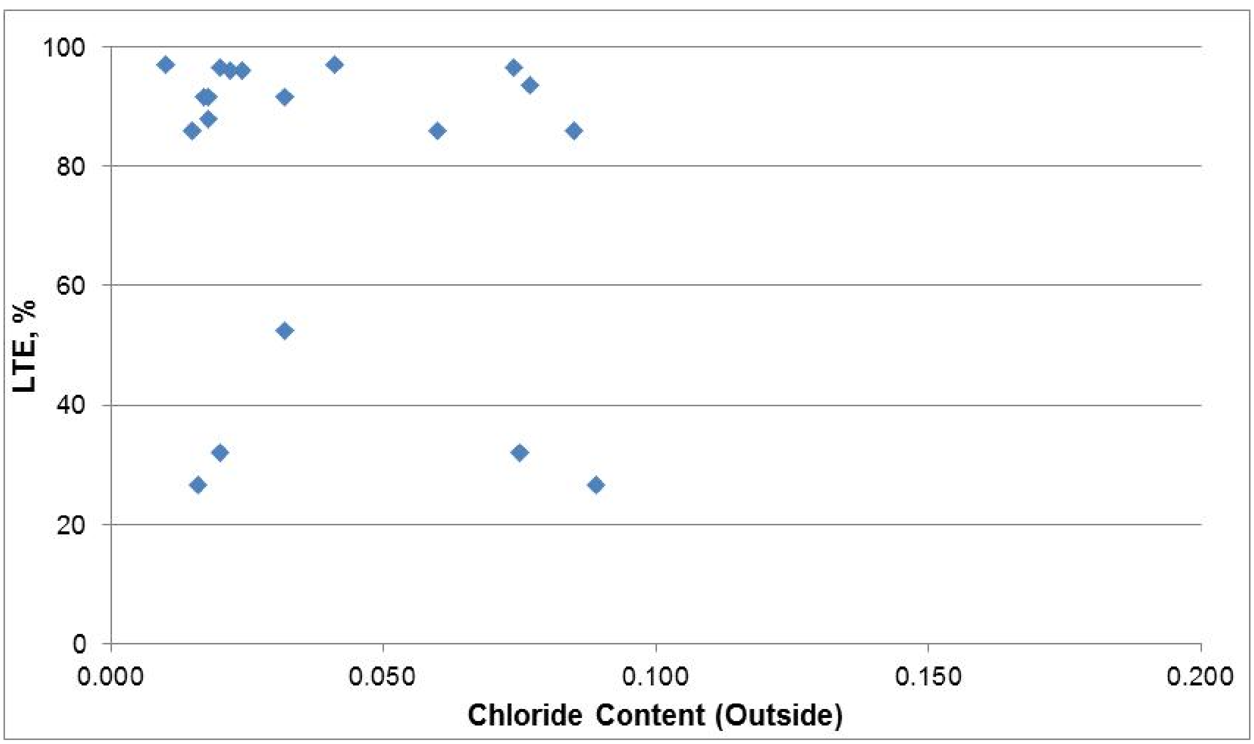

Figure 9. Chloride content (outside of core) vs. LTE for OH 2 (ATH-50).

Overall Roughness Summary of OH 2 Test Sections

Table 4 presents a summary of the current (2011) roughness data for the OH 2 test sections. Even though the FRP bars are exhibiting lower LTE values, those apparently are not affecting the overall rideability of the pavement.

Table 4. Average 2011 roughness values for OH 2 test sections.

Test Section	International Roughness Index, in/mi
Epoxy-Coated Dowels (Control 1)	95.7
Stainless Steel Tubes	72.9
FRP Bars	91.0

Evaluation of SR 7 Project in Belmont County, Ohio (after 28 years of traffic)

One of the earliest known projects incorporating FRP dowel bars was constructed on SR 7 in Belmont County, Ohio, in 1983. The SR 7 project originally consisted of ten joints containing 1.25-in (32-mm) diameter FRP dowels bars made from vinyl ester resin and 78 percent type E glass, but in 1988 one of the joints was removed for laboratory evaluation and replaced with a full-depth repair, which used 1.5-in (38-mm) FRP dowel bars.

In March 2011, FWD testing was performed on the nine remaining FRP joints (and on both sides of the full-depth joint repair) and on two sets of ten adjacent joints (located on either side of the FRP joints) containing conventional 1.25-in (32-mm) diameter, epoxy-coated dowel bars. These data are provided in Table A-3 of Appendix A and are presented graphically in Figure 10. The 2011 deflection data for the nine FRP vinyl ester resin and E-glass joints revealed average LTE values of 43.5 percent on the approach joint and 34 percent on the leave joint. The LTE values for one set of ten conventional epoxy-coated dowel bars averaged 65 percent on the approach joint and 66.5 percent on the leave joint, while the LTE values for the other set of ten conventional epoxy-coated dowel bars was 57 percent on the approach joint and 68 percent on the leave joint. Although the low LTEs on the vinyl ester FRP dowel section may indicate the need for larger diameter FRP dowels, the proven performance of this project (28 years of service with little or no joint distress) suggest that good performance may not be entirely related to load transfer efficiency. Over its 28-year service life, this pavement has sustained approximately 7 million 18-kip (80-kN) ESAL applications.

Coring of dowel bars on BEL-7 (see Table C-3 in Appendix C) showed that the only FRP bar retrieved was exhibiting no deterioration, whereas epoxy-coated bars in nearby adjacent joints were showing various types of deterioration, including the splitting of the epoxy coating, rusting of the bar under the coating, and some pitting where the steel had been exposed. It is interesting that the 1.25-in (32-mm) diameter vinyl ester resin with 78 percent E-glass FRP dowels on SR 7 performed significantly better than the 1.5-in (38-mm) polyester resin FRP dowels on U.S. 50, based on similar age comparisons (SR 7 data in 1998 from the MDA [1998] report after 15 years of service, and the U.S. 50 data collected in 2011 after 14 years of service).

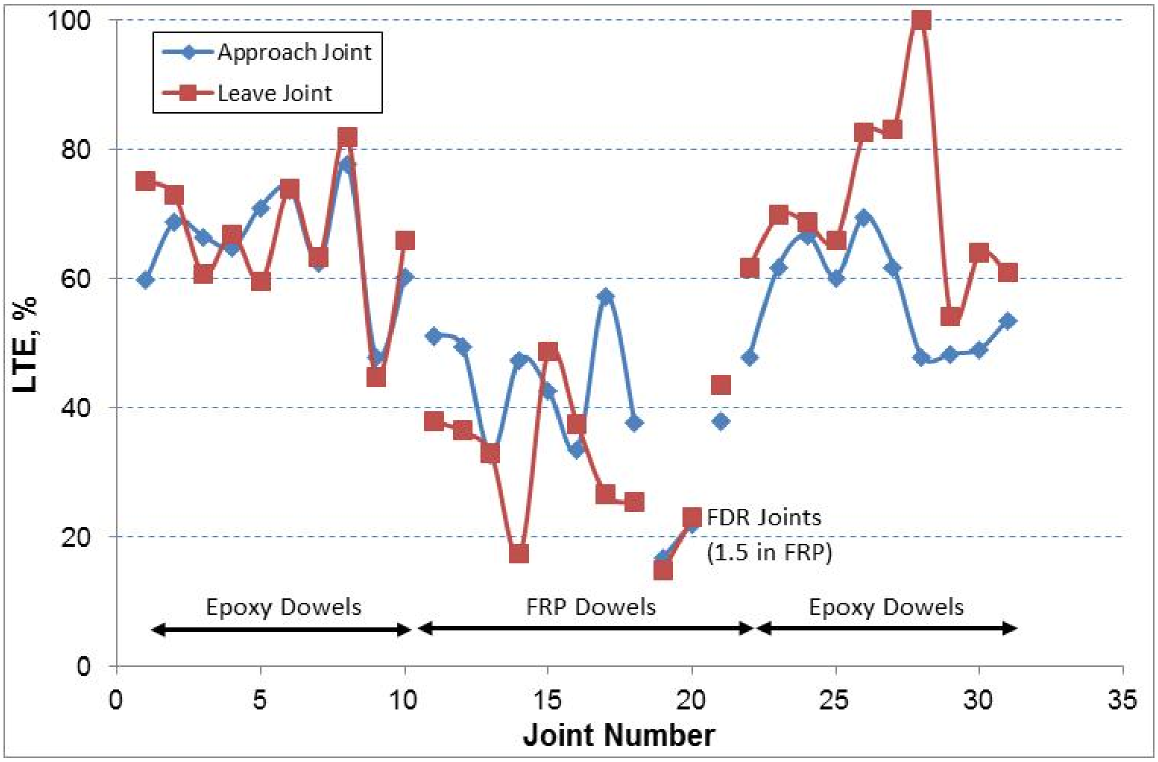

Figure 10. 2011 LTE data for BEL-7.

The data from this project suggest that larger diameter vinyl ester or epoxy resin dowels and ECR glass (minimum fiber content in the range of 75 to 80 percent) are still needed to ensure good long-term performance from FRP dowels. Furthermore, it should be noted that the 1.5-in (38-mm) polyester dowels used for the full-depth repair placed in 1998 to repair the 1.25-in (32-mm) vinyl ester doweled joint removed for laboratory testing, exhibited lower LTE values (ranging from 14.7 to 23.0 percent) than the smaller vinyl ester FRP bars at the other joints.

Evaluation of Plastic-Coated Dowels in Ohio

One of the unexpected results from the FWD testing and coring of older doweled projects was finding plastic coated dowels on two older projects constructed in 1978: Route 81 in Allen County (ALL-81) and Route 59 in Summit County (SUM-59). FWD testing was performed on nine of the twelve joints where cores that were taken; however, elevated temperatures during the testing potentially limit the value of that data. The cores at the joints taken through the dowel bars revealed that the plastic coating was not debonded from the steel in most cases (see summary of dowel condition reported in Table 5). There was also an unknown intermediate coating (black in color) that seemed to provide a secondary layer of protection for the steel even where the plastic had debonded from the bar; this resulted in a lesser degree of damage to the underlying steel in the plastic-coated dowels as compared to the epoxy-coated steel dowels.

The plastic-coating material (polyethylene) conforms to AASHTO M 254, Type A, which has been routinely used in Louisiana since the 1970s. The intermediate coating (undercoating) is reported to be MC-30. The process was originally developed to protect underground pipelines from corrosion. Research conducted to evaluate the suitability of polyethylene and information on the type and thickness of the undercoating is available (Broesti 1966).

Table 5. Summary of plastic-coated dowels retrieved on State Route 59 and State Route 81.

Cty-Rte	Direction	Pavt Age	Core No.	Station	Dowel Position	Dowel Diameter (in)	Dowel Deterioration
ALL-81	westbound	33	64	451+70	18" from EOP	1 1/4	Coating in good condition with one nick and one split. Some minor rust under the split and nick. Remainder of bar in excellent condition
		33	65	452+10	18" from EOP	1 1/4	Coating on leave side of joint appears to have been damaged during installation or placement of concrete. Some rusting and pitting in damaged areas.
		33	66	452+50	18" from EOP	1 1/4	Coating in good condition with several area where coating had small splits. Some minor rust under the splits.
		33	67	452+89	18" from EOP	1 1/4	Coating in good condition. Some areas on side of bar where coating was abraded and thinned.
		33	68	453+29	18" from EOP	1 1/4	Coating damaged in joint area. Elsewhere, coating in fair condition with areas where coating was abraded and thinned. Rusting and pitting in joint area under coating.
		33	69	453+69	18" from EOP	1 1/4	Coating in good condition. Some rust stains on coating in joint area. Light rust under coat in stained area.
SUM-59	northbound	34	61	180+26	21" from inside EOP	1 1/4	Coating in good condition. Some rust stains on coating in joint area. No rust under coat in stained area.
		34	62	180+46	21" from inside EOP	1 1/4	Coating in good condition. Minor rust stain on coating in joint area. Coating in joint area had minor abrading and thinning
		34	63	180+86	21" from inside EOP	1 1/4	Coating in good condition. Rust stain and some failure of the coating in joint area. Coating in joint area had minor abrading and thinning. Some rust under coating in stained area
		34	58	188+90	21" from inside EOP	1 1/4	Coating in good condition except joint area where coating has failed. Rust and some section loss in the joint area
		34	59	189+30	21" from inside EOP	1 1/4	Coating in good condition. Two locations with minor damage in coating
		34	60	189+70	21" from inside EOP	1 1/4	Coating in good condition. Two locations with minor damage in coating

All bars had a black paste type material, possibly grease or asphalt cement, between the coating and the bar. This material protected the bar from extensive rusting when the coating was split or abraded. Almost all bars had a ridge in the joint area which is probably due to movement of the coating and paste material. Joints were tight.

A photo of the dowel coating from a typical core (see Figures 11 and 12) and two photos of the excellent condition of the surface of the pavements (see Figures 13 and 14) demonstrate the outstanding performance of the plastic-coated dowels. Over their 33-year service lives, the pavements on Route 81 in Allen County had sustained nearly 10 million ESAL applications whereas the pavements on Route 59 in Summit County had sustained approximately 6 million ESAL applications.

Figure 11. Photo of plastic-coating of dowel on State Route 81 after 33 years of service.

Figure 12. Photo of plastic-coating of dowel on State Route 59 after 33 years of service.

Figure 13. Photo of surface condition of pavement with plastic-coated dowels (State Route 81).

Figure 14. Photo of surface condition of pavement with plastic-coated dowels (State Route 59).

Plastic-coated dowels appear to be a potential alternative to epoxy-coated dowels. Tests show that polyethylene plastic has excellent resistance to many corrosive agents, and also exhibits good tear resistance, high impact strength, and good puncture resistance. The specially developed undercoating provides excellent adhesion to both metal and plastic. Furthermore, it stays alive and never loses its ability to seal accidental cuts or abrasions (Broesti 1966). In addition, plastic-coated dowels do not require a bond breaker, are about the same cost as epoxy-coated dowels, and are less susceptible to damage during construction. Most cores showed signs of movement in the plastic-coating itself at the joint face but the corrosion protection was still effective. These bars can be easily constructed using dowel bar inserters (preferred method) or welded in metal baskets and the fact that they do not need a bondbreaker during construction is a significant advantage over epoxy-coated dowels.

Evaluation of Wisconsin 2 Project (STH 29)

General information on this project was provided previously in Chapter 3, with more detailed information found in Appendix D. The Wisconsin DOT has performed regular testing of this roadway since it was originally constructed in 1997. In 2009-2010, the WI 2 project was evaluated with some additional FWD testing, coring of selected dowels, and corrosion testing of the concrete at the dowel bar level. The 2009-2010 data collection effort and the detailed data are available in a field summary report (Crovetti 2010), which is reproduced in Appendix E.

Table 6 summarizes the performance and condition of the dowels on the Wisconsin 2 project. A review of the photographic data for the WI 2 project revealed significant concerns about the quality of the concrete at the joint faces that likely will affect the performance of the various dowel bars. In addition, it was reported that the core bit used was in poor condition (very worn with little material left on the diamond segments), which resulted in significant cracking of the cores during the field sampling. However, it was reported that visual inspection and photographs taken during the coring did not indicate any cracking in the concrete pavement at the dowel level. Wisconsin is part of a multi-state study (along with Illinois, Indiana, Iowa, Kansas, Michigan, and New York) investigating concrete joint deterioration under a pooled-fund study.

Table 6. Evaluation of dowel performance on WI 2.

Highway Segment	Pvmt Age	Joint No. Dowel Pos (Dowel Type)	FWD Test Results				Dowel Deterioration	Chloride Content % by mass	
			App LT%	App DT*	Lv LT%	Lv DT*		Joint Face	Outside
STH 29-Clark	12	CTL-Jt6-D10 (Epoxy)	n/a	n/a	n/a	n/a	Minor	0.1225	0.1600
		CTL-Jt6-D3 (Epoxy)	90	9.4	91	9.2	None	0.0780	0.0820
		CTL-Jt7-D10 (Epoxy)	n/a	n/a	n/a	n/a	Extensive	0.1675	0.1725
		CTL-Jt7-D3 (Epoxy)	87	9.8	80	10.4	None	0.1500	0.1450
		CP-Jt3-D10 (FRP)	n/a	n/a	n/a	n/a	None*	0.1475	0.1075
		CP-Jt3-D3 (FRP)	61	9.6	66	10.0	None*	0.0440	0.0720
		GF-Jt7-D10 (FRP)	n/a	n/a	n/a	n/a	None*	0.1525	0.0900
		GF-Jt7-D3 (FRP)	78	10.5	74	10.7	None*	0.1275	0.0380
		RJ-Jt1-D10 (FRP)	n/a	n/a	n/a	n/a	Minor	0.1475	0.1250
		RJ-Jt1-D3 (FRP)	75	10.6	69	11.0	None*	0.1125	0.0970
		HF-Jt8-D10 (SS Tubes)	n/a	n/a	n/a	n/a	None*	0.0980	0.1350
		HF-Jt8-D3 (SS Tubes)	85	9.7	86	9.5	None*	0.1250	0.0880
		SS-Jt5-D10 (Solid SS)	n/a	n/a	n/a	n/a	None*	0.1275	0.1475
		SS-Jt5-D3 (Solid SS)	50	9.0	60	8.8	None*	0.1075	0.0800

App DT* = total joint deflection in mils normalized to a load of 9,000 lbs.

None* = Fiber Reinforced Polymer Dowel, Extensive* indicates samples where no intact concrete was available.

n/a indicates core sample where no dowel bar was provided.

Figure 15 presents a summary of the available LTE data by dowel bar type (2010 data). This figure shows that the epoxy-coated dowel bars have maintained nearly constant LTE values over the nearly 13-year evaluation period, while the FRP bars (Creative Pultrusions, Glasforms, and RJD) are somewhat lower. The sections containing stainless steel tubes still show acceptable LTE values, but the most recent data for the sections with solid stainless steel bars are very low (just over 50 percent). However, it should be noted that the solid stainless steel dowels on this project were placed in two alternative configurations (one with three dowels per wheelpath and one with four dowels per wheelpath), which likely explains the differences.

A summary of the evaluation of the different types of dowel bars included in the project is provided in the next sections.

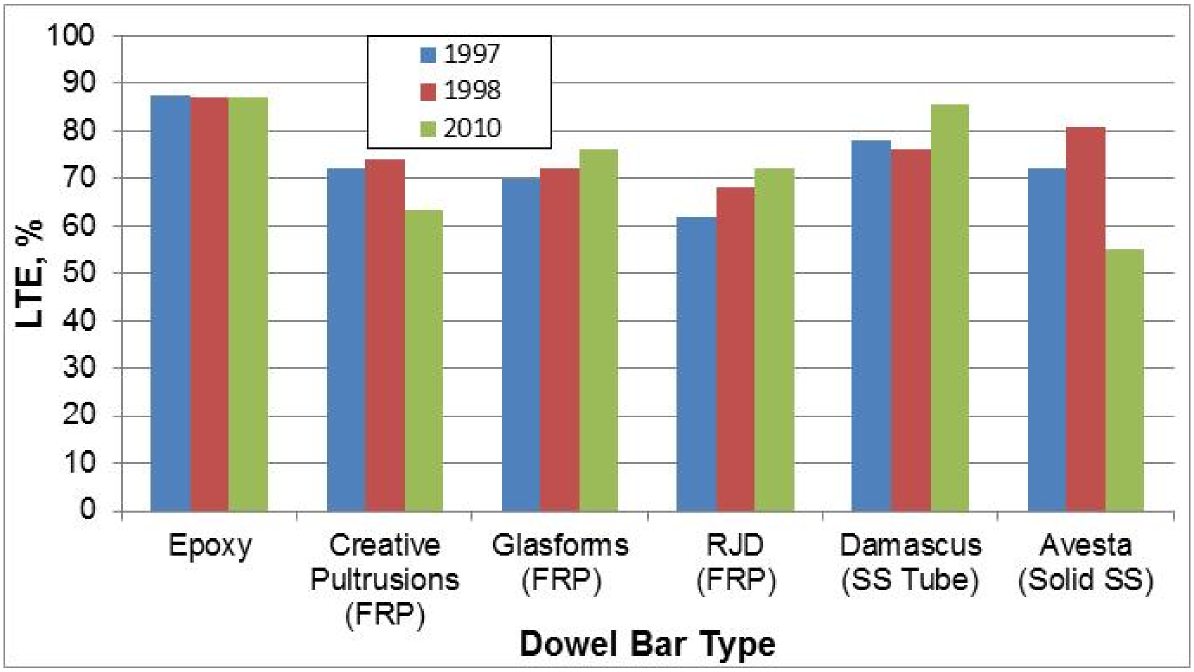

Figure 15. Summary of aggregated, historical LTE values for WI 2.

Evaluation of FRP Dowels

One major issue in evaluating FRP performance is verifying the type of resin and type and minimum percentage of fiberglass used by each of the different manufacturers or suppliers of the FRP dowels on the various experimental projects; this is because of the significant differences in detailed laboratory test results and the effect of those properties on field performance. Further complicating the analysis is that only two states (OH and WI) conducted the follow-up testing and FRP dowels used the OH 2 project were of similar polyester material.

Three manufacturers or suppliers of FRP composite dowel bars were used on the WI 2 project:

- Glasforms (GF): Uncertain resin type.
- Creative Pultrusions (CP): Vinyl ester bisphenol, an epoxy matrix.
- RJD Industries (RJD): Polyester resin with E-glass (same material as used on OH 2).

Based on a single FWD test of the joint (after about 13 years of traffic), the GF dowels had the highest LTE (78 percent on approach and 74 percent on leave slab); the RJ dowels had 75 percent on the approach and 69 percent on the leave slab; and the CP dowels had 61 percent on the approach and 66 percent on the leave slab. These LTE results are low and indicate that larger diameter dowels or closer spacing would be required to increase the LTE. Also, the total joint deflection in mils normalized to a load of 9,000 lbs (40 kN) is 10 mils (0.25 mm) or higher indicating poor joint support by the concrete and the base/subbase/subgrade. Due to their greater flexibility compared to the epoxy-coated mild steel dowels used as a control, FRP dowels appear to be more sensitive to poor support conditions. As mentioned earlier, the considerable deterioration of the concrete crack faces at the joint significantly reduces any contribution from aggregate interlock to help transfer loading at the joints.

Evaluation of Type 304 Hollow Tubes Filled with Mortar

The FWD deflection on the one joint tested was 85 percent on the approach slab and 86 percent on the leave slab. The normalized total joint deflection was 9.7 on the approach slab and 9.5 mils (0.24 mm) on the leave slab. This just meets the proposed desirable criteria for early performance evaluation of a minimum of 85 percent LTE and a maximum total joint deflection of 10 mils (0.25 mm). No corrosion of the dowel bar was noted.

Evaluation of Type 304 Solid Stainless Steel Dowels

The FWD deflection on the one joint tested was 50 percent on the approach and 60 percent on the leave slab. The total joint deflection was 9.0 mils (0.23 mm) on the approach slab and 8.8 mils (0.22 mm) on the leave slab. The lower LTE values for this dowel bar are likely attributed to the alternative dowel layout used on this project (3 and 4 dowels per wheelpath). No corrosion of the dowel bar was noted.

Evaluation of Epoxy-Coated Mild Steel Dowels (Control)

The FWD deflections at two of the epoxy-coated dowel control joints showed 90 percent and 87 percent LTEs on the approach slabs and 91 percent and 80 percent on the leave slabs. The total joint deflections were 9.4 mils (0.24 mm) and 9.8 mils (0.25 mm) on the approach slabs and 9.2 mils (0.23 mm) and 10.4 mils (0.26 mm) on the leave slabs. The leave slab with 80 percent LTE had the 10.4 mils (0.26 mm) total deflection indicating poor support conditions. The corrosion of the epoxy-coated dowel bars will be addressed in more detail in Chapter 6.

Roughness and Faulting Summary for WI 2 Sections

Roughness and faulting measurements were collected on the WI 2 sections in 2010. Figure 16 presents the average IRI values for each section by traffic lane. This figure shows that all sections are providing reasonable levels of rideability, with one of the FRP sections providing the lowest roughness based on 2011 measurements. The highest roughness is exhibited by the section with stainless steel tubes. A historical summary of the IRI values by section is provided in Figure 17 for the 13-year monitoring period. The sections shown in Figures 16 and 17 are designated as follows:

- C1 = Control 1 (epoxy in standard dowel layout)
- CP = Creative Pultrusions (FRP)
- GF = Glasforms (FRP)
- RJD = RJD Industries (FRP)
- HF = SS Tube
- 3Ea = Epoxy in alternate layout
- 3S = Solid stainless steel in alternate layout
- 4S = Solid stainless steel in alternate layout
- 4E = Epoxy in alternate layout
- 3Eb = Epoxy in alternative layout
- 2E = Epoxy in alternate layout
- 1E = Epoxy in alternate layout
- C2 = Control 2 (epoxy in standard dowel layout)

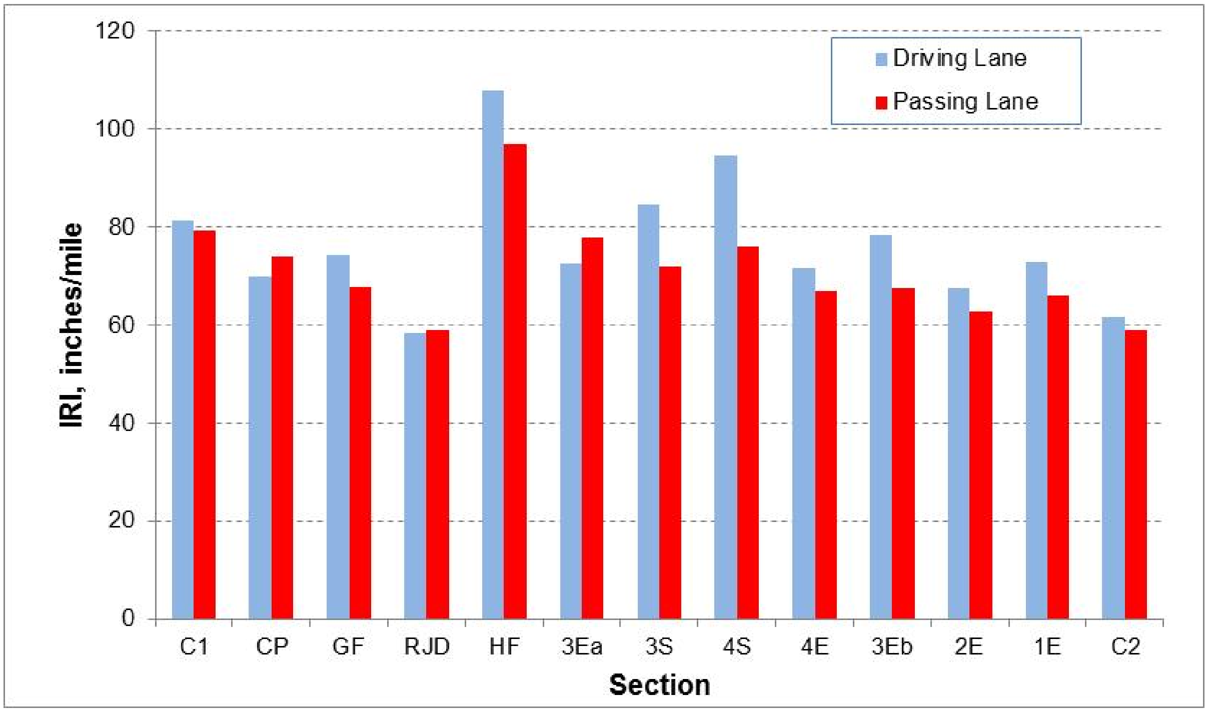

Figure 16. 2010 roughness values for WI 2 sections by lane.

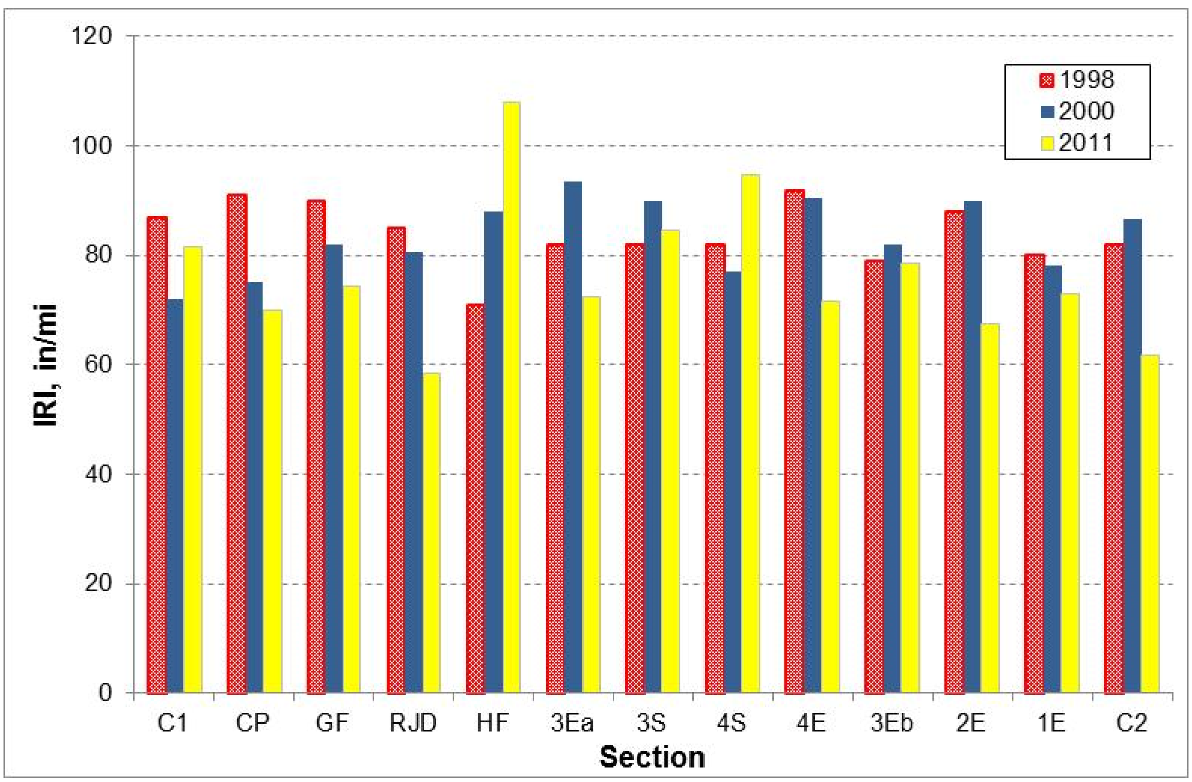

Figure 17. Historical summary of roughness data for WI 2 sections.

Figure 18 presents the average 2010 faulting data for each of the WI 2 sections. The faulting levels are generally reasonable for a 13-year-old pavement, with levels exceeding about 0.1 in (2.5 mm) beginning to have a noticeable effect on ride. To date, only one section has exceeded that value (HF, the section with stainless steel tubes).

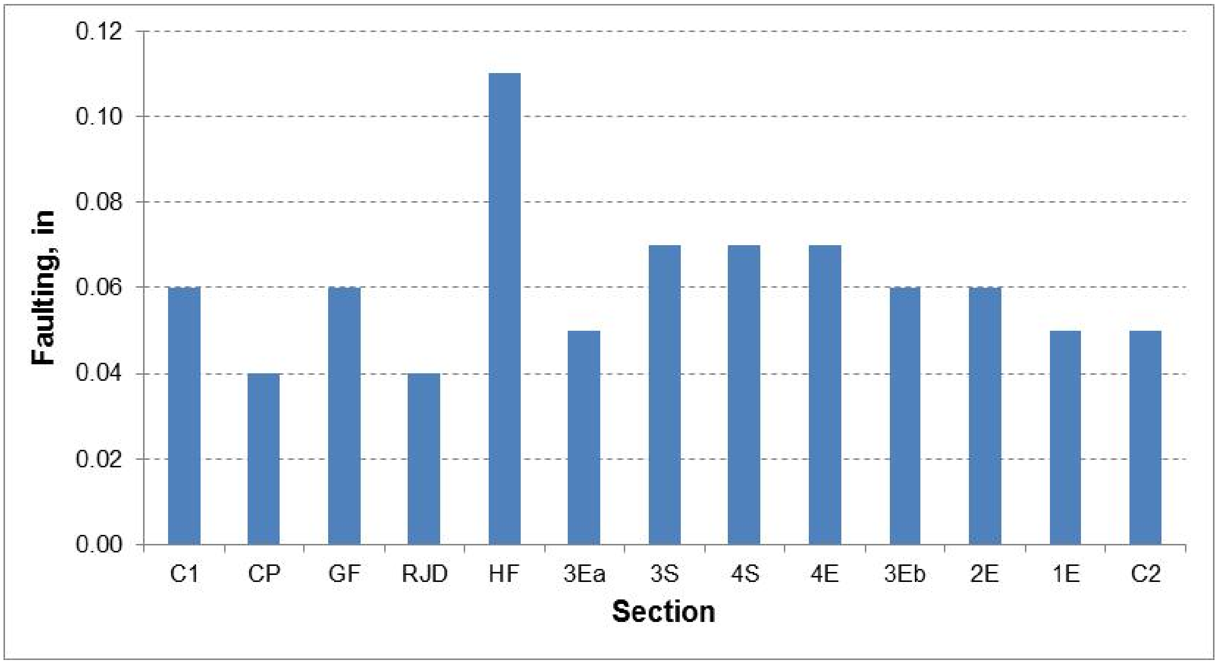

Figure 18. Average 2010 faulting values for WI 2 sections.

Chloride Analysis of WI 2 Sections

The WI 2 sections that were cored as part of the 2010 field evaluation work were also subjected to a chloride content analysis, using a process and procedure similar to what was used by the Ohio DOT in their work. These data were presented earlier in Table 6. An attempt was made to correlate dowel condition with chloride content, as shown in Figure 19. Unfortunately, no clear trend exists between the chloride content and dowel condition, although it is noted that the joint face chloride contents are consistently higher than those for the outside of the joint.

General Comment: Performance of Alternative Dowel Bar Materials on WI 2

The limited investigation of the dowel bars on this experimental section after 13 years of traffic verified the low LTEs on the FRP dowel sections, which likely will affect their long-term performance. Larger diameter dowels or closer dowel spacing will be required to help increase the LTE. The greater flexibility of the FRP dowels also appears to decrease performance compared to the epoxy-coated mild steel dowels used as control when poor support conditions (poor durability of the concrete at the joint, or poor support of the base/subbase/subgrade) are present. There was minor corrosion/deterioration of one of the FRP dowels likely due to abrasion of the surface. One of the four epoxy-coated dowels removed from the cores showed loss of a small section of coating at the joint and one dowel bar had some loss of section due to corrosion at the joint.

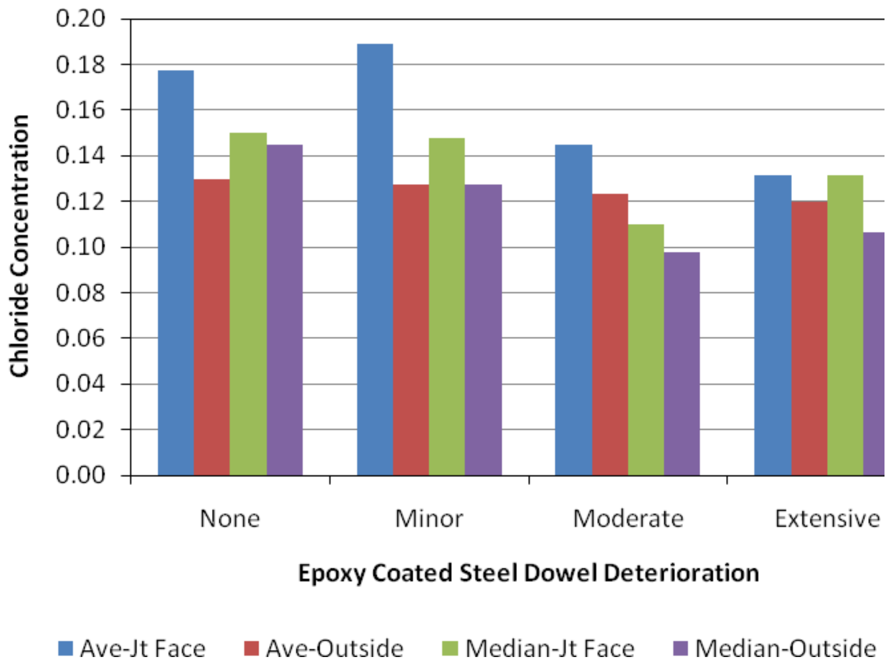

Figure 19. Chloride concentration vs. dowel bar condition.

CHAPTER 6. EVALUATION OF EPOXY-COATED DOWELS USED AS CONTROL AND AFTER 15 TO 30+ YEARS OF SERVICE

Introduction

As described in Chapter 4, selected highway agencies collected information on the condition of their conventional epoxy-coated dowel bars placed in older concrete pavement projects. This was intended as a means of determining the extent of the dowel bar corrosion problem. Ohio and Wisconsin collected this information on a number of older concrete pavement projects, which included retrieving cores and inspecting the condition of the epoxy-coated dowels, as well as the determination of chloride contents at locations just above the dowels at the joint face and at the outside edge of the core.

Evaluation of Epoxy-Coated Dowels in Ohio

Table 7 summarizes the condition of the dowel bars on OH 2 after 12 to 14 years of traffic. With reference specifically to the epoxy-coated dowel bars, in most cases the epoxy coating was damaged at the joint face. Furthermore, the epoxy coating could be easily removed and the mild steel bar was rusted under the coating, although the cross section of the dowel had not yet been seriously compromised. This finding is somewhat surprising after only 12 to 14 years of traffic.

Table 8 summarizes testing conducted on Ohio statewide concrete pavement projects with 1.25- to 1.5-in (32- to 38-mm) diameter epoxy-coated dowels after 15 to 30^+ years of traffic. As part of the data collection efforts, twelve cores were taken of joints with plastic-coated dowels.

A subjective evaluation of the condition of the epoxy coating was made after the dowel bar was removed from the core. In most cases the core had to be broken so the dowel could be removed. The subjective evaluation consisted of the following assessment categories:

- No deterioration.
- Epoxy coating damaged.
- Rust stain on the epoxy coating.
- Coating easily removed.
- Rust under the coating.
- Bar pitted under the coating.
- Amount of dowel bar section loss (if any).

The loss of section is the most critical form of deterioration. While there was some loss of section noted on a few projects, the most common form of deterioration on the older Ohio projects was the epoxy debonded from the mild steel bar with rusting or pitting under the coating. Unfortunately, the amount of deterioration of the dowel needed to adversely affect the performance of the pavement cannot be determined from this limited sample, but the concrete quality on the OH 2 project was reported to be poor. In general, the epoxy coating appears to be performing well for 25 to 30 years. For long-life pavements (30 to 50 years or more), more durable concrete and more durable dowel bars likely are required.

Table 7. Evaluation of dowel bar deterioration on OH 2.

Cty-Rte	Dir	Pvmt Age (yrs)	Core No.	Station	Dowel Position (in)	Dowel Bar Material	FWD Test Results		Dowel Deterioration	Chloride Content % by mass	
							App LT%	Leave LT%		Jt Face	Outside
ATH-50	Eastbound	14	1-1-O	102+16	6	Epoxy coated steel	90	93	Rust stain in epoxy coating at joint location. Coating easily removed. Bar rusted under coating.	0.095	0.032
			1-1-I	102+16	90	Epoxy coated steel			Rust stain in epoxy coating at joint location. Coating easily removed. Bar rusted under coating.	0.055	0.017
			1-2-O	103+02	6	Epoxy coated steel	97	95	None.	0.083	0.024
			1-2-I	103+02	90	Epoxy coated steel			“Bubble” in epoxy coating along dowel axis. Coating easily removed. Bar rusted under bubble.	0.066	0.022
			2-1-O	103+41	6	Grout filled stainless steel tube	98	95	Surface of bar has dull appearance.	0.108	0.074
			2-1-I	103+41	90	Grout filled stainless steel tube			None.	0.082	0.020
			2-2-O	104+05	6	Grout filled stainless steel tube	97	97	Rust stain on top of bar at joint face.	0.096	0.041
			2-2-I	104+05	90	Grout filled stainless steel tube			None.	0.077	0.010
			3-1-O	106+79	6	Composite	16	37	One end of dowel is rough and pitted.	0.159	0.089
			3-1-I	106+79	90	Composite			One end of dowel is slightly pitted.	0.048	0.016
			3-2-O	107+22	6	Composite	30	34	One end of dowel is rough and pitted.	0.084	0.075
			3-2-I	107+22	90	Composite				0.053	0.020
			4-1-O	108+37	6	Epoxy coated steel	84	88	Epoxy coating damaged near joint face.	0.116	0.060
			4-1-I	108+37	90	Epoxy coated steel				0.107	0.085
			4-2-O	108+88	6	Epoxy coated steel	92	95	None.	0.114	0.077
	Westbound	12		526+00	6	Epoxy coated steel	90	86	Epoxy coating damaged at joint location. Coating easily removed. Some rust under coating.	*	
				526+00	90	Epoxy coated steel				0.060*	0.018*
				525+49	6	Epoxy coated steel	85	87	Epoxy coating in excellent condition.	*	
				525+49	90	Epoxy coated steel			Epoxy coating damaged at joint location. Coating easily removed. Some rust under coating.	0.052*	0.015*
				524+13	6	Stainless steel clad	88	95	Coating in excellent condition with minor corrosion, minor pitting at joint face.	*	
				524+13	90	Type 316 Stainless steel clad			Coating in excellent condition with minor corrosion, minor pitting and small rust stains at joint face.	0.089	0.018*
				523+62	6	Stainless steel clad	40	65	Gap around bar on approach side. Coating in excellent condition with minor corrosion, minor pitting at joint face.	*	
				523+62	90	Stainless steel clad			Coating in excellent condition with minor corrosion, minor pitting at joint face.	0.081*	0.032*

All dowels are 1.5-in (38-mm) diameter. Type 316 stainless steel tube wall thickness is 0.25 in (6.4 mm).

* Samples from 6 in (152 mm) and 90 in (2286 mm) dowel positions were combined prior to testing.

Table 8. Evaluation of epoxy-coated dowel bars on older concrete pavement projects in Ohio.

Cty-Rte	Dir	Pvmt Age (yrs)	Core No.	Station	Dowel Position	Dowel Dia. (in)	Dowel Deterioration	Chloride Content % by mass	
								Jt Face	Outside
CUY-176	Southbound		1	108+92	wheelpath	1.5	Rust stain in epoxy coating at joint location. Coating easily removed around stain. Bar pitted under coating.	0.069	0.290
			2	108+50	wheelpath	1.5	Bubbles in epoxy coating above “seam” in bar. Coating easily removed. Bar pitted.	0.065	0.021
			3	108+71	wheelpath	1.5	None.	0.066	0.036
			4	105+98	wheelpath	1.5	Rust stain in epoxy coating at joint location. Coating easily removed around stain. Bar bitted under coating.	0.051	0.030
			5	105+79	wheelpath	1.5	None. Epoxy coating damage during removal from core.	0.056	0.039
			6	105+58	wheelpath	1.5	None.	0.007	0.006
		14	7	191+95	wheelpath	1.5	None.	0.129	0.129
			8	191+74	wheelpath	1.5	None.	0.059	0.057
SUM-76	Eastbound	15-17	9	697+65	wheelpath	1.5	Epoxy coating damaged in joint area. Bar pitted under coating.	0.086	0.044
			10	697+44	wheelpath	1.5	None.	0.101	0.055
			11	696+18	wheelpath	1.5	None.	0.123	0.038
			12	697+23	wheelpath	1.5	None. Epoxy coating not fully bonded to bar.	0.097	0.036
			13	695+76	wheelpath	1.5	Rustin stain in epoxy coating at joint location. Coating easily removed around stain. Bar pitted under coating.	0.079	0.045
BEL-7	Southbound		14	750+48	42	1.25	Epoxy worn to metal on moving end of dowel. Epoxy easily removed. Rust under the epoxy.	0.181	0.077
			15	750+88	30*	1.25	Corrosion visible in areas with deteriorated epoxy coating. Coating easily removed. Rust and pitting over much of the bar under epoxy coating.	0.186	0.165
			16	750+88	42*	1.25	Epoxy coating damaged, some possibly due to coring operation. Some epoxy coating easily removed. Areas of rust under epoxy coating.	0.225	0.158
			16	751+28	42	1.25	Corrosion visible in areas with deteriorated epoxy coating. Coating easily removed. Rust, pitting and scaling under epoxy in joint area. Some section loss.		
		28	17	754+48	18*	1.25	Epoxy missing over most of bar. Bar rusty and pitted, with section loss.		
			17	754+48	30*	1.25	Epoxy missing over most of bar. Bar rusty and pitted, with section loss	0.234	0.206
			18	754+88	30	1.25	Rust and corrosion in joint area. Epoxy coating easily	0.274	0.248
			19	755+28	30	1.25	Epoxy deteriorated/ missing in joint area. Epoxy coating easily removed. Areas of pitted and scaling with some section loss.	0.244	.0101

*Due to joint deterioration, two cores were taken. Material removed from both cores as needed to get sufficient materials for testing.

Table 8. Evaluation of epoxy-coated dowel bars on older concrete pavement projects in Ohio (continued).

Cty-Rte	Dir	Pvmt Age (yrs)	Core No.	Station	Dowel Position	Dowel Dia. (in)	Dowel Deterioration	Chloride Content % by mass	
								Jt Face	Outside
BEL-7	Southbound	28	20	746+93	18*	1.25	None.	0.172	0.128
				746+93	42*	1.25	Epoxy split, steel exposed at joint face. Epoxy easily removed from bar. Bar pitted in joint area. Some section loss.		
			21	746+76	18	1.25	Epoxy coating split at joint face, some corrosion. Damage to epoxy coating during removal (red circle). Epoxy easily removed. Some rust and pitting under epoxy.	0.204	0.155
			22	746+60	18	1.25	Epoxy coating worn on one end of dowel. Epoxy coating easily removed on this end. Some rust under epoxy.	0.186	0.176
				746+60	30	1.25	Epoxy coating split at joint face. Epoxy worn to metal on moving end of dowel. Epoxy easily removed. Rust under the epoxy.		
			32	743+36	42	1.25	Composite bar; no deterioration.	0.166	0.240
			23	742+86	18	1.25	Epoxy split on one end of dowel, rust stain at the joint face. Epoxy easily removed from that end of dowel. Steel rusted and pitted.	0.183	0.139
			24	742+68	18	1.25	Epoxy split, steel exposed at joint face. Epoxy easily removed from bar. Bar rusted and pitted in area where metal exposed. Some section loss.	0.183	0.132
			25	742+51	18	1.25	Epoxy split, steel exposed at joint face. Epoxy easily removed from bar. Bar rusted and pitted in area where metal exposed.	0.240	0.135
MOT-35	Westbound	20-24	26	787+54	wheelpath	1.25	Epoxy damaged at joint face, damaged area rusted. Coating easily removed around rusted area. Bar pitted under coating.	0.017	0.011
			27	737+69	wheelpath	1.25	Epoxy damaged at joint face, damaged area rusted.	0.048	0.012
			28	737+84	wheelpath	1.25	None.	0.022	0.016
			29	756+00	wheelpath	1.25	Rust stain in epoxy coating at joint face.	0.140	0.045
			30	756+15	wheelpath	1.25	Epoxy missing from bottom half of one end of dowel, steel pitted in area of missing epoxy. Some loss of section.	0.065	0.028
			31	756+30	wheelpath	1.25	Epoxy damaged on one end of dowel.	0.064	0.033

*Due to joint deterioration, two cores were taken. Material removed from both cores as needed to get sufficient materials for testing.

There does not appear to be a direct correlation of chloride content at the joint face and the dowel bar deterioration. It is more likely that the quality of the initial epoxy coating and/or damage to the coating during construction is a more significant factor than the chloride content at the joint.

As noted earlier, twelve cores were taken that contain plastic-coated dowels (33 years old). The plastic coating was occasionally debonded from the steel but the surface condition of the joint was noted to be very good for the age of the pavement. An intermediate coating (black in color, reportedly an MC-30) was also present that appeared to provide a secondary layer of protection to the steel; this may help explain why the steel was still in good condition even where the plastic coating had debonded.

Evaluation of Epoxy-Coated Dowels in Wisconsin

Table 6, presented previously, provides a summary of the dowel deterioration on WI 2, including the condition of the conventional epoxy-coated dowel bars. Only one of the epoxy-coated dowel bars was noted to have any damage, which was somewhat extensive. Interestingly, the chloride content for that dowel bar was the highest observed for any of the dowels.

Table 9 summarizes testing conducted on Wisconsin statewide concrete pavement projects after more than 15 years of service. A preliminary evaluation of the core data results indicated that of the 44 cores:

- Only ten cores were retrieved intact.
- Three cores indicated inadequate dowel bar embedment length (end of bar in the core).
- One core contained no dowel bar.
- Five cores were retrieved above the dowel.

The large number of deteriorated core samples was believed to be at least partially the result of the very poor condition of the coring bit. Visual observations and photographs taken during coring showed no cracking of the concrete pavement from which the cores were removed. However, photographs taken of the cores show a significant amount of deterioration of the concrete in the joint area, which makes any significant contribution from aggregate interlock to joint load transfer unlikely. Also, the inadequate dowel bar embedment length suggests poor quality control of the joint sawing operation during construction. It appears critical that the quality of the concrete be improved if service lives longer than 30 years are desired. More corrosion-resistant dowels would also be required.

The field evaluation report (Crovetti 2010, reproduced in Appendix E) shows that the extent of corrosion present did not correlate with the chloride level at the joint face. Also, the type of joint seal present (if any) was noted but it also did not appear to significantly affect the condition of the dowel bar at the joint. The dowel bar deterioration was noted as moderate to extensive for most pavement projects over 25 years of age. There was a significant loss of section of the epoxy-coated dowels in a number of cores. The oldest project in the study (33 years) had an asphalt overlay.

Table 9. Evaluation of epoxy-coated dowel bars on older concrete pavement projects in Wisconsin.

Highway Segment	Pvmt Age	Joint No. - Dowel Pos	FWD Test Results				Dowel Deterioration	Chloride Content % by mass	
			App LT%	App DT*	Lv LT%	Lv DT*		Joint Face	Outside
STH67-Waukesha	33	1-D3	n/a	n/a	n/a	n/a	Extensive*	0.0036	0.0047
		2-D3	n/a	n/a	n/a	n/a	Extensive*	0.0220	0.0140
		3-D3	n/a	n/a	n/a	n/a	Moderate	0.1025	0.1050
		4-D3	n/a	n/a	n/a	n/a	Minor	0.0880	0.0570
I 43-Sheboygan	30	1-D3	n/a	n/a	n/a	n/a	Moderate	0.1050	0.0880
		2-D3	n/a	n/a	n/a	n/a	Minor	0.1500	0.1075
		3-D3	n/a	n/a	n/a	n/a	Extensive	0.0930	0.0920
		4-D3	n/a	n/a	n/a	n/a	Extensive	0.0845	0.1000
		5-D3	n/a	n/a	n/a	n/a	None	0.1300	0.1400
		6-D3	n/a	n/a	n/a	n/a	Extensive	0.1725	0.1125
I 94-Dunn	25	3-D3	87	8.2	n/a	n/a	Extensive	0.1700	0.1475
		4-D3	86	8.9	n/a	n/a	Minor	0.0087	0.2300
		5-D3	87	8.4	n/a	n/a	Moderate	0.1700	0.1450
		40-D3	n/a	n/a	n/a	n/a	Moderate	0.1600	0.2150
		41-D3	n/a	n/a	n/a	n/a	Extensive	0.2150	0.2600
		42-D3	n/a	n/a	n/a	n/a	Moderate	0.2325	0.2450
STH29-Brown	20	S3-Jt4-D3	94	14.4	n/a	n/a	Extensive	0.0520	0.0240
		S3-Jt5-D3	93	14.9	n/a	n/a	None	0.1200	0.0480
		S3-Jt6-D3	90	16.1	n/a	n/a	Moderate	0.0790	0.0480
		S4-Jt1-D3	88	11.5	n/a	n/a	Moderate	0.0695	0.0360
		S4-Jt2-D3	93	13.7	n/a	n/a	Moderate	0.1125	0.0580
		S4-Jt3-D3	95	12.9	n/a	n/a	Minor	0.0720	0.0415
USH18/151-Dane	20	13-Jt1-D10	91	8.2	n/a	n/a	Minor	0.1075	0.1425
		13-Jt1-D3	n/a	n/a	n/a	n/a	Minor	0.1400	0.1300
		13-Jt2-D10	94	13.1	n/a	n/a	None	0.2000	0.1550
		13-Jt2-D3	n/a	n/a	n/a	n/a	Minor	0.2000	0.1650
		13-Jt3-D10	91	7.5	n/a	n/a	None	0.1625	0.1450
		13-JT3-D3	n/a	n/a	n/a	n/a	Minor	0.1300	0.1225
		14-Jt1-D10	91	8.9	n/a	n/a	n/a	0.1625	0.1250
		14-Jt1-D3	n/a	n/a	n/a	n/a	n/a	0.0820	0.1500
		14-Jt2-D10	89	6.4	n/a	n/a	n/a	0.0990	0.1225
STH16-Waukesha	19	1-D3	n/a	n/a	n/a	n/a	Extensive	0.0950	0.0510
		2-D3	n/a	n/a	n/a	n/a	Moderate	0.1075	0.0910
		3-D3	n/a	n/a	n/a	n/a	n/a	0.0940	0.0350
STH29-Chippewa	15	1-D3	n/a	n/a	n/a	n/a	None	0.4000	0.1950
		2-D3	n/a	n/a	n/a	n/a	Minor	0.3125	0.1125
		3-D3	n/a	n/a	n/a	n/a	Moderate	0.3125	0.2000
		4-D3	n/a	n/a	n/a	n/a	Minor	0.3050	0.0910
		5-D3	n/a	n/a	n/a	n/a	Minor	0.4000	0.1500
		6-D3	n/a	n/a	n/a	n/a	Minor	0.3650	0.1500

App DT* = total joint deflection in mils normalized to a load of 9,000 lbs.

n/a indicates core sample where no dowel bar was provided.

CHAPTER 7. SUMMARY AND RECOMMENDATIONS

The recommended evaluation approach emphasizing FWD testing of joints with the alternative dowel bar materials and assessing overall roughness/rideability of the various test sections appears to be a cost-effective process. Due to the relatively short evaluation period, a direct correlation between LTEs measured and overall pavement condition cannot be made, although it is expected that the low LTEs measured for many of the alternative dowel bars will result in faulting and/or slab deterioration in the future. Also, coring of the dowels at selected joints provided valuable supporting information on concrete quality and condition of the dowel bars including extent of corrosion/deterioration. However, the results of testing for the chloride content at the dowel bar level near the crack face and at the outer edges of the core did not correlate with the dowel bar corrosion/deterioration observed.

The performance of 1.5-in (38 mm) FRP dowels with polyester resin and E-glass was not considered acceptable due to the low load transfer efficiency measured at an early age. FRP composite dowels with vinyl ester or epoxy resin and a minimum of 75 to 80 percent ECR-glass and larger bar diameters or closer dowel spacing are needed to improve the LTE and long-term performance. The performance data indicates that 1.5-in (38-mm) diameter FRP composite dowels will not provide satisfactory service compared to the 1.5-in (38-mm) epoxy-coated mild steel dowels used as the control. Other research suggests that 1.75-in (44-mm) diameter vinyl ester resin and ECR-glass dowels may be satisfactory for pavements with strong support (such as an unbonded concrete overlay) while 2.0-in (51-mm) diameter FRP dowels may be necessary for pavements with poor base/subbase/subgrade support conditions in order to achieve longer (say, more than 30 years) of service life. However, improving the quality of the base/subbase/subgrade likely would be more desirable than just increasing the size of the FRP dowels.

Further research will be needed to optimize the design of FRP composite dowels. It is imperative that better documentation of the experimental materials being evaluated be provided so critical design parameters can be identified and related to performance. It is suggested that standard guide specifications for FRP composite dowels (specifying resin type and minimum fiber content) be developed (perhaps by an agency such as the AASHTO materials group) to ensure their long-term performance based on the extensive laboratory testing that has been performed by a number of different agencies.

The evaluation of alternative stainless steel clad dowels and concrete filled stainless steel tubes or pipes (Type 304 or Type 316) was inconclusive due to the small sample and the relatively short (14 years maximum) evaluation period. It appears that they will perform satisfactorily in excess of 30 years given the minimal deterioration noted during this limited evaluation. Evaluation of available accelerated laboratory testing results should be referred to for guidance until extended field testing data are available.

The life of the epoxy coating on mild steel dowels evaluated in Ohio and Wisconsin appears to be in the 25- to 30-year range. A significant number of epoxy-coated dowels examined revealed that the epoxy coating was debonded from the mild steel dowel and the surface of the mild steel dowel under the coating was pitted and rusted. In most cases there was not a significant loss of dowel bar cross section. There was more loss of section due to corrosion in Wisconsin than in Ohio, but no apparent correlation of chloride concentration in the concrete at the dowel bar level and observed corrosion in either state. The extent of early corrosion may be related to the initial quality of the epoxy coating and/or any damage inflicted during their installation. The performance of the epoxy coating appears acceptable for a 30-year design but should not be considered for longer 50-year design life pavements in areas where deicing chemicals or salt water are likely to contribute to excessive corrosion and loss of dowel bar section.

A review of two older projects in Ohio (State Route 81 in Allen County and State Route 59 in Summit County) constructed with plastic-coated dowels indicated that the dowels were in excellent condition after 33 years of traffic. Although the coating was debonded in some instances, the dowel bars themselves showed little deterioration, which was likely aided by the presence of an intermediate protective layer (undercoating) between the plastic and the steel dowel bars. The overall pavement condition of these projects was also very good, with little if any visible joint deterioration.

Because plastic-coated dowels are similar to epoxy-coated dowels in terms of costs, they appear to be a very cost-effective alternative to conventional epoxy-coated dowels. Moreover, the plastic coating is more durable and is less susceptible to damage during handling and construction. In addition, no bond breaker is required which is a definite advantage whether the dowels are inserted or installed in baskets. Additional research and evaluation of these materials is warranted.

CHAPTER 8. IMPLEMENTATION PLAN

Introduction

This chapter presents a recommended implementation plan for the findings and results obtained from this study. The focus of the plan is how the Ohio DOT and other highway agencies can use and apply the project findings in order to achieve better transverse joint performance and ultimately longer-lasting concrete pavements.

Recommendations For Implementation

Key recommendations from this study are summarized below:

- The corrosion potential for dowel bars varies widely throughout the U.S. As a result, state highway agencies are encouraged to conduct an evaluation of the long-term performance of epoxy-coated dowels in their states to determine if their corrosion protection performance (and, by implication, their construction specifications and quality assurance procedures) is cost-effective. As a minimum, improved quality control checks to control holidays and to coat bar ends with epoxy and ensure care is taken during shipping, storage, and installation should be implemented to help provide good, long-term performance. The recommended testing procedures also allow the evaluation of joint load transfer effectiveness including an evaluation of total deflection at the joint (to evaluate the quality of base/subbase/subgrade support), of the quality of concrete at the joint, and of the adequacy of dowel placement tolerances and related quality assurance procedures.
- FRP dowels should be required to have a minimum of 75 to 80 percent E-CR glass (BEL-7 with excellent performance over 28 years had 78 percent E-glass fibers) and be constructed with either vinyl ester or epoxy resin. Based on the limited data of this study, polyester resin FRP dowels do not appear to provide satisfactory LTE performance, but the potential adverse effect of that lower LTE on pavement performance is not clear.
- It is currently too soon to evaluate the acceptability of Type 304 versus Type 316 stainless steel dowel alternatives (solid, clad, mortar filled tubes, or hollow structural pipe). Consequently, the results from accelerated laboratory testing should be relied upon for interim guidance.
- Epoxy-coated dowels appear to provide up to 30 years of service life in a midwestern-U.S. environment (where heavy salting is performed for winter maintenance). However, a number of projects have high loss of section due to corrosion and may have an effective 25 year maximum service life before significant repairs may be needed.
- A limited sample of plastic-coated dowels (twelve dowels conforming to AASHTO M-254 on two projects) has shown outstanding performance after 33 years of traffic and should be considered as a cost-effective alternative to epoxy-coated dowels. Project-level distress ratings should be conducted on similar age plastic-coated and epoxy-coated dowel projects to verify their apparent excellent relative performance.
- Additional long-term performance evaluations of vinyl ester FRP dowels and Type 304 stainless steel dowels are needed to verify their expected long-term performance.

The above recommendations are based on a very limited sample (only OH and WI conducted detailed field evaluations after more than 12 years of traffic). In general, the Ohio FRP polyester dowels performed poorly in terms of LTE, and the poor concrete quality in Wisconsin (plus the poor condition of the coring bit used which undoubtedly damaged the cores) and actual dowel bar placement likely impacted the performance results significantly. It appears that two of the three FRP dowels used in WI 2 were polyester dowels (Crovetti 2006). The results are based on the best data available but are too limited in sample size to produce statistically based findings.

Steps Needed To Implement

Highway agencies that have experimental vinyl ester FRP dowels or Type 304 stainless steel dowels should conduct an evaluation of their performance relative to epoxy-coated dowels generally used as the control. It is also recommended that the AASHTO Materials Group develop a generic specification for vinyl ester FRP dowels to minimize state-to-state or manufacturer variations in the material provided. In particular, the minimum percentage of glass fibers and the type (E-glass or E-CR glass) should be specified along with the resin type.

Given the excellent performance of plastic-coated dowels (AASHTO M 254) in Ohio, it is recommended that states that have used similar materials in the past document their performance and, if satisfactory, consider plastic-coated dowels and vinyl ester FRP dowels as optional standard materials and initiate a test program to evaluate their performance. It is known that Louisiana has over 30 years of satisfactory experience with plastic-coated dowels in areas where the pavement is exposed to salt water.

Highway agencies should conduct a limited FWD testing and coring program similar to that used in Ohio and Wisconsin to not only evaluate the adequacy of long-term corrosion protection of the dowels used, but also to evaluate adequacy of construction procedures and quality assurance procedures to help ensure satisfactory joint performance. However, based on the work done in Ohio and Wisconsin, it appeared that the chloride content did not correlate with visual observations of dowel deterioration and may not be needed.

Suggested Timeframe For Implementation

It is recommended that the above steps be initiated within 2 years and completed within an additional 2-year period thereafter.

Expected Benefits From Implementation

The performance of the transverse joints generally controls and determines the service life of jointed concrete pavements. Steps taken to improve material specifications, construction specifications and quality assurance procedures will pay big dividends in reducing maintenance costs and traffic disruptions during repairs and extending the service life of the pavement. This is particularly important now as many agencies are moving towards long-life, low-maintenance concrete pavement designs, particularly in high-volume urban corridors.

Potential Risks And Obstacles To Implementation

The cost (monetary and personnel) to conduct the evaluations and short term traffic disruptions during FWD testing and coring are restraints. However, by verifying the performance of joints and making any necessary adjustments, the service lives of jointed concrete pavements are expected to be significantly extended and required maintenance activities significantly reduced.

Strategies To Overcome Potential Risks And Obstacles

Testing and coring should be conducted during off-peak periods to minimize traffic disruptions and to minimize danger to testing personnel. Lane closures for the FWD testing and coring should be coordinated to minimize cost and traffic disruptions.

Studies should be initiated to verify the cost-effectiveness of FRP and plastic-coated dowels when revised standards are implemented.

Potential Users And Other Organizations That May Be Affected

Both field testing and laboratory testing personnel will be affected along with required traffic control personnel. Some traffic disruptions will be encountered by users but are expected to be far less than would be needed if poor joint performance significantly requires additional maintenance activities.

Estimated Costs

Costs will vary by agency (whether performed in-house or by contract), by the number of experimental materials being evaluated, and by the mileage and age of jointed concrete pavements in each state. This limited evaluation should pay big dividends in reducing jointed concrete pavement maintenance and extending the overall service life.

Costs for changing to plastic-coated dowels as standard should be minimal compared to the cost of epoxy-coated dowels currently used and the service lives of the pavements should be significantly extended. It is currently unclear what extension of service life is required to justify the higher cost of FRP dowels, but with improved FRP materials, the joint performance is expected to be significantly improved.

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APPENDIX A
SUMMARY OF OHIO FWD DATA

Table A-1. Joint load transfer efficiency data for OH 2 EB.

Joint Number	Station	11/6/1997 RWP				11/15/1999 RWP			
		Temperature = 42º F				Temperature = 38º F			
		Joint Approach		Joint Leave		Joint Approach		Joint Leave	
		LT (%)	Norm Defl*	LT (%)	Norm Defl*	LT (%)	Norm Defl*	LT (%)	Norm Defl*
Standard Epoxy-Coated Steel Dowels - Station 101+95 to 103+20									
1	102+16	87.4	3.60	83.7	3.78	104.2	13.59	99.1	14.04
2	102+38	96.5	4.14	73.7	5.22	103.1	17.73	100.9	18.09
3	102+59	81.3	4.05	82.0	3.78	90.4	15.21	94.4	14.58
4	102+80	86.4	3.24	79.9	3.42	91.4	16.20	97.0	15.39
5	103+02	82.6	3.33	108.5	2.79	86.2	9.54	93.2	8.91
6	103+23	88.4	3.15	111.2	3.24	91.9	11.70	104.1	10.35
Average		87.1	3.59	89.8	3.71	94.5	14.00	98.12	13.56
Grout Filled Stainless Steel Tubes: Station 103+41 to 105+00									
1	103+41	78.3	3.51	74.8	3.78	95.2	15.48	95.7	16.38
2	103+62	78.5	4.32	83.5	4.32	96.2	16.83	103.7	16.20
3	103+83					83.7	16.65	86.8	16.83
4	104+05	79.2	4.05	76.0	4.23	86.9	13.68	97.6	13.14
5	104+26	86.5	3.96	74.3	3.87	88.9	14.04	98.7	13.14
6	104+47								
Average		80.6	3.96	77.2	4.05	90.2	15.34	96.5	15.14
FRP Composite Dowels: Station 106+71 to 107+84									
1	106+79		5.13	69.8	4.86	73.9	18.54	96.4	15.75
2	107+00	88.0	4.32	86.9	4.77	83.7	13.32	89.7	12.51
3	107+22	63.9	5.31	65.7	5.58	86.0	13.23	67.6	15.75
4	107+42	65.7	3.69	59.7	3.33	55.9	15.93	78.7	14.40
5	107+64	60.0	4.32	65.5	4.14	60.6	13.50	92.5	10.53
6	107+85	81.8	6.48	73.4	7.92	85.8	17.10	88.5	16.92
Average		71.9	4.88	70.2	5.10	74.3	15.27	85.6	14.31
Standard Epoxy Coated Steel: Station 108+05									
1	108+06								
2	108+22								
3	108+37								
4	108+53								
5	108+74								
6	108+88								
Average									

* Maximum deflection normalized to 9,000 lb

Table A-1. Joint load transfer efficiency data for OH 2 EB (continued).

Joint Number	Station	8/2/2001 RWP				12/8/2003 RWP			
		Temperature = 86º F				Temperature = 34º F			
		Joint Approach		Joint Leave		Joint Approach		Joint Leave	
		LT (%)	Norm Defl*	LT (%)	Norm Defl*	LT (%)	Norm Defl*	LT (%)	Norm Defl*
Standard Epoxy-Coated Steel Dowels - Station 101+95 to 103+20									
1	102+16					88.0	5.49	80.4	5.85
2	102+38					83.9	4.50	87.8	4.23
3	102+59					93.0	5.85	92.0	6.12
4	102+80					89.8	7.29	92.5	7.29
5	103+02					91.1	6.57	90.3	6.66
6	103+23					91.1	4.23	82.9	4.50
Average						89.5	5.66	87.7	5.78
Grout Filled Stainless Steel Tubes: Station 103+41 to 105+00									
1	103+41	91.1	4.41	90.3	4.41	85.8	3.87	75.8	4.23
2	103+62	86.7	4.68	87.8	4.50	93.7	5.04	83.2	5.58
3	103+83	90.3	4.77			82.9	3.96	79.8	4.14
4	104+05			91.0	4.50	96.7	5.13	90.0	5.49
5	104+26	83.3	6.39	90.7	5.85	82.5	5.67	84.1	5.76
6	104+47	91.6	4.50	94.0	4.32	83.0	6.66	91.5	6.12
Average		88.6	4.95	90.7	4.77	87.4	5.06	84.1	5.22
FRP Composite Dowels: Station 106+71 to 107+84									
1	106+79	59.6	12.24	84.3	9.45	88.3	6.75	88.5	7.20
2	107+00	86.1	4.50	95.7	4.14	81.5	3.87	78.1	4.05
3	107+22	77.6	7.92	70.7	7.92	92.2	4.32	88.9	4.50
4	107+42	69.4	6.12	72.8	5.67	94.0	4.14	84.1	4.68
5	107+64	39.8	9.36	70.0	7.74	28.7	8.37	45.3	10.35
6	107+85	89.0	6.12	88.5	6.30	63.8	7.74	67.2	7.92
Average		70.3	7.71	80.3	6.87	74.8	5.87	75.4	6.45
Standard Epoxy Coated Steel: Station 108+05									
1	108+06	86.4	4.14	88.4	3.96	60.0	7.92	42.3	11.43
2	108+22	83.1	3.69	87.3	3.51	24.7	13.32	44.5	12.78
3	108+37	88.3	5.49	89.5	5.31	28.7	9.36	46.7	8.01
4	108+53	86.6	3.87	88.4	3.69	70.9	12.15	71.0	12.06
5	108+74	89.5	6.03	96.5	5.58	98.6	5.22	87.7	6.03
6	108+88	90.3	5.67	94.4	5.13	93.4	4.68	87.3	5.22
Average		87.4	4.82	90.8	4.53	62.7	8.78	63.3	9.26

* Maximum deflection normalized to 9,000 lb

Table A-1. Joint load transfer efficiency data for OH 2 EB (continued).

Joint Number	Station	5/24/2004 RWP				1/13/2005 RWP			
		Temperature = 82º F				Temperature = 57º F			
		Joint Approach		Joint Leave		Joint Approach		Joint Leave	
		LT (%)	Norm Defl*	LT (%)	Norm Defl*	LT (%)	Norm Defl*	LT (%)	Norm Defl*
Standard Epoxy-Coated Steel Dowels - Station 101+95 to 103+20									
1	102+16	91.8	4.23	89.9	4.23	92.1	6.66	90.5	6.66
2	102+38	91.4	3.87	90.3	3.69	82.4	6.75	87.8	6.30
3	102+59	90.4	4.23	85.7	4.32	78.6	4.86	87.0	2.52
4	102+80	89.0	4.95	83.1	5.13				
5	103+02	90.0	3.87	86.9	3.96				
6	103+23	89.2	3.51	88.4	3.42				
Average		90.3	4.11	87.4	4.13	84.4	6.09	88.4	5.16
Grout Filled Stainless Steel Tubes: Station 103+41 to 105+00									
1	103+41					76.6	4.05	84.7	2.61
2	103+62					76.1	4.32	86.5	2.61
3	103+83					76.7	4.41	86.7	2.70
4	104+05								
5	104+26								
6	104+47								
Average						76.5	4.26	86.0	2.64
FRP Composite Dowels: Station 106+71 to 107+84									
1	106+79	87.8	4.05	87.2	4.05	77.5	3.42	86.2	2.61
2	107+00	92.6	3.15	87.8	3.24	74.4	6.75	82.4	2.70
3	107+22	92.3	3.24	90.6	3.15	75.4	4.59	81.6	3.15
4	107+42	89.8	2.97	85.3	3.06				
5	107+64	84.0	5.85	76.7	6.21				
6	107+85	55.4	4.32	61.0	4.41				
Average		83.7	3.93	81.4	4.02	75.8	4.92	83.4	2.82
Standard Epoxy Coated Steel: Station 108+05									
1	108+06								
2	108+22	80.5	5.40	66.4	6.21				
3	108+37	70.0	5.40	67.9	5.85				
4	108+53	47.1	5.58	54.1	5.76				
5	108+74	91.9	4.77	85.5	5.04				
6	108+88	94.5	3.78	86.0	4.05				
Average		76.8	4.99	72.0	5.38				

* Maximum deflection normalized to 9,000 lb

Table A-1. Joint load transfer efficiency data for OH 2 EB (continued).

Joint Number	Station	12/18/06 RWP Temperature = 49° F Joint Approach LT (%)	12/18/06 RWP Temperature = 49° F Joint Approach Norm Defl*	12/18/06 RWP Temperature = 49° F Joint Leave LT (%)	12/18/06 RWP Temperature = 49° F Joint Leave Norm Defl*	10/29/08 RWP Temperature = 38° F Joint Approach LT (%)	10/29/08 RWP Temperature = 38° F Joint Approach Norm Defl*	10/29/08 RWP Temperature = 38° F Joint Leave LT (%)	10/29/08 RWP Temperature = 38° F Joint Leave Norm Defl*
Standard Epoxy-Coated Steel Dowels - Station 101+95 to 103+20									
1	102+16	86.4	5.76	90.2	5.40	88.9	7.11	89.7	7.02
2	102+38	89.7	6.39	95.2	5.94	93.1	6.30	88.4	6.57
3	102+59	92.5	7.74	89.8	7.74	94.3	9.00	94.6	8.91
4	102+80	87.4	8.91	91.7	8.46	88.9	9.81	92.1	9.81
5	103+02	82.8	5.85	89.5	5.49	91.4	5.40	90.8	5.22
6	103+23	89.7	4.41	89.0	4.32	91.3	4.41	88.4	4.50
Average		88.1	6.51	90.9	6.23	91.3	7.01	90.7	7.01
Grout Filled Stainless Steel Tubes: Station 103+41 to 105+00									
1	103+41	90.3	8.55	87.4	8.82	89.0	8.01	88.0	8.46
2	103+62	50.3	13.23	75.8	11.79	32.0	11.34	68.6	8.28
3	103+83	92.5	7.20	92.8	7.11	91.8	7.38	94.3	7.65
4	104+05	87.9	8.82	88.2	8.73	80.8	10.08	91.2	9.63
5	104+26	96.4	5.40	93.3	5.31	82.1	8.64	89.2	8.55
6	104+47	89.3	8.37	90.5	8.01	96.1	5.58	96.1	5.94
Average		84.5	8.60	88.0	8.30	78.6	8.51	87.9	8.09
FRP Composite Dowels: Station 106+71 to 107+84									
1	106+79	15.3	13.14	34.5	12.33	15.6	12.42	33.5	12.78
2	107+00	45.3	7.47	40.4	8.01	39.9	7.56	43.3	7.20
3	107+22	32.4	11.34	21.8	15.03	25.0	9.18	17.7	14.04
4	107+42	12.0	18.63	16.2	16.11	15.2	12.15	18.2	10.08
5	107+64	27.8	10.35	36.2	9.72	33.2	7.20	30.8	8.55
6	107+85	51.7	13.95	62.0	12.78	40.9	11.70	49.1	10.53
Average		30.8	12.48	35.2	12.33	28.3	10.04	32.1	10.53
Standard Epoxy Coated Steel: Station 108+05									
1	108+06								
2	108+22								
3	108+37								
4	108+53								
5	108+74								
6	108+88								
Average									

* Maximum deflection normalized to 9,000 lb

Table A-1. Joint load transfer efficiency data for OH 2 EB (continued).

Joint Number	Station	10/30/09 RWP Temperature = 63° F				11/03/10 RWP Temperature = 36° F			
		Joint Approach		Joint Leave		Joint Approach		Joint Leave	
		LT (%)	Norm Defl*	LT (%)	Norm Defl*	LT (%)	Norm Defl*	LT (%)	Norm Defl*
Standard Epoxy-Coated Steel Dowels - Station 101+95 to 103+20									
1	102+16	90.1	4.95	89.10	4.86	90.3	7.11	93.2	6.66
2	102+38	95.8	4.95	92.80	4.95	97.4	9.63	96.0	9.72
3	102+59	96.7	6.03	90.60	6.12	97.4	10.53	98.6	10.08
4	102+80	88.8	4.95	85.20	4.95	97.4	17.19	99.8	17.28
5	103+02	95.3	4.68	93.20	4.59	96.5	6.48	94.5	6.57
6	103+23	88.8	3.96	85.70	4.05	93.2	5.58	90.9	5.85
Average		92.6	4.92	89.4	4.92	95.4	9.42	95.5	9.36
Grout Filled Stainless Steel Tubes: Station 103+41 to 105+00									
1	103+41	89.5	4.86	84.5	5.13	98.4	11.70	95.0	12.33
2	103+62	36.4	6.03	57.2	5.40	90.4	19.80	85.7	17.28
3	103+83	97.3	4.68	87.2	5.13	99.1	12.96	96.3	13.50
4	104+05	80.4	5.85	83.2	5.94	96.6	16.74	97.2	16.92
5	104+26	80.6	5.22	81.5	5.13	97.8	18.09	97.8	18.63
6	104+47	98.6	4.05	91.0	4.23	98.9	9.81	97.4	10.08
Average		80.5	5.12	80.8	5.16	96.9	14.85	94.9	14.79
FRP Composite Dowels: Station 106+71 to 107+84									
1	106+79	25.0	7.02	23.6	8.01	15.5	16.74	37.4	17.82
2	107+00	39.7	5.94	37.7	5.76	53.9	10.89	54.2	10.89
3	107+22	26.5	7.20	20.2	9.27	30.4	16.29	33.5	19.89
4	107+42	20.8	8.28	22.4	7.38	13.6	18.36	19.1	15.66
5	107+64	36.0	5.22	21.7	5.85	28.6	9.36	29.3	10.71
6	107+85	39.8	8.19	45.3	8.37	56.4	18.54	65.7	16.65
Average		31.3	6.98	28.5	7.44	33.1	15.03	39.9	15.27
Standard Epoxy Coated Steel: Station 108+05									
1	108+06					97.7	6.93	92.2	7.83
2	108+22					94.9	5.49	95.2	5.58
3	108+37					83.6	6.48	87.5	6.30
4	108+53					94.9	4.50	91.3	4.59
5	108+74					89.6	7.02	98.8	6.75
6	108+88					91.7	8.37	94.5	8.73
Average						92.1	6.47	93.3	6.63

* Maximum deflection normalized to 9,000 lb

Table A-2. Joint load transfer efficiency data for OH 2 WB.

Joint Number	Station	Material	5/24/04 RWP Temperature = 79º F				3/8/2011 RWP Temperature = 34º F			
			Joint Approach		Joint Leave		Joint Approach		Joint Leave	
			LT (%)	Norm Defl*	LT (%)	Norm Defl*	LT (%)	Norm Defl*	LT (%)	Norm Defl*
1	526+00	epoxy					92.6	8.46	92.8	8.28
2	525+83	epoxy					93.4	6.21	93.4	6.12
3	525+66	epoxy					91.4	8.19	88.3	8.28
4	525+49	epoxy					84.7	5.58	87.4	5.31
5	525+32	epoxy					94.8	9.36	101.4	8.82
6	525+15	epoxy					95.8	8.46	95.4	8.46
7	524+98	epoxy	93.6	2.79	87.1	2.88	92.0	5.94	95.7	5.76
8	524+80	epoxy	90.7	2.97	91.3	2.88	70.5	6.03	87.7	4.77
9	524+64	epoxy	93.7	3.42	88.2	3.60	90.4	6.75	96.0	6.30
10	524+47	epoxy	94.5	4.95	85.8	3.51	90.9	6.66	92.0	6.57
11	524+30	?	84.7	3.60	85.2	3.15	87.8	5.94	89.5	5.76
12	524+13	SS clad	84.4	3.42	84.1	2.88	86.6	5.85	95.2	5.31
13	523+96	SS clad	80.1	3.15	89.3	3.24	96.0	7.20	101.6	6.84
14	523+78	SS clad	95.7	3.06	86.3	3.33	89.5	3.42	97.1	3.06
15	523+61	SS clad	92.8	2.52	84.3	2.70	39.2	6.84	58.9	5.31
16	523+44	SS clad					92.9	4.05	96.7	3.78
17	523+27	SS clad					89.4	7.11	101.7	6.39
18	523+10	SS clad					87.2	6.75	98.8	6.03
19	522+93	?					84.7	6.66	98.9	5.67
20	522+76	epoxy					87.7	5.76	100.4	5.04

* Maximum deflection normalized to 9,000 lb

Table A-3. Joint load transfer efficiency for BEL-7 (SR 7, Belmont County, 1983 construction).

Joint Number	Station	Material	3/23/2011 Temperature = 48º F Joint Approach LT (%)	Joint Approach Norm Defl*	Joint Leave LT (%)	Joint Leave Norm Defl*
1	746+25	1.25-in epoxy coated control	60.0	8.34	75.2	7.56
2	746+08	1.25-in epoxy coated control	68.9	11.89	72.9	12.12
3	745+91	1.25-in epoxy coated control	66.6	8.47	60.7	9.52
4	745+74	1.25-in epoxy coated control	64.9	15.79	66.8	14.77
5	745+57	1.25-in epoxy coated control	71.2	10.06	59.5	11.02
6	745+40	1.25-in epoxy coated control	74.0	6.87	73.9	7.33
7	745+23	1.25-in epoxy coated control	62.5	9.82	63.2	9.95
8	745+06	1.25-in epoxy coated control	77.9	6.35	82.0	6.64
9	744+89	1.25-in epoxy coated control	48.1	7.95	44.8	8.39
10	744+72	1.25-in epoxy coated control	60.5	8.51	65.9	7.92
Average			65.5	9.41	66.5	9.52
11	744+55	1.25-in vinyl ester FRP	51.2	7.52	37.9	10.28
12	744+37	1.25-in vinyl ester FRP	49.7	6.44	36.4	8.99
13	744+20	1.25-in vinyl ester FRP	32.9	6.20	32.9	9.11
14	744+03	1.25-in vinyl ester FRP	47.5	4.30	17.4	10.19
15	743+86	1.25-in vinyl ester FRP	42.8	8.02	48.6	6.80
16	743+69	1.25-in vinyl ester FRP	33.6	5.01	37.5	4.31
17	743+52	1.25-in vinyl ester FRP	57.5	5.34	26.5	10.32
18	743+35	1.25-in vinyl ester FRP	37.9	4.53	25.4	6.71
21	743+01	1.25-in vinyl ester FRP	38.0	8.89	43.4	7.73
Average			43.5	6.25	34.0	8.27
19	743+20	1.5-in polyester FRP[1]	16.8	7.08	14.7	10.44
20	743+16	1.5-in polyester FRP[1]	22.1	10.34	23.0	9.89
Average			19.5	8.71	18.9	10.17
22	742+84	1.25-in epoxy coated control	48.0	9.16	61.7	7.84
23	742+67	1.25-in epoxy coated control	61.9	5.48	69.8	4.87
24	742+47	1.25-in epoxy coated control	66.8	5.97	68.7	5.61
25	742+27	1.25-in epoxy coated control	60.3	8.78	65.9	8.23
26	742+06	1.25-in epoxy coated control	69.7	8.73	82.7	7.49
27	741+86	1.25-in epoxy coated control	61.9	7.88	83.2	6.27
28	741+67	1.25-in epoxy coated control	47.9	7.03	111.9	5.08
29	741+47	1.25-in epoxy coated control	48.4	5.82	54.2	5.17
30	741+27	1.25-in epoxy coated control	49.1	8.33	64.0	6.79
31	741+06	1.25-in epoxy coated control	53.7	4.51	60.9	3.90
Average			56.8	7.17	72.3	6.13

* Maximum deflection normalized to 9,000 lb

[1] In 1988 the original vinyl ester FRP joint was removed for laboratory testing and replaced with the full-depth repair

APPENDIX B
SUMMARY OF WISCONSIN FIELD DATA

Table B-1. Evaluation of dowel performance on WI 2.

Highway Segment	Pvmt Age	Joint No. Dowel Pos (Dowel Type)	FWD Test Results				Dowel Deterioration	Chloride Content % by mass	
			App LT%	App DT*	Lv LT%	Lv DT*		Joint Face	Outside
STH 29-Clark	12	CTL-Jt6-D10 (Epoxy)	n/a	n/a	n/a	n/a	Minor	0.1225	0.1600
		CTL-Jt6-D3 (Epoxy)	90	9.4	91	9.2	None	0.0780	0.0820
		CTL-Jt7-D10 (Epoxy)	n/a	n/a	n/a	n/a	Extensive	0.1675	0.1725
		CTL-Jt7-D3 (Epoxy)	87	9.8	80	10.4	None	0.1500	0.1450
		CP-Jt3-D10 (FRP)	n/a	n/a	n/a	n/a	None*	0.1475	0.1075
		CP-Jt3-D3 (FRP)	61	9.6	66	10.0	None*	0.0440	0.0720
		GF-Jt7-D10 (FRP)	n/a	n/a	n/a	n/a	None*	0.1525	0.0900
		GF-Jt7-D3 (FRP)	78	10.5	74	10.7	None*	0.1275	0.0380
		RJ-Jt1-D10 (FRP)	n/a	n/a	n/a	n/a	Minor	0.1475	0.1250
		RJ-Jt1-D3 (FRP)	75	10.6	69	11.0	None*	0.1125	0.0970
		HF-Jt8-D10 (SS Tubes)	n/a	n/a	n/a	n/a	None*	0.0980	0.1350
		HF-Jt8-D3 (SS Tubes)	85	9.7	86	9.5	None*	0.1250	0.0880
		SS-Jt5-D10 (Solid SS)	n/a	n/a	n/a	n/a	None*	0.1275	0.1475
		SS-Jt5-D3 (Solid SS)	50	9.0	60	8.8	None*	0.1075	0.0800

App DT* = total joint deflection in mils normalized to a load of 9,000 lbs.

None* = Fiber Reinforced Polymer Dowel, Extensive* indicates samples where no intact concrete was available.

n/a indicates core sample where no dowel bar was provided.

Table B-2. Evaluation of epoxy-coated dowel bars on older concrete pavement projects in Wisconsin.

Highway Segment	Pvmt Age	Joint No. - Dowel Pos	FWD Test Results				Dowel Deterioration	Chloride Content % by mass	
			App LT%	App DT*	Lv LT%	Lv DT*		Joint Face	Outside
STH67-Waukesha	33	1-D3	n/a	n/a	n/a	n/a	Extensive*	0.0036	0.0047
		2-D3	n/a	n/a	n/a	n/a	Extensive*	0.0220	0.0140
		3-D3	n/a	n/a	n/a	n/a	Moderate	0.1025	0.1050
		4-D3	n/a	n/a	n/a	n/a	Minor	0.0880	0.0570
I 43-Sheboygan	30	1-D3	n/a	n/a	n/a	n/a	Moderate	0.1050	0.0880
		2-D3	n/a	n/a	n/a	n/a	Minor	0.1500	0.1075
		3-D3	n/a	n/a	n/a	n/a	Extensive	0.0930	0.0920
		4-D3	n/a	n/a	n/a	n/a	Extensive	0.0845	0.1000
		5-D3	n/a	n/a	n/a	n/a	None	0.1300	0.1400
		6-D3	n/a	n/a	n/a	n/a	Extensive	0.1725	0.1125
I 94-Dunn	25	3-D3	87	8.2	n/a	n/a	Extensive	0.1700	0.1475
		4-D3	86	8.9	n/a	n/a	Minor	0.0087	0.2300
		5-D3	87	8.4	n/a	n/a	Moderate	0.1700	0.1450
		40-D3	n/a	n/a	n/a	n/a	Moderate	0.1600	0.2150
		41-D3	n/a	n/a	n/a	n/a	Extensive	0.2150	0.2600
		42-D3	n/a	n/a	n/a	n/a	Moderate	0.2325	0.2450
STH29-Brown	20	S3-Jt4-D3	94	14.4	n/a	n/a	Extensive	0.0520	0.0240
		S3-Jt5-D3	93	14.9	n/a	n/a	None	0.1200	0.0480
		S3-Jt6-D3	90	16.1	n/a	n/a	Moderate	0.0790	0.0480
		S4-Jt1-D3	88	11.5	n/a	n/a	Moderate	0.0695	0.0360
		S4-Jt2-D3	93	13.7	n/a	n/a	Moderate	0.1125	0.0580
		S4-Jt3-D3	95	12.9	n/a	n/a	Minor	0.0720	0.0415
USH18/151-Dane	20	13-Jt1-D10	91	8.2	n/a	n/a	Minor	0.1075	0.1425
		13-Jt1-D3	n/a	n/a	n/a	n/a	Minor	0.1400	0.1300
		13-Jt2-D10	94	13.1	n/a	n/a	None	0.2000	0.1550
		13-Jt2-D3	n/a	n/a	n/a	n/a	Minor	0.2000	0.1650
		13-Jt3-D10	91	7.5	n/a	n/a	None	0.1625	0.1450
		13-JT3-D3	n/a	n/a	n/a	n/a	Minor	0.1300	0.1225
		14-Jt1-D10	91	8.9	n/a	n/a	n/a	0.1625	0.1250
		14-Jt1-D3	n/a	n/a	n/a	n/a	n/a	0.0820	0.1500
		14-Jt2-D10	89	6.4	n/a	n/a	n/a	0.0990	0.1225
STH16-Waukesha	19	1-D3	n/a	n/a	n/a	n/a	Extensive	0.0950	0.0510
		2-D3	n/a	n/a	n/a	n/a	Moderate	0.1075	0.0910
		3-D3	n/a	n/a	n/a	n/a	n/a	0.0940	0.0350
STH29-Chippewa	15	1-D3	n/a	n/a	n/a	n/a	None	0.4000	0.1950
		2-D3	n/a	n/a	n/a	n/a	Minor	0.3125	0.1125
		3-D3	n/a	n/a	n/a	n/a	Moderate	0.3125	0.2000
		4-D3	n/a	n/a	n/a	n/a	Minor	0.3050	0.0910
		5-D3	n/a	n/a	n/a	n/a	Minor	0.4000	0.1500
		6-D3	n/a	n/a	n/a	n/a	Minor	0.3650	0.1500

App DT* = total joint deflection in mils normalized to a load of 9,000 lbs.
n/a indicates core sample where no dowel bar was provided.

APPENDIX C
OHIO 2 CORE PHOTOS AND 2001/2004 FWD DATA

Figure C-1. Cores taken from OH 2 (ATH-50). Top: Epoxy-coated dowels; Middle: Stainless steel tubes filled with mortar; Bottom: FRP bars

Figure C-2. Epoxy-coated dowels from OH 2 (ATH-50).

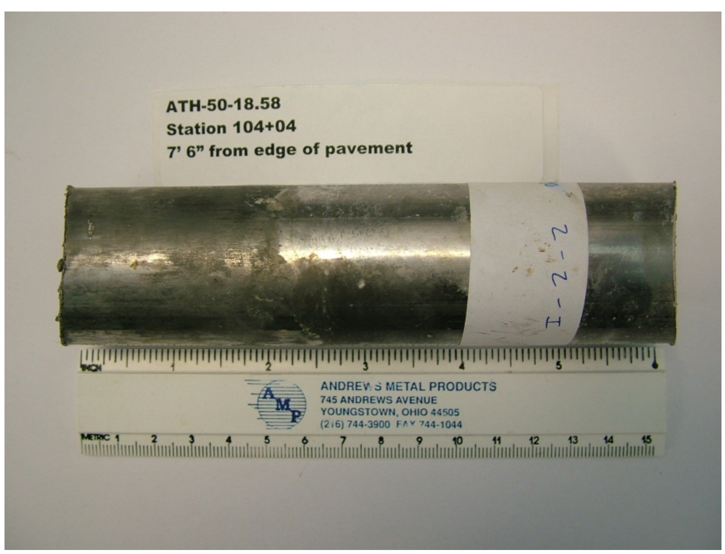

Figure C-3. Stainless steel tubes from OH 2 (ATH-50).

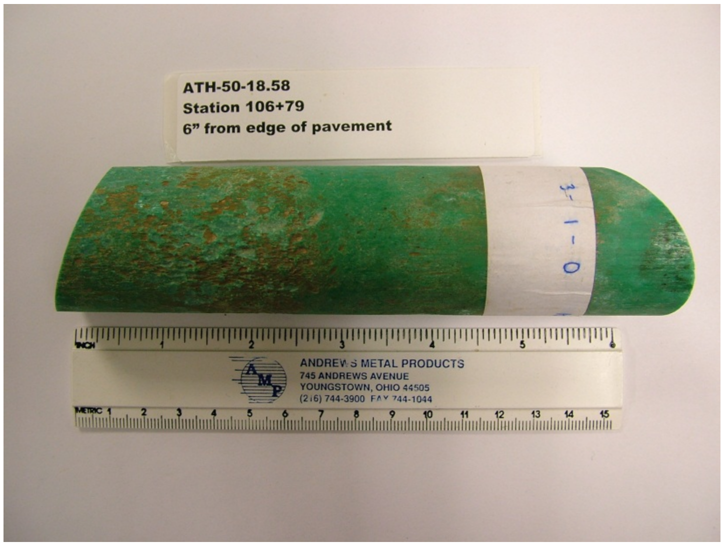

Figure C-4. FRP dowel bars from OH 2 (ATH-50).

Figure C-5. Stainless steel clad dowel bars from OH 2 (ATH-50).

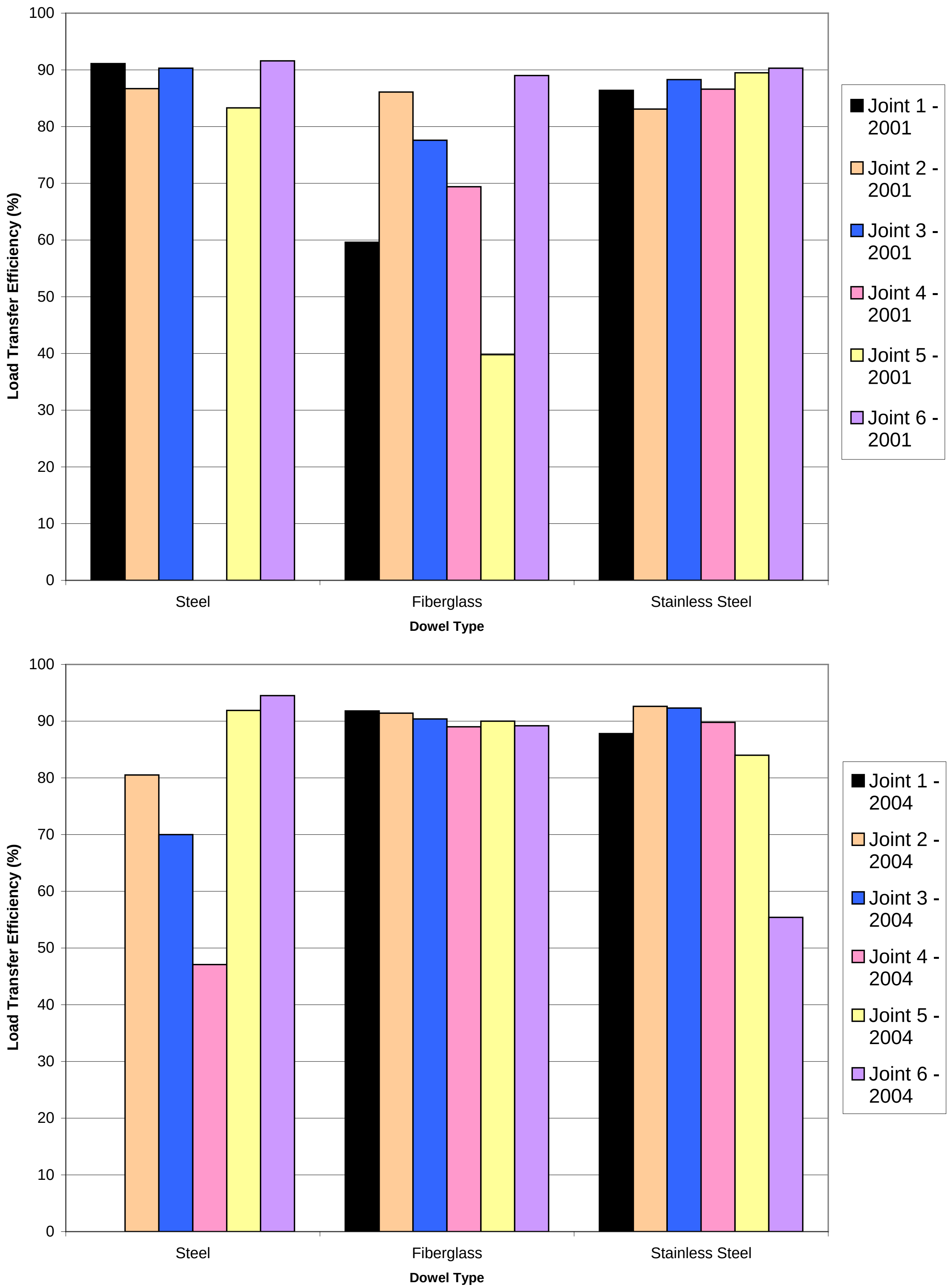

Figure C-6. 2001 and 2004 LTE data for OH 2 (ATH-50).

Table C-1. Summary of core retrieval and chloride analysis on OH 2 (ATH-50).

Cty-Rte	Direction	Pavt Age	Core No.	Station	Dowel Position	Dowel bar material	Dowel Deterioration	Chloride Content joint face	Chloride Content outside
ATH-50	eastbound	14	1-1-O	102+16	6"	epoxy coated steel	Rust stain in epoxy coating at joint location. Coating easily removed. Bar rusted under coating	0.095	0.032
		14	1-1-I	102+16	90"	epoxy coated steel	Rust stain in epoxy coating at joint location. Coating easily removed. Bar rusted under coating	0.055	0.017
		14	1-2-O	103+02	6"	epoxy coated steel	none	0.083	0.024
		14	1-2-I	103+02	90"	epoxy coated steel	"bubble" in epoxy coating along dowel axis. Coating easily removed. Bar rusted under bubble	0.066	0.022
		14	2-1-O	103+41	6"	grout filled stainless steel tube	surface of bar has dull appearance	0.108	0.074
		14	2-1-I	103+41	90"	grout filled stainless steel tube	none	0.082	0.020
		14	2-2-O	104+05	6"	grout filled stainless steel tube	rust stain on top of bar at joint face	0.096	0.041
		14	2-2-I	104+05	90"	grout filled stainless steel tube	none	0.077	0.010
		14	3-1-O	106+79	6"	composite	one end of dowel is rough and pitted	0.159	0.089
		14	3-1-I	106+79	90"	composite	one end of dowel is slightly pitted	0.048	0.016
		14	3-2-O	107+22	6"	composite	one end of dowel is rough and pitted	0.084	0.075
		14	3-2-I	107+22	90"	composite	one end of dowel is rough and pitted	0.053	0.020
		14	4-1-O	108+37	6"	epoxy coated steel	epoxy coated damaged near joint face	0.116	0.060
		14	4-1-I	108+37	90"	epoxy coated steel	epoxy coated damaged near joint face	0.107	0.085
		14	4-2-O	108+88	6"	epoxy coated steel	none	0.114	0.077
	westbound	12		526+00	6"	epoxy coated steel	epoxy coating damaged at joint location. Coating easily removed. Some rust under coating	0.060*	0.018*
		12		526+00	90"	epoxy coated steel	epoxy coating damaged at joint location. Coating easily removed. Some rust under coating		
		12		525+49	6"	epoxy coated steel	epoxy coating in excellent condition	0.052*	0.015*
		12		525+49	90"	epoxy coated steel	epoxy coating damaged at joint location. Coating easily removed. Some rust under coating		
		12		524+13	6"	stainless steel clad	coating in excellent condition with minor corrosion, minor pitting at joint face	0.089*	0.018*
		12		524+13	90"	stainless steel clad	coating in excellent condition with minor corrosion, minor pitting and small rust stains at joint face		
		12		523+62	6"	stainless steel clad	gap around bar on approach side. coating in excellent condition with minor corrosion, minor pitting at joint face	0.081*	0.032*
		12		523+62	90"	stainless steel clad	coating in excellent condition with minor corrosion, minor pitting at joint face		

all dowels are 1 1/2" diameter

stainless steel tube wall thickness is 1/4"

Table C-2. Summary of core retrieval and chloride analysis on ALL-81 and SUM-59 (each containing plastic-coated dowels).

Cty-Rte	Direction	Pavt Age	Core No.	Station	Dowel Position	Dowel Diameter (in)	Dowel Deterioration	Chloride Content joint face	Chloride Content outside
ALL-81	westbound	33	64	451+70	18" from EOP	1 1/4	Coating in good condition with one nick and one split. Some minor rust under the split and nick. Remainder of bar in excellent condition	0.046	0.048
		33	65	452+10	18" from EOP	1 1/4	Coating on leave side of joint appears to have been damaged during installation or placement of concrete. Some rusting and pitting in damaged areas.	0.054	0.035
		33	66	452+50	18" from EOP	1 1/4	Coating in good condition with several area where coating had small splits. Some minor rust under the splits.	0.056	0.040
		33	67	452+89	18" from EOP	1 1/4	Coating in good condition. Some areas on side of bar where coating was abraded and thinned.	0.048	0.031
		33	68	453+29	18" from EOP	1 1/4	Coating damaged in joint area. Elsewhere, coating in fair condition with areas where coating was abraded and thinned. Rusting and pitting in joint area under coating.	0.065	0.021
		33	69	453+69	18" from EOP	1 1/4	Coating in good condition. Some rust stains on coating in joint area. Light rust under coat in stained area.	0.043	0.044
SUM-59	northbound	34	61	180+26	21" from inside EOP	1 1/4	Coating in good condition. Some rust stains on coating in joint area. No rust under coat in stained area.	0.107	0.138
		34	62	180+46	21" from inside EOP	1 1/4	Coating in good condition. Minor rust stain on coating in joint area. Coating in joint area had minor abrading and thinning	0.105	0.114
		34	63	180+86	21" from inside EOP	1 1/4	Coating in good condition. Rust stain and some failure of the coating in joint area. Coating in joint area had minor abrading and thinning. Some rust under coating in stained area	0.130	0.108
		34	58	188+90	21" from inside EOP	1 1/4	Coating in good condition except joint area where coating has failed. Rust and some section loss in the joint area	0.143	0.117
		34	59	189+30	21" from inside EOP	1 1/4	Coating in good condition. Two locations with minor damage in coating	0.173	0.091
		34	60	189+70	21" from inside EOP	1 1/4	Coating in good condition. Two locations with minor damage in coating	0.093	0.105

All bars had a black paste type material, possibly grease or asphalt cement, between the coating and the bar. This material protected the bar from extensive rusting when the coating was split or abraded. Almost all bars had a ridge in the joint area which is probably due to movement of the coating and paste material. Joints were tight.

Table C-3. Summary of core retrieval and chloride analysis on older, epoxy-coated dowels in Ohio.

Cty-Rte	Direction	Pavt Age	Core No.	Station	Dowel Position	Dowel Diameter (in)	Dowel Deterioration	Chloride Content joint face	Chloride Content outside
CUY-176	southbound		1	108+92	wheel path	1 1/2	Rust stain in epoxy coating at joint location. Coating easily removed around stain. Bar pitted under coating	0.063	0.290
			2	108+50	wheel path	1 1/2	Bubbles in epoxy coating above "seam" in bar. Coating easily removed. Bar pitted.	0.065	0.021
			3	108+71	wheel path	1 1/2	None	0.066	0.036
			4	105+98	wheel path	1 1/2	Rust stain in epoxy coating at joint location. Coating easily removed around stain. Bar pitted under coating	0.051	0.030
			5	105+79	wheel path	1 1/2	none. Epoxy coating damage during removal from core.	0.056	0.039
			6	105+58	wheel path	1 1/2	None	0.007	0.006
CUY-176	SB		7	191+95	wheel path	1 1/2	None	0.129	0.129
			8	191+74	wheel path	1 1/2	None	0.059	0.057
SUM-76	eastbound		9	697+65	wheel path	1 1/2	Epoxy coating damaged in joint area. Bar pitted under coating	0.086	0.044
			10	697+44	wheel path	1 1/2	none	0.101	0.055
			11	696+18	wheel path	1 1/2	none	0.123	0.038
			12	697+23	wheel path	1 1/2	None. Epoxy coating not fully bonded to bar.	0.097	0.036
			13	695+76	wheel path	1 1/2	Rust stain in epoxy coating at joint location. Coating easily removed around stain. Bar pitted under coating	0.079	0.045
BEL-7	southbound		14	750+48	42	1 1/4	Epoxy worn to metal on moving end of dowel. Epoxy easily removed. Rust under the epoxy.	0.181	0.077
			15	750+88	30*	1 1/4	Corrosion visible in areas with deteriorated epoxy coating. Coating easily removed. Rust and pitting over much of the bar under epoxy coating	0.186	0.165
			16	750+88	42*	1 1/4	epoxy coating damaged, some possibly due to coring operation. Some epoxy coating easily removed. Areas of rust under epoxy coating.	0.225	0.158
			16	751+28	42	1 1/4	Corrosion visible in areas with deteriorated epoxy coating. Coating easily removed. Rust, pitting and scaling under epoxy in joint area. Some section loss		
			17	754+48	18*	1 1/4	Epoxy missing over most of bar. Bar rusty and pitted, with section loss.		
			17	754+48	30*	1 1/4	Epoxy missing over much of bar. Bar rusty and pitted, with section loss.	0.234	0.206
			18	754+88	30	1 1/4	Rust and corrosion in joint area. Epoxy coating easily removed. Some area of the bar pitted and scaling with some section loss.	0.274	0.248
			19	755+28	30	1 1/4	epoxy deteriorated/missing in joint area. Epoxy coating easily removed. Areas of pitting and scaling with some section loss.	0.244	0.101
BEL-7	southbound		20	746+93	18*	1 1/4	none	0.172	0.128
				746+93	42*	1 1/4	epoxy split, steel exposed at joint face. Epoxy easily removed from bar. Bar pitted in joint area. Some section loss		
			21	746+76	18	1 1/4	epoxy coating split at joint face, some corrosion. Damage to epoxy coating during removal (red circle) Epoxy easily removed, some rust and pitting under epoxy.	0.204	0.155
			22	746+60	18	1 1/4	Epoxy coating worn on one end of dowel. Epoxy coating easily removed on this end. Some rust under epoxy.	0.186	0.178
				746+60	30	1 1/4	epoxy coating split at joint face. Epoxy worn to metal on moving end of dowel. Epoxy easily removed. Rust under the epoxy.		
			32	743+36	42	1 1/4	composite bar, no deterioration	0.166	0.140
			23	742+86	18	1 1/4	epoxy split on one end of dowel, rust stain at the joint face. Epoxy easily removed from that end of dowel. Steel rusted and pitted.	0.183	0.139
			24	742+69	18	1 1/4	epoxy split, steel exposed at joint face. Epoxy easily removed from bar. Bar rusted and pitted in area where metal exposed. Some section loss	0.183	0.132
			25	742+51	18	1 1/4	epoxy split, steel exposed at joint face. Epoxy easily removed from bar. Bar rusted and pitted in area where metal exposed.	0.240	0.135

Table C-3. Summary of core retrieval and chloride analysis on older, epoxy-coated dowels in Ohio (continued).

Cty-Rte	Direction	Pavt Age	Core No.	Station	Dowel Position	Dowel Diameter (in)	Dowel Deterioration	Chloride Content joint face	Chloride Content outside
MOT-35	westbound		26	737+54	wheel path	1 1/4	epoxy damaged at joint face, damaged area rusted. Coating easily removed around rusted area. Bar pitted under coating	0.017	0.011
			27	737+69	wheel path	1 1/4	epoxy damaged at joint face, damaged area rusted	0.048	0.012
			28	737+84	wheel path	1 1/4	none	0.022	0.016
			29	756+00	wheel path	1 1/4	rust stain in epoxy coating at joint face.	0.140	0.045
			30	756+15	wheel path	1 1/4	epoxy missing from bottom half of one end of dowel, steel pitted in area of missing epoxy. Some loss of section.	0.065	0.028
			31	756+30	wheel path	1 1/4	epoxy damaged on one end of dowel	0.064	0.033
CUY-175-7.40	northbound		52	9+89	54"	1 1/4	epoxy in good condition	0.131	0.118
			53	10+29	54"	1 1/4	Epoxy coating abraded and thinned in joint area. Joint area rusted and pitted.	0.246	0.240
			54	10+69	54"	1 1/4	Epoxy coating abraded and thinned in joint area. Joint area rusted and pitted. Most staining is on exterior of epoxy.	0.194	0.187
			55	16+10	54"	1 1/4	epoxy in good condition. Coating damaged during removal from core	0.121	0.095
			56	16+50	54"	1 1/4	Epoxy coating abraded and thinned in joint area. Joint area rusted and pitted. Most staining is on exterior of epoxy. Area with ruptured epoxy coating with rust and pitting under.	0.239	0.232
			57	16+90	54"	1 1/4	Epoxy coating abraded and thinned in joint area. Joint area rusted and pitted. Most staining is on exterior of epoxy. Appears to have been a horizontal bar perpendicular to the dowel bar which has totally rusted.	0.289	0.209

* Due to joint deterioration, two cores were taken. Material removed from both cores as needed to get sufficient material for testing

APPENDIX D
ALTERNATIVE DOWEL BAR PROJECT SUMMARIES

APPENDIX D—Alternative Dowel Bar Project Summaries

(Note: This appendix contains chapters describing alternative dowel bar projects in Illinois, Iowa, Ohio, and Wisconsin, which were taken directly from the FHWA Report "High Performance Concrete Pavements: Summary Report")

CHAPTER 5. ILLINOIS 1 (I-55 SB, Williamsville)

Introduction

This project was the first constructed by the Illinois Department of Transportation (IDOT) to evaluate alternative dowel bars for use in jointed concrete pavements. Constructed in 1996, the project is located on the exit ramp of a weigh station in the southbound direction of I-55 (milepost 107) near Williamsville, just north of Springfield (see figure 2). Although not a TE-30 project, it did serve as a springboard for future IDOT projects evaluating alternative dowel bars under the TE-30 program.

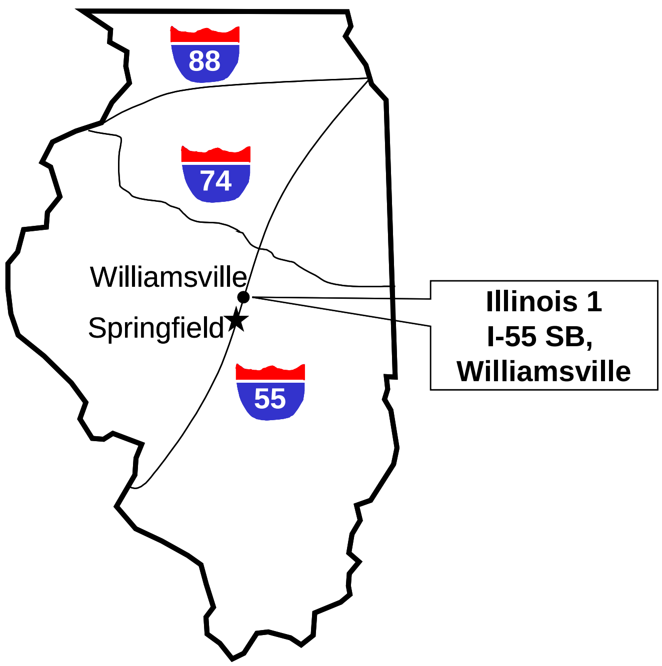

Figure 2. Location of IL 1 project.

Study Objectives

On most concrete pavements, steel dowel bars are used at transverse joints to provide positive load transfer between adjacent slabs. However, even if epoxy coated, these dowel bars are susceptible to corrosion, which can create locked or "frozen" joints that can spall and crack the concrete, significantly reducing the service life of the pavement. The purpose of this study, therefore, is to compare the performance of non-corrosive type 'E' fiberglass and polyester dowels to the performance of conventional epoxy-coated dowel bars in a side-by-side field evaluation project.

Project Design and Layout

This project was constructed in 1996 and consists of a 280-mm (11.25-in) slab placed on a 100-mm (4-in) bituminous aggregate subbase (BAM) (Gawedzinski 2000). In accordance with IDOT practices at the time, the jointed concrete pavement was constructed as a hinge-joint design, in which conventional doweled transverse joints are spaced at 13.7-m (45-ft) intervals and intermediate “hinge” joints containing tie bars are placed at 4.6-m (15-ft) intervals between the doweled joints (see figure 3); this pavement is essentially a jointed reinforced design with the reinforcing steel concentrated at locations where the pavement is expected to crack. The hinge joints contain number 6 epoxy-coated tie bars, 900-mm (36-in) long and placed at 450-mm (18-in) intervals across the joint (Gawedzinski 2000). Preformed compression seals (32-mm [1.25-in] wide) are placed in the doweled transverse joints and a hot-pour joint seal placed in the tied hinge joints (Gawedzinski 2000).

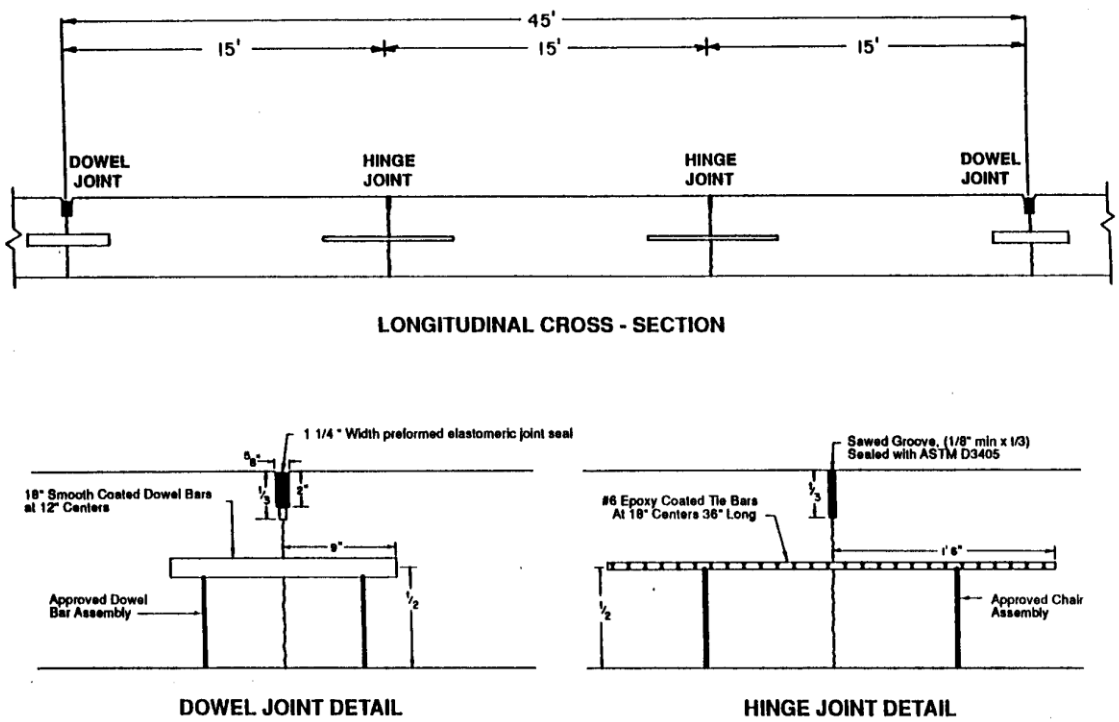

Figure 3. Illinois DOT hinge joint design (IDOT 1989).

The pavement was paved 4.9-m (16-ft) wide, and a 3.0-m (10-ft) tied portland cement concrete (PCC) shoulder was placed adjacent to the mainline exit ramp. The shoulders were tied using number 6 epoxy-coated tie bars, 900-mm (36-in) long and placed at 750-mm (30-mm) intervals (Gawedzinski 2000).

A total of seven joints (excluding hinge joints) are included in the project, the layout of which is shown in figure 4. The first two regular transverse joints of the project contain conventional epoxy-coated steel dowel bars (38-mm [1.5-in] diameter). The next four regular transverse joints contain type ‘E’ fiberglass and polyester bars (38-mm [1.5-in] diameter and 450-mm [18-in] long). The fiberglass and polyester resin bars were manufactured by RJD Industries of Laguna Hills, CA. The final regular transverse joint in the project contains conventional epoxy-coated steel dowel bars.

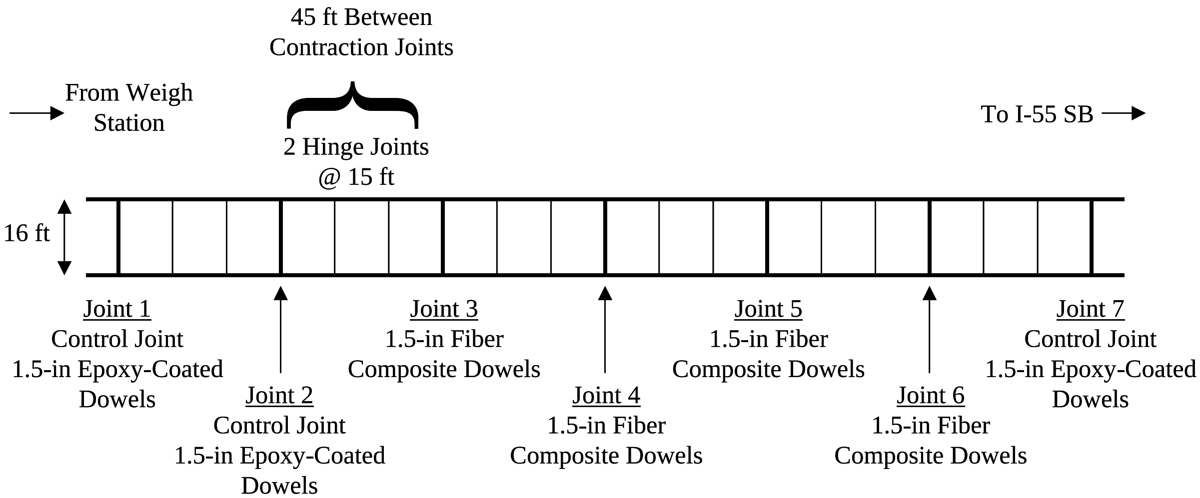

Figure 4. Layout of IL 1 project.

State Monitoring Activities

IDOT collects traffic data from the sorter scale located at the entrance ramp of the weigh station. Traffic totals from the period from September 1996 to September 1999 are summarized in table 2 (Gawedzinski 2000).

Table 2. Traffic data for IL 1 (September 1996 to September 1999) (Gawedzinski 2000).

Truck Type	Number of Vehicles	Accumulated 18-kip ESAL Applications
Single-Unit Trucks	95,623	31,324
Multiple Unit Trucks	1,860,542	3,056,458
TOTALS	1,956,165	3,087,783

All seven joints in the project are evaluated at least semi-annually by IDOT to assess their performance. This evaluation consists of both distress surveys and nondestructive testing using the falling weight deflectometer (FWD). Results from the FWD testing program are plotted in figures 5 and 6 (Gawedzinski 2000). Figure 5 shows the load transfer across each of the seven joints as a function of time, whereas figure 6 shows the maximum joint deflection measured at each joint as a function of time.

A gradual decrease in overall load transfer efficiency is observed in figure 5, with the conventional steel dowel bars consistently showing higher levels of load transfer then the fiber composite bars. But, as seen in figure 6, the largest deflection is consistently shown by one of the conventional doweled joints, although the other two conventional doweled joints show consistently low deflections. However, for both load transfer types, the load transfer efficiency is still relatively high and the magnitude of the joint deflections relatively low.

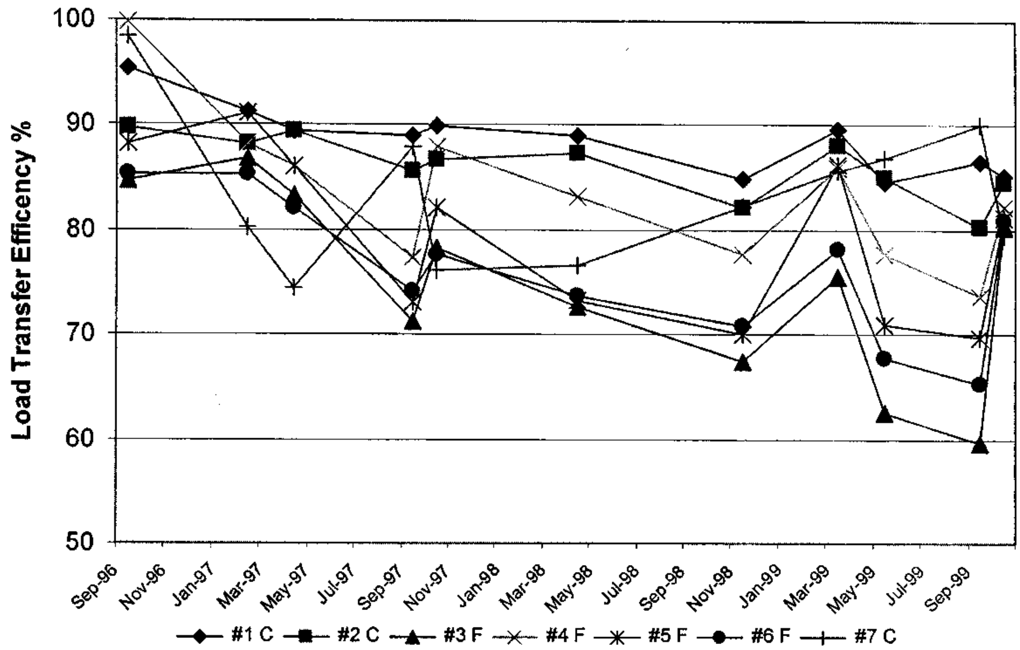

Figure 5. Load transfer efficiency on IL 1 (Gawedzinski 2000).

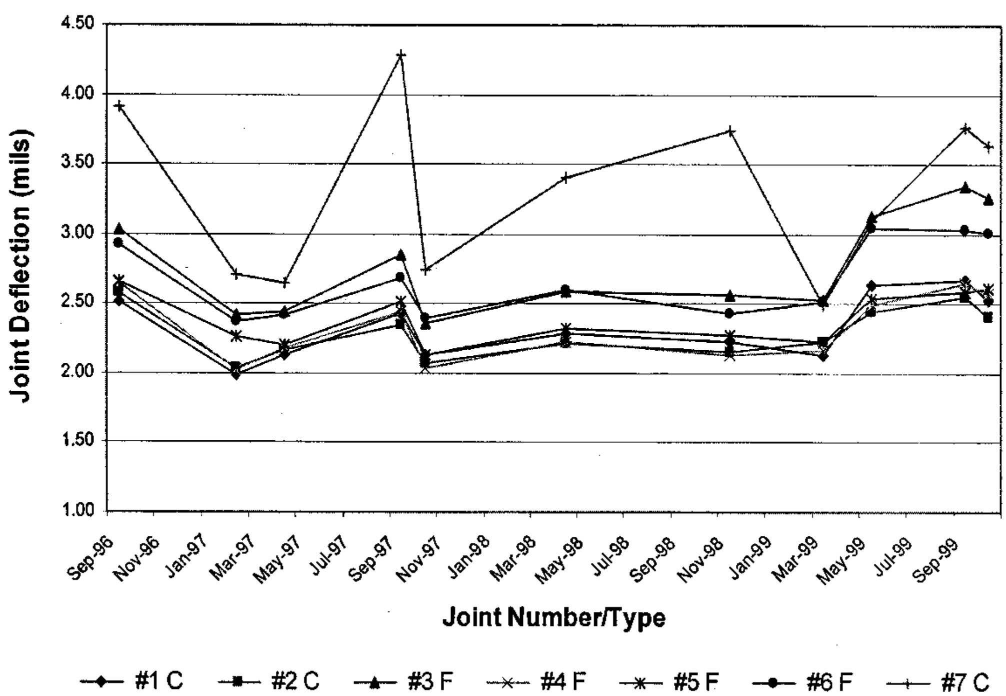

Figure 6. Maximum joint deflections on IL 1 (Gawedzinski 2000).

Preliminary Results/Findings

After about 4 years of service, this project is performing well. None of the joints is exhibiting any signs of distress. IDOT will continue monitoring the project to assess the relative performance of the different dowel bar types.

Interim Project Status, Results & Findings

Truck data continues to be gathered from the sorter scale installed in the entrance ramp of the weigh station. Equivalent Single Axle Loads (ESALs) were computed using scale vendor software and standard IDOT design coefficients. Reported ESAL counts are lower than actual applied ESALs due to the failure of the hard drive on the sorter scale computer for a 13½ month period of time from January 23, 2002 to March 13, 2003. ESAL counts for the missing period of time were projected using the truck data previously gathered from the scale and manual counts obtained from scale operators. Cumulative ESAL estimates are provided in table 3 (Gawedzinski 2004).

Table 3. Cumulative ESALs as of the Day of FWD Testing (Gawedzinski 2004).

Date	Cumulative ESALs
09/26/96	1519.7
2/18/97	292,817.5
4/22/97	485,194.8
9/23/97	1,047,809.7
10/28/97	1,167,329.0
4/27/98	1,637,109.1
11/17/98	2,173,905.1
3/24/99	2,525,120.4
5/13/99	2,719,695.7
9/28/99	3,114,261.8
10/6/99	3,164,730.8
4/13/00	3,710,619.8
6/14/01	5,704,438.6
10/11/01	6,487,023.9
4/17/02	7,551,381.9
10/3/02	8,666,353.0
4/16/03	9,719,309.1
6/11/30	9,841,810.9
10/2/03	10,075,492.5
10/24/03	10,103,714.9

Visual observations of the joints show no obvious signs of pavement distress; neither faulting nor spalling was evident at any of the seven joints. The original construction had the joints sealed with a preformed elastomeric joint seal material compressed into a 5/8" thick saw cut. Over time, the preformed elastomeric joint material has been pushed deeper into the saw cut, especially in the wheel paths. Load Transfer Efficiency Percentage (LTE %) and joint deflection values were determined for each of the seven pavement joints. The average values were determined from deflections measured as simulated 4, 8, and 12 kip loads were applied to the pavement on the approach and leave sides of the joints. The joints were tested at both inner and outer wheel paths and at the center of the lane for a total of 18 tests per joint.

Figure 7 (Gawedzinski 2004) provides a summary of the LTE % versus ESALs, as measured over time. Figure 8 (Gawedzinski 2004) provides a graph of average pavement temperature at a four inch depth verses LTE %.

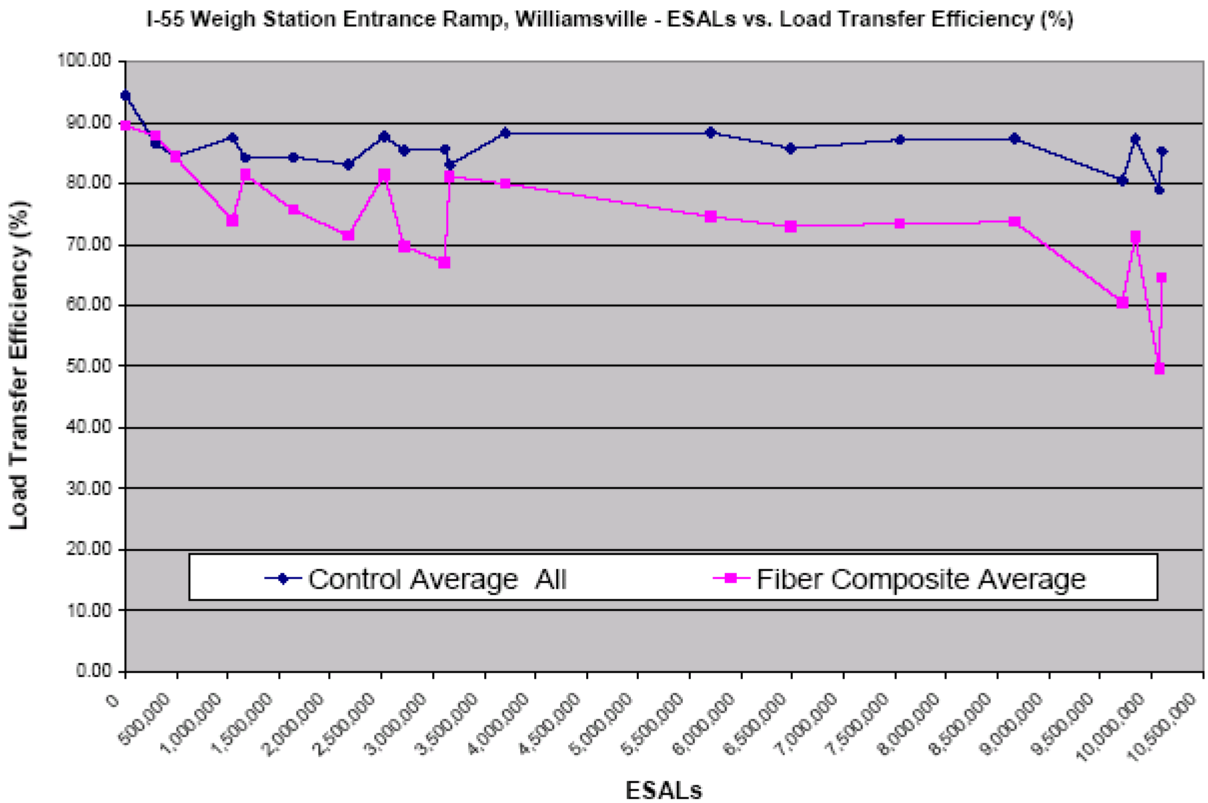

Figure 7. Load Transfer Efficiency vs. ESALs (Gawedzinski 2004).

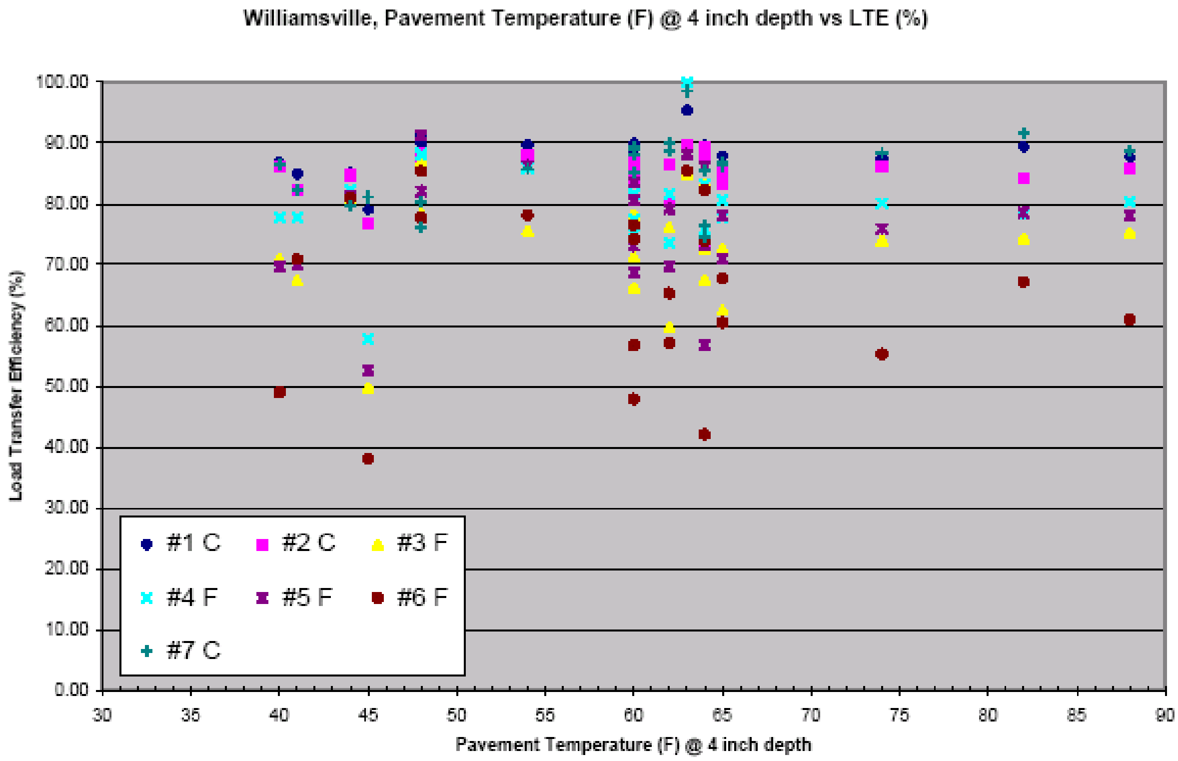

Figure 8. Load Transfer Efficiency vs. Pavement Temperature (Gawedzinski 2004).

Current Observations (Gawedzinski 2004)

Williamsville is the oldest TE-30 test site in Illinois, at 7½ years, and over 10.1 million ESALs. The joints at Williamsville show very little sign of distress or damage. The preformed elastomeric joint seal is still intact showing only that it is deeper in the joints under the wheel paths. Overall, only very minor spalling is displayed at the joints; however, it is not known if this was due to damage during the cutting of the original saw cuts or if it has occurred over time. Evaluation of the FWD data indicate that, on average, the fiber composite dowel bars perform somewhat less effectively than the carbon steel control dowel bars. Graphs showing the individual joint performance show that changes in deflection and LTE% are related to the "overall pavement system" performance, rather than changes in individual joint performance. Dips and spikes in deflection and LTE % are similar to some degree for all of the joints, rather than the joints behaving individually. More frequent FWD testing is planned for the Williamsville site in order to evaluate what causes this response for the bars. Data show LTE% and joint deflection do not appear to be affected by changes in pavement temperature. It is unknown what the moisture content is at the dowel bar/joint interface and how much the moisture content effects LTE% and joint deflections.

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CHAPTER 6. ILLINOIS 2 (Route 59, Naperville)

Introduction

The first TE-30 project constructed in Illinois is located in the southbound lanes of Illinois Route 59 between 75th and 79th Streets, just east of Naperville, a suburb of Chicago (see figure 7). This is IDOT's second project evaluating alternative dowel bar materials, and was constructed in 1997 as part of the reconstruction and widening of Illinois Route 59 (Gawedzinski 2000).

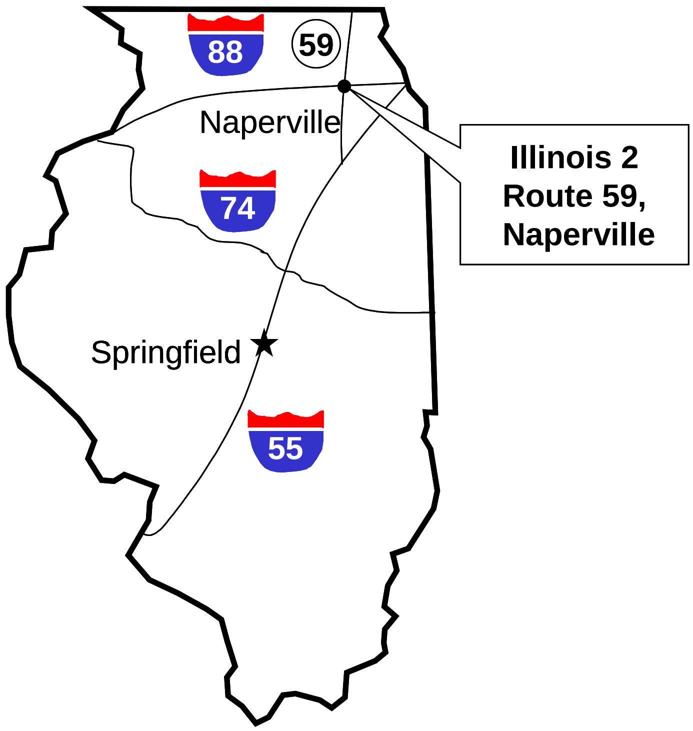

Figure 7. Location of IL 2 project.

Study Objectives

The purpose of this project is to continue IDOT's investigation into alternative dowel bar materials by comparing the performance of IDOT's standard steel dowel bars to several different types of alternative dowel bars (Gawedzinski 2000). This project essentially expands on the IL 1 study by incorporating additional alternative dowel bars from several other manufacturers.

Secondary objectives of the study include an evaluation of different transverse joint reservoir designs and a comparison of different traffic counters. Transverse joint reservoir designs include a standard transverse joint configuration containing preformed joint seals, narrow-width joints containing a hot-poured sealant, and narrow-width joints left unsealed. The traffic counters included in the project are conventional loop detectors/piezo electric axle sensors and a new device that measures traffic-induced changes to the earth's magnetic field (Gawedzinski 2000).

Project Design and Layout

This project was constructed in 1997 and consists of a 255-mm (10-in) slab placed on a 305-mm (12-in) aggregate base course (Gawedzinski 2000). A porous granular embankment subgrade (PGES) material meeting the gradation shown in table 3 is located beneath the aggregate base course (Gawedzinski 1997).

Table 3. Gradation of PGES crushed stone material.

Sieve Size	Percent Passing
150 mm (6 in)	97 ± 3
100 mm (4 in)	90 ± 10
50 mm (2 in)	45 ± 25
75 μm (#200)	5 ± 5

Pavement designs for the experimental sections consist of both hinge-joint designs and all-doweled designs. As described for IL 1, the hinge-joint design contains conventional doweled transverse joints spaced at 13.7-m (45-ft) intervals and intermediate "hinge" joints containing tie bars at 4.6-m (15-ft) intervals between the doweled joints (see figure 3). The hinge joints contain number 6 epoxy-coated tie bars, 900-mm (36-in) long and placed at 450-mm (18-in) intervals across the joint. The all-doweled designs have transverse joints spaced at 4.6-m (15-ft) intervals and contain dowel bars across every joint. The project has three lanes in the southbound direction (total width of 10.8-m [36-ft]), with the inside and center lanes paved together and the outside lane paved later. A tied curb and gutter was placed adjacent to both the inside and outside lanes.

In addition to pavement design, another variable being evaluated under the study is type of load transfer device. The following five load transfer devices are included (Gawedzinski 1997; Gawedzinski 2000):

- Conventional 38-mm (1.5-in) diameter epoxy-coated steel dowel bars conforming to ASTM M227.
- 38-mm (1.5-in) diameter polyester and type E fiberglass dowel bars, manufactured by RJD Industries.
- 44-mm (1.75-in) diameter polyester and type E fiberglass dowel bars, manufactured by RJD Industries.
- 38-mm (1.5-in) diameter polyester and type E fiberglass dowel bars, manufactured by Corrosion Proof Products, Inc.
- 38-mm (1.5-in) diameter epoxy resin and type E fiberglass dowel bars, manufactured by Glasforms, Inc.

Joint width and joint sealant are other variables that are being evaluated under the study. Two of the sections were constructed with 16-mm (0.62-in) wide transverse joints; these were used on the hinge-joint designs only, and were sealed with preformed elastomeric joint seals conforming to AASHTO M220 (Gawedzinski 2000). The other six sections were constructed with narrow 3-mm (0.12-in) transverse joints; five of these were sealed with a hot-poured sealant conforming to ASTM D3405 and one section was left unsealed (Gawedzinski 1997).

The layout of the sections is presented in figure 8. This figure summarizes the main features included in each of the sections. The experimental design matrix for this project is shown in table 4.

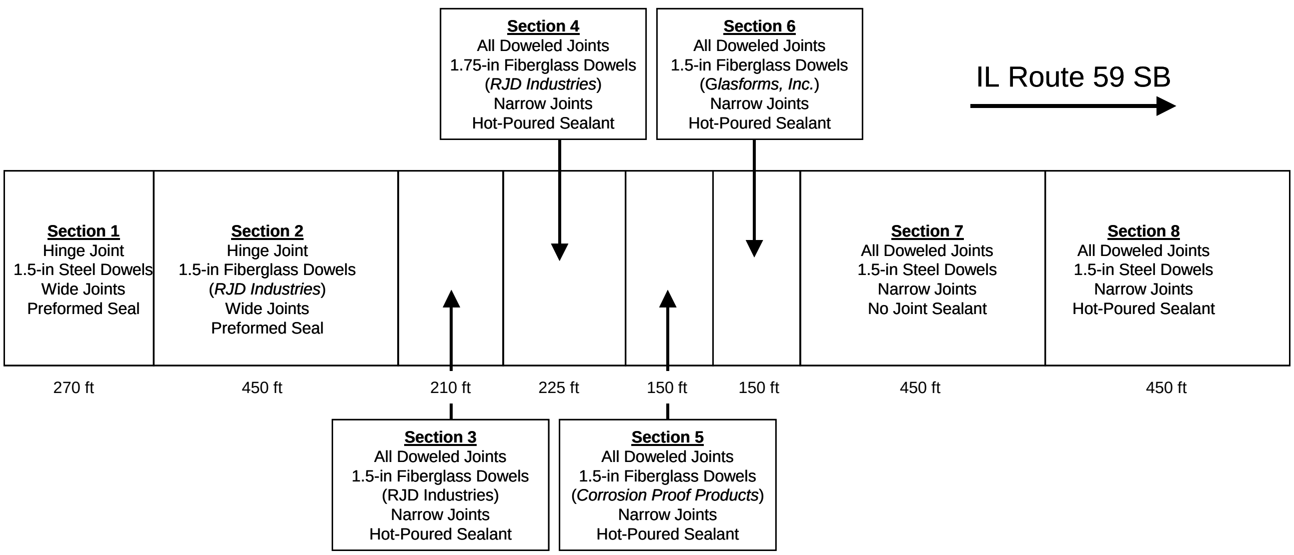

Figure 8. Layout of IL 2 project.

State Monitoring Activities

IDOT collects traffic data for the three southbound lanes and the three northbound lanes using the following devices:

- Peek 241 traffic classifier.
- Nu-Metrics Groundhog® traffic sensors.

The Peek 241 uses traditional traffic loop detectors placed in the subbase, with piezo electric axle sensors installed in channels sawed in the surface of the pavement (Gawedzinski 1997). The Groundhog® uses changes in the earth's magnetic field to classify vehicles, and requires only a 178-mm (7-in) diameter hole cored in the new pavement to install the device. However, problems were encountered with the Groundhog® device and therefore no comparisons between the devices are possible (Gawedzinski 2000).

Table 4. Experimental design matrix for IL 2.

	JRCP Hinge-Joint Design 45-ft Joint Spacing			**JPCP All-Doweled Joints 15-ft Joint Spacing**		
	Preformed Seal (wide joints)	**Hot-Poured Sealant (narrow joints)**	**No Sealant**	**Preformed Seal (wide joints)**	**Hot-Poured Sealant (narrow joints)**	**No Sealant**
38-mm (1.5-in) Epoxy-Coated Steel Dowel Bars	Section 1 (270 ft long, 6 doweled joints)				Section 8 (450 ft long, 30 doweled joints)	Section 7 (450 ft long, 30 doweled joints)
38-mm (1.5-in) Polyester and Type E Fiberglass Dowel Bars (*RJD Industries*)	Section 2 (450 ft long, 10 doweled joints)				Section 3 (210 ft long, 14 doweled joints)	
44-mm (1.75-in) Polyester and Type E Fiberglass Dowel Bars (*RJD Industries*)					Section 4 (225 ft long, 15 doweled joints)	
38-mm (1.5-in) Polyester and Type E Fiberglass Dowel Bars (*Corrosion Proof Products, Inc.*)					Section 5 (150 ft long, 10 doweled joints)	
38-mm (1.5-in) Epoxy-Resin and Type E Fiberglass Dowel Bars (*Glasforms, Inc.*)					Section 6 (150 ft long, 10 doweled joints)	

Traffic data for the three experimental southbound lanes are summarized in table 5 (Gawedzinski 2000). These data are for the period of September 25, 1997 to January 31, 2000. The number of ESALs for each lane was estimated by applying the percentage of vehicles in each lane to the total number of ESALs that were reported for all three traffic lanes (1,515,401).

Table 5. Traffic data for IL 2 (September 25, 1997 to January 31, 2000) (Gawedzinski 2000).

Project Traffic Lane	**Total Number of Vehicles**	**% of All Vehicles**	**Estimated ESALs Based on Vehicle %**
Outside Lane 1	4,687,659	28.6	433,404
Middle Lane 2	6,040,237	36.8	557,668
Center Lane 3	5,689,235	34.6	524,329
TOTALS	16,417,687	100.0	1,515,401

This project is evaluated by IDOT on at least a semi-annual basis. This evaluation consists of both distress surveys and nondestructive testing using the FWD. Results from the FWD testing program are plotted in figures 9 and 10 for sections 1 through 6 only (Gawedzinski 2000). Figure 9 shows the average load transfer for these six test sections as a function of time, whereas figure 10 shows the average maximum joint deflection measured for these six test sections as a function of time.

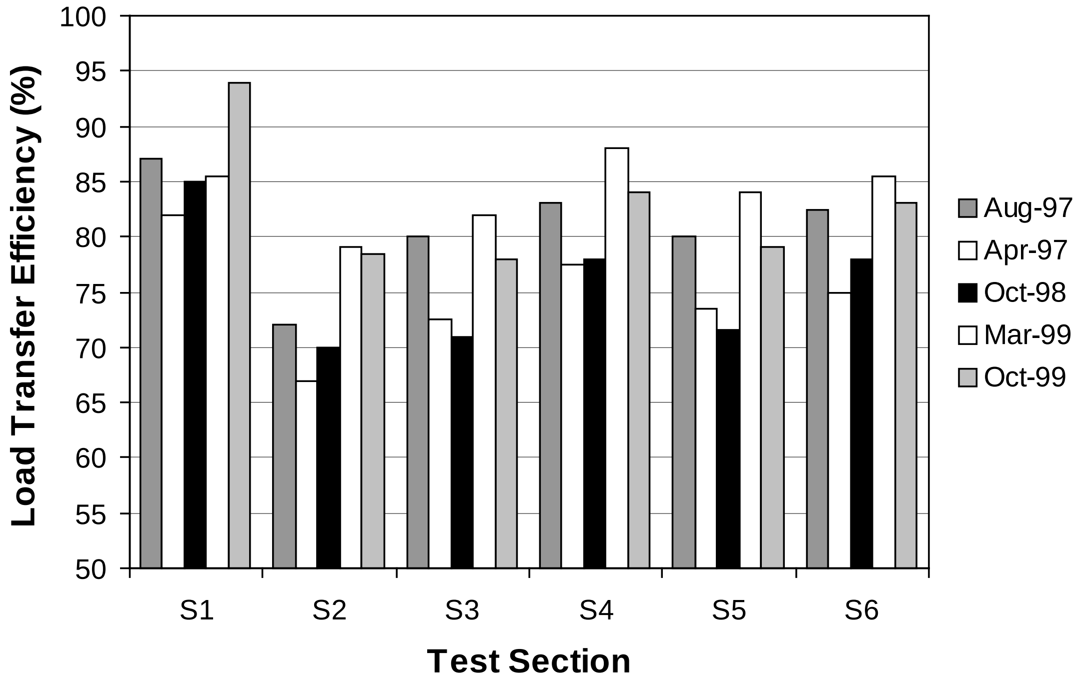

Figure 9. Load transfer efficiency on IL 2 (Gawedzinski 2000).

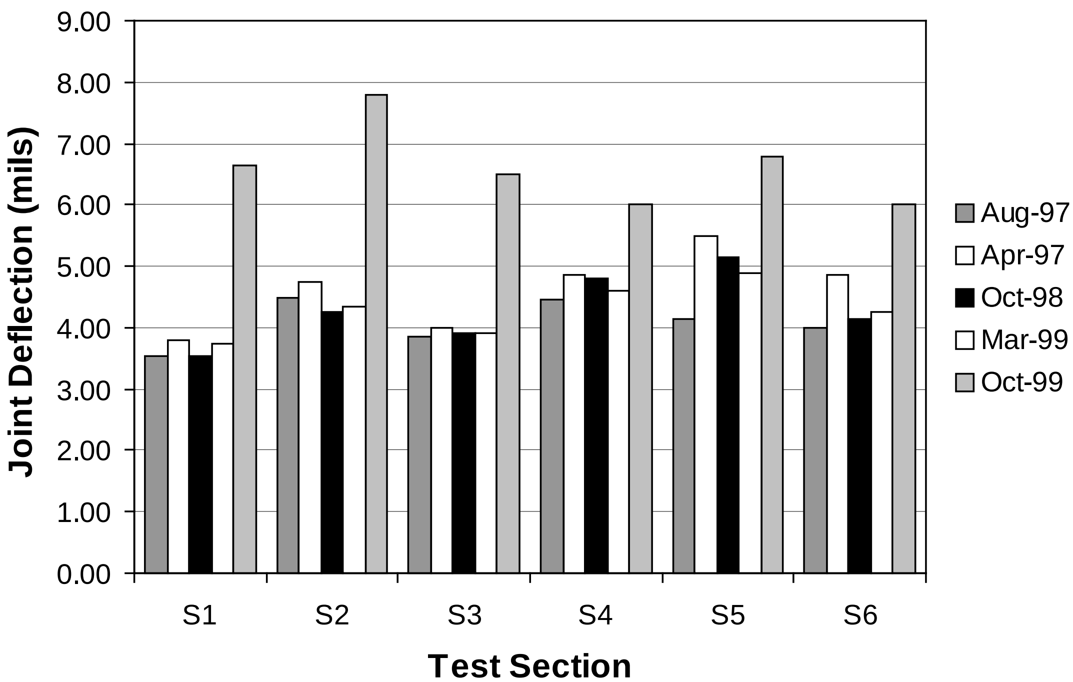

Figure 10. Maximum joint deflections on IL 2 (Gawedzinski 2000).

The best overall load transfer is exhibited by section 1, which contains the conventional steel dowel bars. The other sections all vary from about 70 to 85 percent, but it is interesting to note how the load transfer fluctuates over time, presumably because of the season and the temperature at the time of testing. Figure 10 shows that the maximum deflections for all joints is increasing over time, with the maximum deflection at the most recent testing (October 1999) significantly larger for all six sections than the previous maximum deflection values.

Preliminary Results/Findings

After about 3 years of service, this project is performing well. None of the joints is exhibiting any signs of distress. IDOT will continue monitoring the project to assess the relative performance of the different dowel bar types and of the sealed/unsealed joints.

One issue for consideration in future installations of fiber composite dowel bars is the method used to secure the bar to the basket. During the construction of the middle and inner lanes of this project, it was noted that the fiber composite bars were loose and only partially attached to the upper support wire of the basket (Gawedzinski 1997). A special metal spring clip provided by RJD Industries was ultimately used to secure the dowel bars to the dowel basket and also to provide an additional frictional force to the bar to prevent it from moving as concrete was placed over the basket (Gawedzinski 1997).

Interim Project Status, Results & Findings

Traffic data were obtained using preformed loop detectors and piezo sensors placed in each of the three lanes. The detectors and sensors were wired to a Model 241Traffic Classifier produced by Peek Traffic. In August of 2002, the traffic classifier was replaced with a Road Reporter manufactured by International Traffic Corporation/PAT America, Inc. Daily traffic files are polled periodically and tabulated to provide monthly traffic totals for classification. Standard conversion factors used by the Illinois Department of Transportation are used to convert Single Unit (SU) and Multiple Unit (MU) truck counts to ESALs. In May of 2003, land development work on the properties on the east side of IL 59 resulted in an east-west access road intersecting IL 59 at the location of the traffic classifier loops and piezo sensors. Traffic signals associated with the new road necessitated relocating the traffic classifier site approximately 0.4 miles to the south. Work on relocating the site will be complete in 2004. Cumulative ESAL information for each lane, as reported by the Illinois Department of Transportation (Gawedzinski 2004) are provided in table 6.

FWD tests are currently performed annually across all of the test sections. Certain sections were dropped from the FWD testing for a period of time due to traffic safety issues. These issues were resolved and now FWD results are obtained for both wheel paths and the center of the lane for all three lanes. Visual observations of joint performance are performed periodically, noting any changes in the appearance of the pavement. Results of the FWD tests are provided in figures 11 through 13 for the right, center and left lanes respectively.

Table 6. Traffic data for IL 2 (September 25, 1997 to June 16, 2003) (Gawedzinski 2004).

Date	Cumulative ESALs		
	Right Lane	Center Lane	Left Lane
8/25/97	1,751	4,288	1,008
4/6/98	73,677	146,779	33,118
10/19/98	160,540	306,559	71,363
3/29/99	210,187	412,343	95,277
10/13/99	319,964	614,230	141,165
4/24/00	393,299	761,761	173,867
10/16/00	480,678	909,423	212,076
5/15/01	560,141	981,053	280,037
5/1/02	661,433	1,110,816	326,719
6/16/03	728,208	1,249,667	357,084

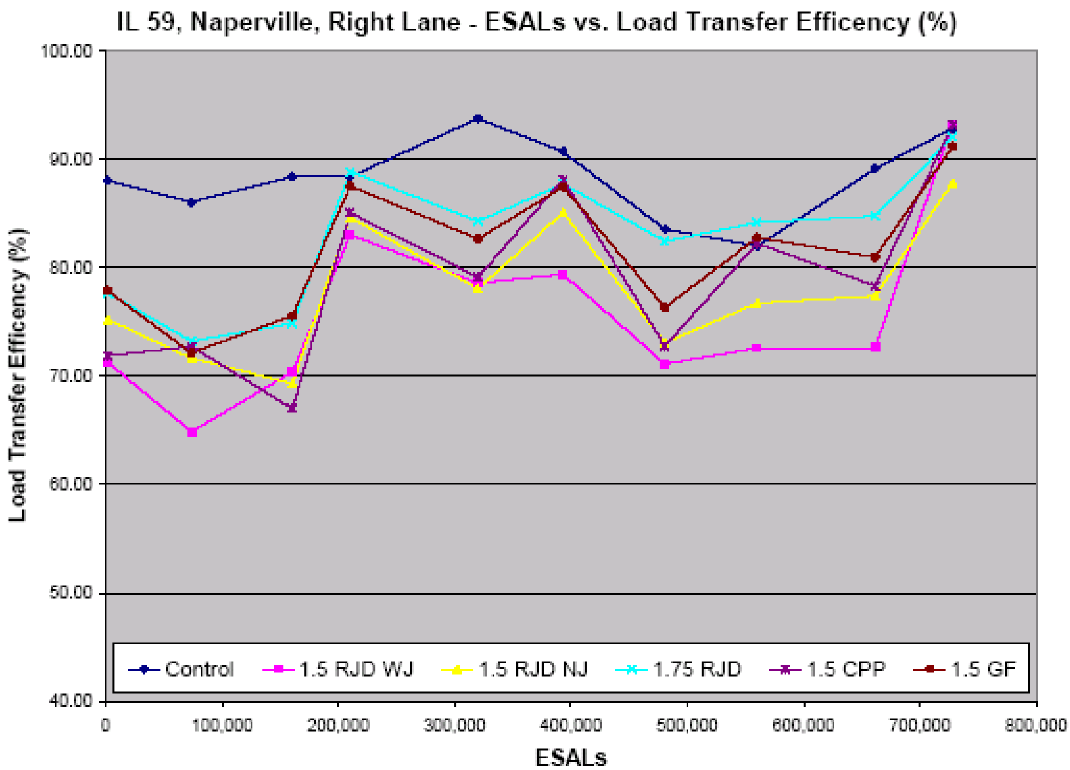

Figure 11. Load Transfer Efficiency vs. ESALs for the Right Lane (Gawedzinski 2004).

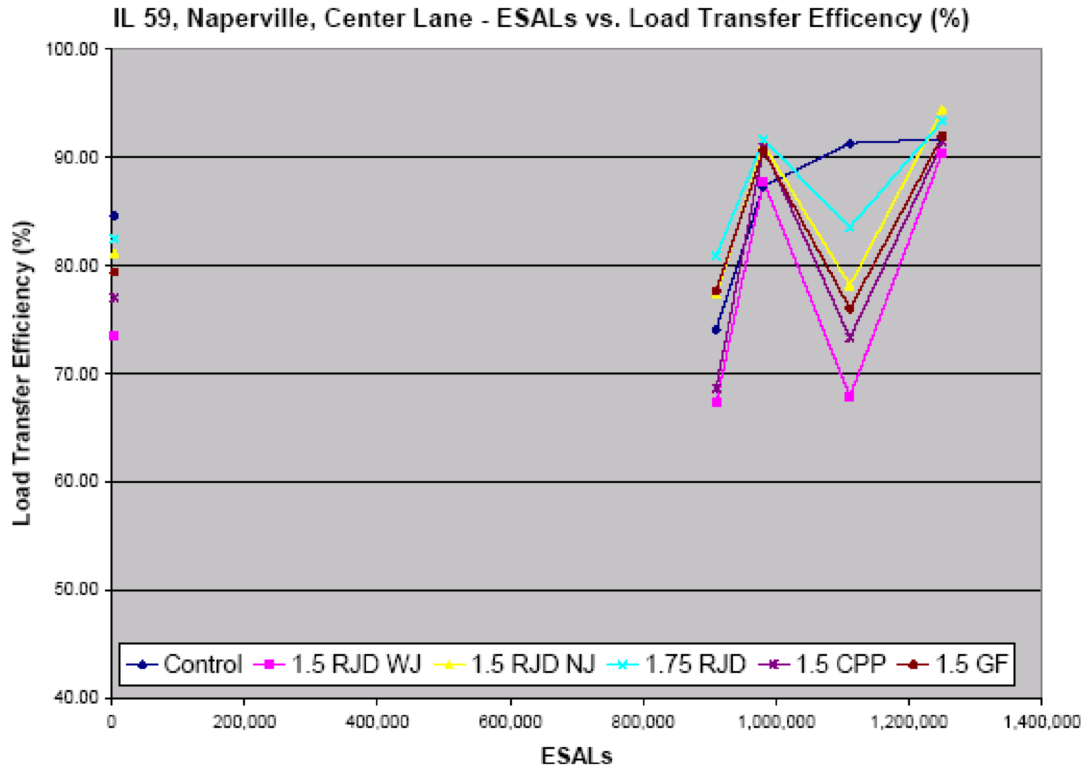

Figure 12. Load Transfer Efficiency vs. ESALs for the Center Lane (Gawedzinski 2004).

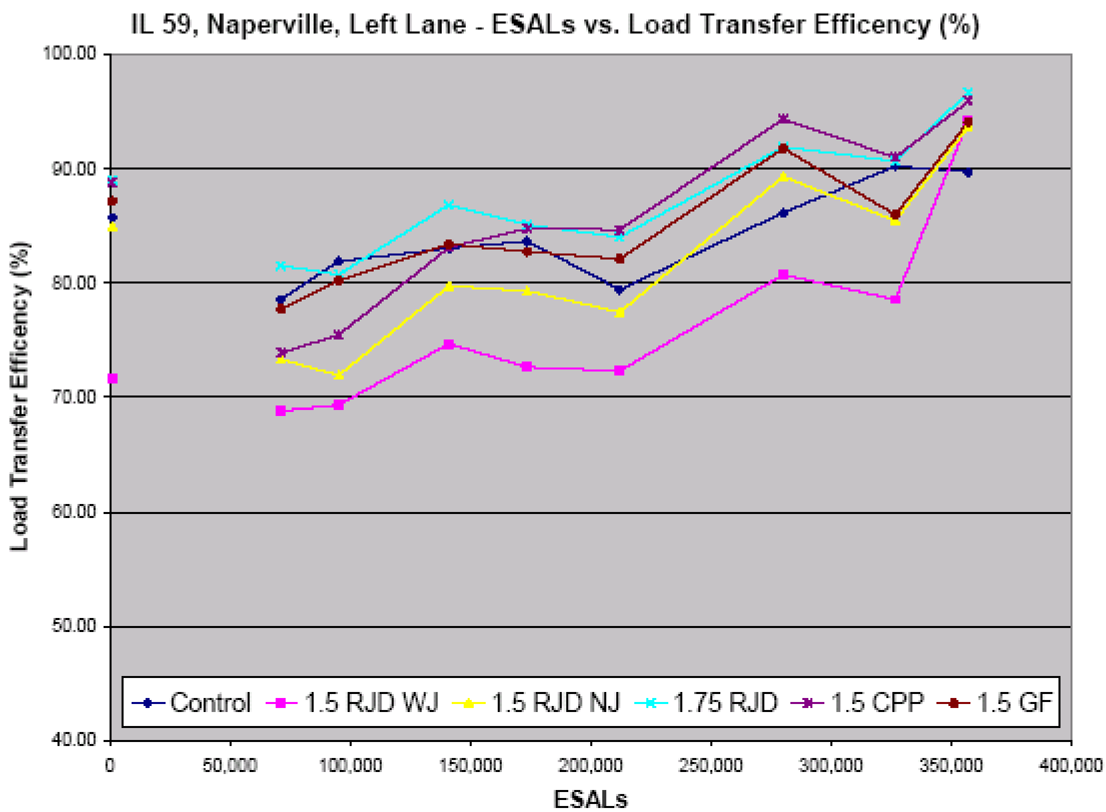

Figure 13. Load Transfer Efficiency vs. ESALs for the Left Lane (Gawedzinski 2004).

Current Observations (Gawedzinski 2004)

Evaluation of the joints shows typical behavior of the joints and the joint sealer/filler material with no obvious signs of spalling or faulting. The preformed elastomeric joint sealer remains intact, while the ASTM D-6690 (formerly ASTM D-3405) material is acting more as joint filler in that there are areas across several joints where the material has become debonded from the pavement, allowing water and incompressibles into the joint.

Observations of the LTE% vs. time and ESALs graphs, as well as the joint deflection vs. time and ESALs graphs, show somewhat consistent behavior for joint deflection, with sections averaging between 3 to 5 mils. LTE% graphs show behavior consistent with a decrease in joint deflection. Figure 14 shows the same type of behavior displayed at the Williamsville, IL test site (Illinois 1). Plots of average values show no relationship between LTE% or joint deflection and average pavement temperature. The control bars (1½" Ø epoxy coated carbon steel) have a higher LTE% and lower joint deflection than any of the fiber composites, but the overall performance of the fiber composite bars appears to be very close to the behavior of the epoxy coated steel control set.

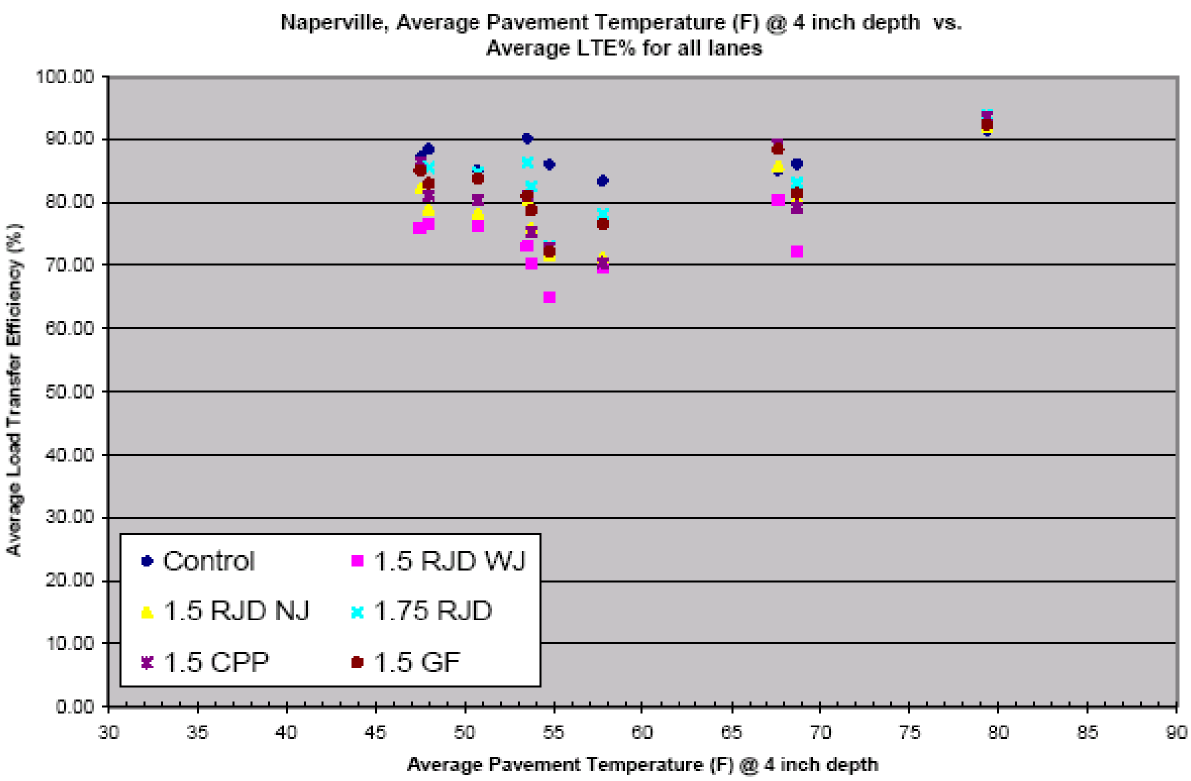

Figure 14. Average Load Transfer Efficiency vs. Pavement Temperature for all Lanes (Gawedzinski 2004).

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CHAPTER 7. ILLINOIS 3 (U.S. Route 67, Jacksonville)

Introduction

IDOT's second TE-30 project, and their third evaluating alternative dowel bar materials, is located on the two westbound lanes of U.S. Route 67, west of Jacksonville (see figure 11). This project was constructed in 1999.

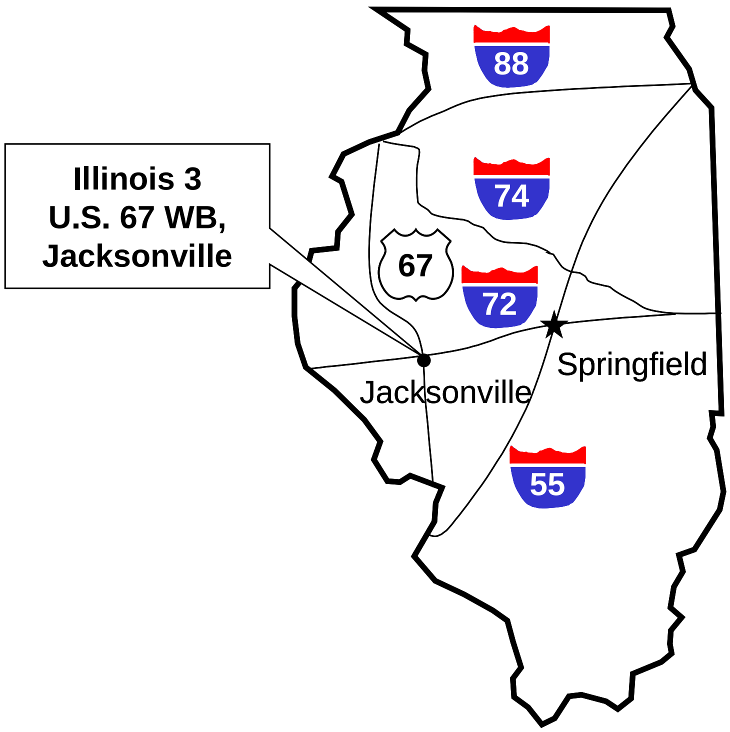

Figure 11. Location of IL 3 project.

Study Objectives

This project continues IDOT's investigation of alternative dowel bar materials and joint sealing effectiveness (Gawedzinski 2000). Several additional fiber composite dowel bars are evaluated in this study that were not included in previous studies, and these comparisons are all done using IDOT's now standard all-doweled jointed plain concrete pavement (JPCP) design. In addition, an unsealed section is included to further investigate the performance of unsealed joints.

Project Design and Layout

Constructed in 1999, the basic pavement design for each section is a 250-mm (10-in) thick JPCP placed on a 100-mm (4-in) cement aggregate mixture (CAM) base course (Gawedzinski 2000). The existing subgrade was stabilized to a depth of 300 mm (11.8 in) with lime (Gawedzinski 2000). Transverse joints are spaced at 4.6-m (15-ft) intervals and tied concrete shoulders are incorporated as part of the construction project.

The project consists of seven test sections evaluating alternative dowel bar materials and unsealed joints. The following load transfer devices are included in the study (Gawedzinski 2000):

- 38-mm (1.5-in) diameter polyester and type E fiberglass dowel bars, manufactured by RJD Industries.
- 38-mm (1.5-in) diameter vinyl ester and type E fiberglass dowel bars, manufactured by Strongwell (Morrison Molded Fiber Glass Company).
- 38-mm (1.5-in) diameter vinyl ester and type E fiberglass dowel bars, manufactured by Creative Pultrusions, Inc.
- Fiber-Con™ dowel bar, manufactured by Concrete Systems, Inc. and consisting of a fibrillated type E fiberglass and polyester resin tube filled with hydraulic cement.
- 38-mm (1.5-in) diameter carbon steel rods clad with grade 316 stainless steel, manufactured by Stelax Industries Inc.
- Conventional 38-mm (1.5-in) diameter epoxy-coated steel dowel bars conforming to ASTM M227.

All but one of the sections was sealed with a hot-poured joint sealant conforming to ASTM D 3405. One section was left unsealed to compare the performance of pavements with unsealed joints to that of sealed joints.

The layout of the sections is presented in figure 12. This figure summarizes the main features included in each of the sections. The experimental design matrix for this project is shown in table 6.

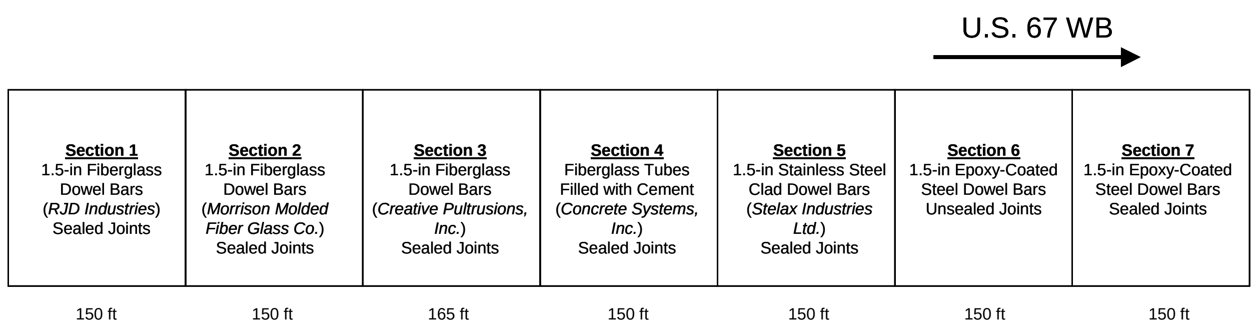

Figure 12. Layout of IL 3 project.

State Monitoring Activities

IDOT installed an automatic traffic recording station at the project site in February 2000. Traffic data are recorded using a Peek series 3000 ADR traffic classifier (Gawedzinski 2000). No traffic data are currently available.

Table 6. Experimental design matrix for IL 3.

	250-mm (10-in) JPCP 4.6-m (15-ft) Joint Spacing	
	Sealed Joints (ASTM D3405)	**Unsealed Joints**
38-mm (1.5-in) diameter polyester and type E fiberglass dowel bars (*RJD Industries*)	Section 1 (150 ft long, 10 joints)	
38-mm (1.5-in) diameter vinyl ester and type E fiberglass dowel bars (*Morrison Molded Fiber Glass Company*)	Section 2 (150 ft long, 10 joints)	
38-mm (1.5-in) diameter vinyl ester and type E fiberglass dowel bars (*Creative Pultrusions, Inc*.)	Section 3 (150 ft long, 11 joints)	
Fiber-Con™ dowel bar, consisting of a fibrillated type E fiberglass and polyester resin tube filled with hydraulic cement (*Concrete Systems, Inc*.)	Section 4 (150 ft long, 10 joints)	
38-mm (1.5-in) diameter carbon steel rods clad with grade 316 stainless steel (*Stelax Industries Inc*.)	Section 5 (150 ft long, 10 joints)	
38-mm (1.5-in) diameter epoxy-coated steel dowel bars	Section 7 (150 ft long, 10 joints)	Section 6 (150 ft long, 10 joints)

Before the pavement was opened to traffic, IDOT conducted FWD testing on the experimental sections in June 1999. Results from the FWD testing program are plotted in figures 13 and 14 (Gawedzinski 2000). Figure 13 shows the average load transfer for the seven experimental sections in both the driving and passing lanes, whereas figure 14 shows the average maximum joint deflection measured for each of the seven experimental sections in both the driving and passing lanes. Although the joint deflections are low, the load transfer efficiencies are not as high as might be expected for a new concrete pavement. These initial FWD results will serve as a baseline for comparison with future testing values.

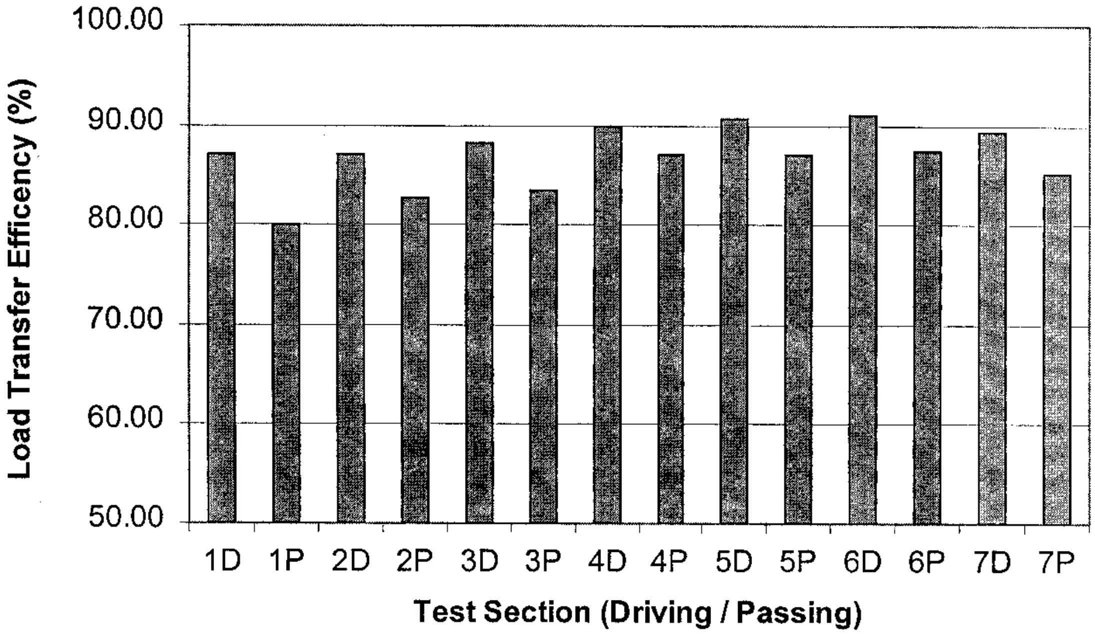

Figure 13. Load transfer efficiency on IL 3 (Gawedzinski 2000).

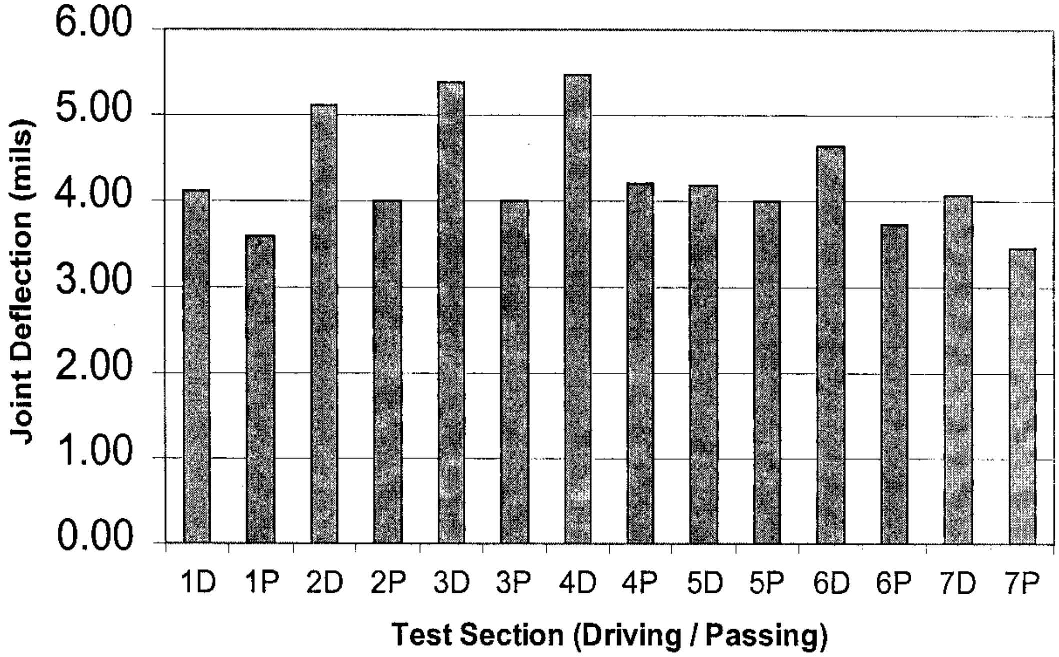

Figure 14. Maximum joint deflections on IL 3 (Gawedzinski 2000).

Preliminary Results/Findings

This pavement is performing well after 1 year of service. None of the joints are exhibiting any signs of distress. IDOT will continue monitoring the project to assess the relative performance of the different dowel bar types and of the sealed/unsealed joints.

Interim Project Status, Results & Findings

FWD tests are conducted semi-annually along with periodic visual observations of joint performance. Traffic data is collected using an ADR 3000, manufactured by Peek Traffic. The data is periodically polled and converted to ESALs using standard IDOT conversion factors. A summary of the cumulative ESALs is provided in table 7.

Joints are also periodically observed, to look for signs of joint deterioration or distress. Joints were formed using a thin saw cut and sealed with an ASTM D 6690 (formerly ASTM D 3405) hot pour joint seal material. Problems affecting ride quality became apparent, due to several of the joints being overfilled with the 3405 joint seal material. Subsequent evaluations noted failure of the 3405 joint seal material to maintain a bond with either side of the pavement at the joint.

Table 7. Current Traffic for Driving and Passing Lanes (Gawedzinski 2004).

Date	Cumulative ESALs	
	Driving Lane	Passing Lane
6/23/99	0	0
6/27/00	68,604	9,7420
10/10/00	95,413	13,764
4/18/01	160,805	22,940
10/11/01	240,558	34,305
4/18/02	310,034	43,193
10/01/02	372,800	48,871
4/16/03	442,221	54,892
10/21/03	493,053	59,488
11/25/03	504,163	

Current Observations (Gawedzinski 2004)

Several joints were observed where the joint seal material was either missing from the wheel paths, or had been pushed deeper in the joint and was debonded from both sides of the pavement joint. A large amount of small rocks were also compressed into the joint seal material at the joint surface. As with the other sites (IL 1 & IL 2), no obvious signs of joint distress were apparent during the visual observations.

Similar behavior as observed at the older two sites (IL 1 & IL2) is shown in the following figures. The control set (1½” Ø epoxy coated steel), unsealed epoxy coated steel bars, stainless steel clad carbon steel bars, and fibrillated wound fiber composite bars exhibit better LTE and lower joint deflections than the pultruded fiber composite bars, but do not show excessive joint deflection indicating failure of the joints. Pavement at Jacksonville (IL 3) was constructed on a cement aggregate mixture subbase (CAM2 w/ a minimum of 200 lbs of cement per cubic yard) rather than a granular subbase as in Naperville (IL 2) or a bituminous aggregate mixture subbase (BAM) at Williamsville (IL 1).

An additional FWD test was performed on the driving lane of US 67 in November of 2003 to evaluate the joint deflections which had occurred earlier that year. Testing was not conducted in the passing lanes due to traffic control problems at the time of the November tests. The large shift in average joint deflection vales between the April and October tests necessitated the November retest. More frequent testing is scheduled for 2004.

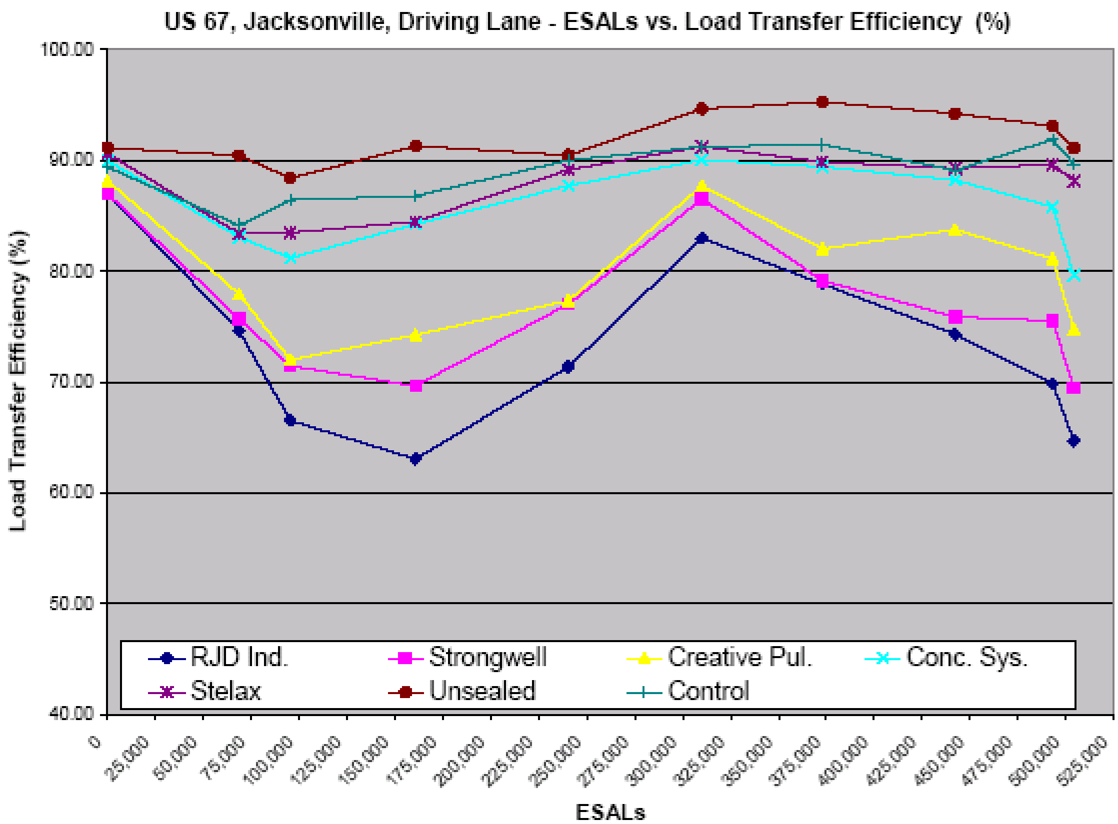

Figure 15. Driving Lane Load Transfer Efficiency vs. ESALs (Gawedzinski 2004).

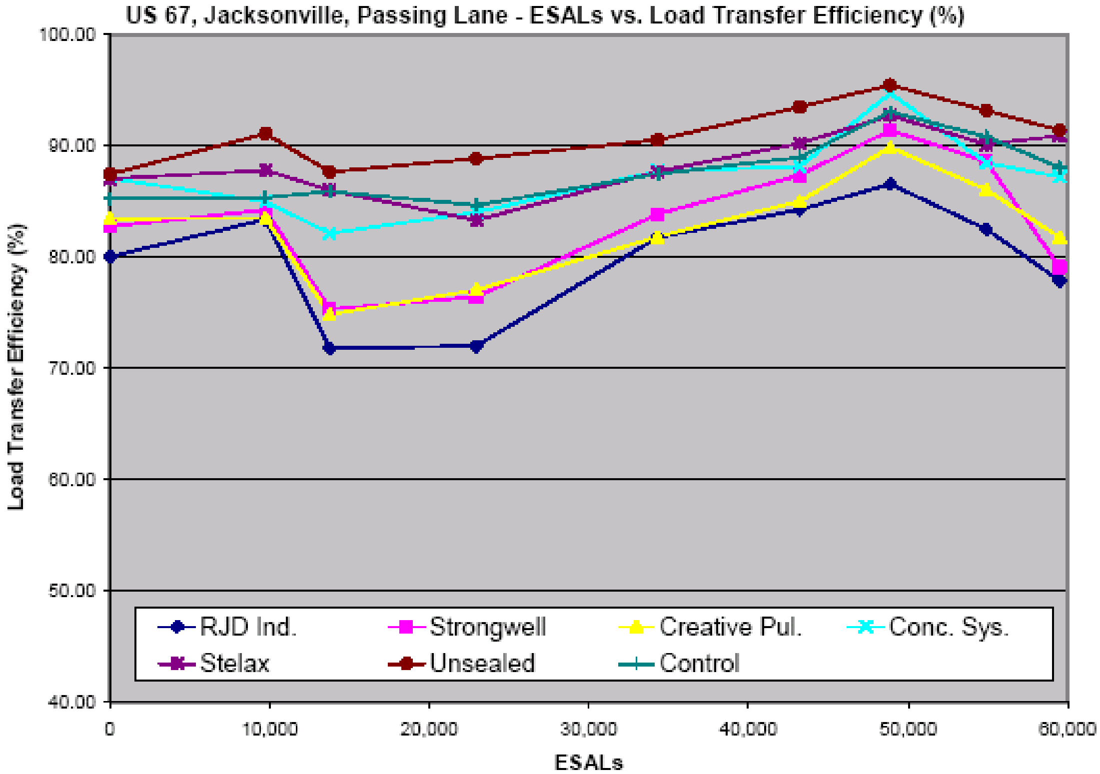

Figure 16. Passing Lane Load Transfer Efficiency vs. ESALs (Gawedzinski 2004).

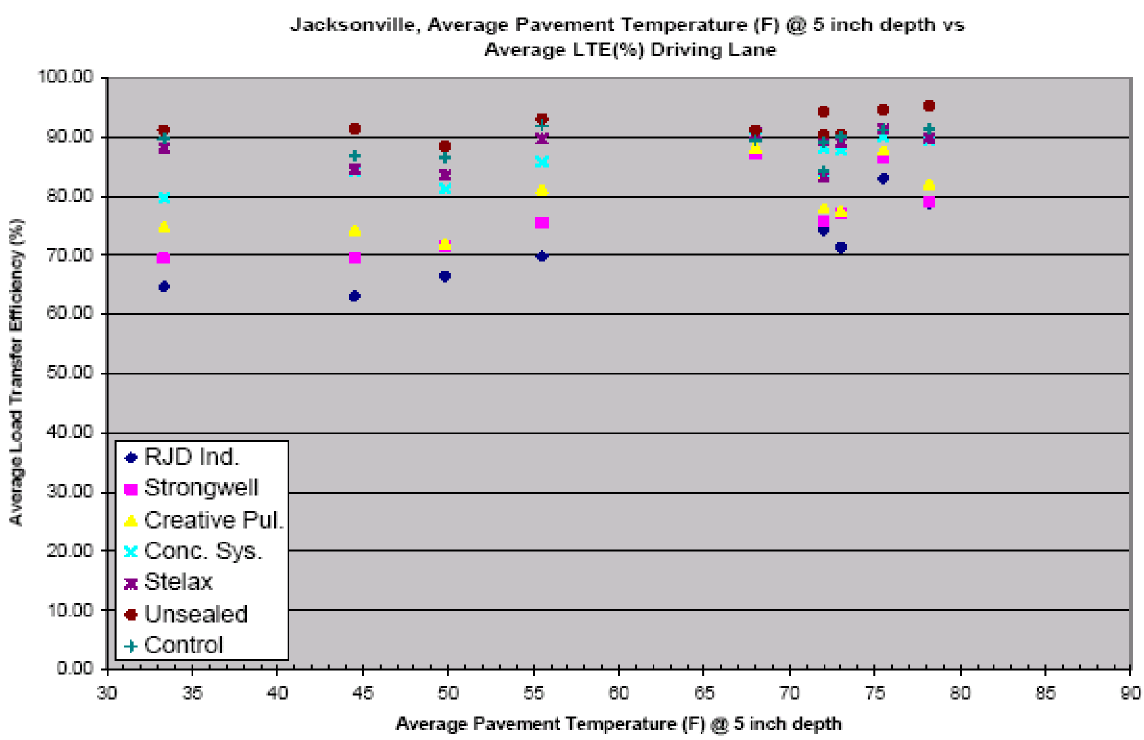

Figure 17. Average Load Transfer Efficiency vs. Average Pavement Temperature (Gawedzinski 2004).

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Gawedzinski, M. 2000. *TE-30 High Performance Rigid Pavements Illinois Project Review*. Illinois Department of Transportation, Springfield, IL.

Gawedzinski, M. 2004. *TE-30 High Performance Concrete Pavements: An Update of Illinois Projects*. Illinois Department of Transportation, Springfield, IL.

CHAPTER 8. ILLINOIS 4 (Route 2, Dixon)

Introduction

A fourth project evaluating alternative dowel bars was constructed by IDOT in the April 2000. The experimental project is located in the driving lane of the northbound direction of Illinois Route 2 in Dixon (see figure 15) where it replaces an existing concrete pavement (Gawedzinski 2000).

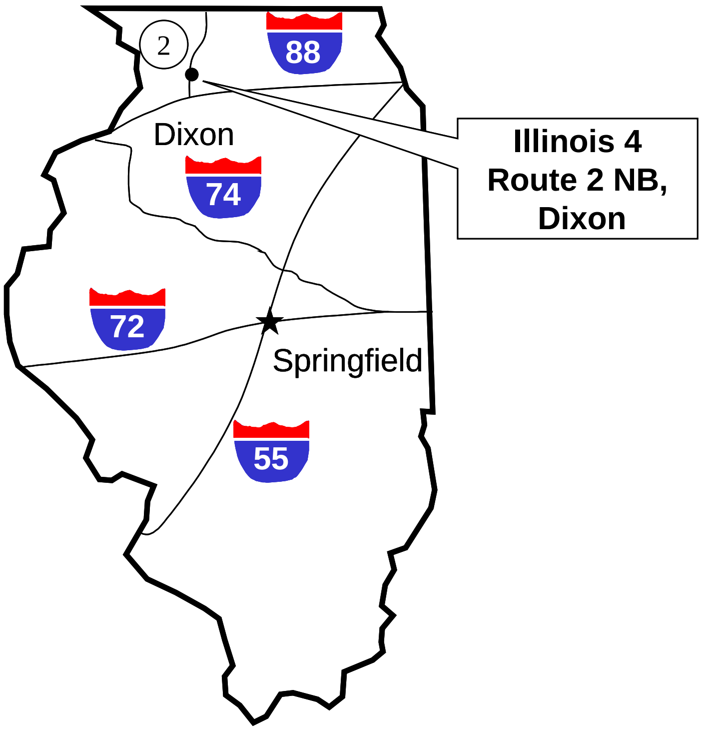

Figure 15. Location of IL 4 project.

Study Objectives

Although not an official TE-30 project, this project carries on IDOT's investigation of alternative dowel bar materials. The alternative dowel bar materials used in the project included stainless steel tubes filled with cement grout, stainless steel clad carbon steel tubes, and fiber composite tubes filled with cement grout. Two different diameters, 38-mm (1.5 in) and 44.5-mm (1.75 in), were used for the stainless steel tubes and for the stainless steel clad dowels. The fiber composite tubes were formed using a pultrusion process and were approximately 50-mm (2 in) in diameter. The pultrusion process produced a much smoother bar, compared to the first generation, fibrillated bars. Additionally two different methods of securing the bars to the baskets, welding and using cable ties, were used in the four sections. Additional construction details are presented in the literature.

Project Design and Layout

The pavement design for each section is a 240-mm (9.5-in) doweled JPCP placed over a 300-mm (12-in) granular base course (Gawedzinski 2000). Transverse joints are spaced at 4.6-m (15-ft) intervals and are sealed with a hot-poured sealant. A tied curb and gutter is placed adjacent to the outer driving lane of the project.

The experimental project consists of five test sections evaluating the following alternative dowel bar materials (Gawedzinski 2000):

- Fiber-Con™ dowel bar, manufactured by Concrete Systems, Inc. and consisting of a pultruded fiber composite tube composed of type 'E' fiberglass and polyester resin and filled with hydraulic cement.
- 38-mm (1.5-in) diameter, 2.76 mm (0.109 in) thick grade 316 stainless steel tube filled with cement grout.
- 44.5-mm (1.75-in) diameter, 2.76 mm (0.109 in) thick grade 316 stainless steel tube filled with cement grout.
- 38-mm (1.5-in) diameter carbon steel rods clad with grade 316 stainless steel, manufactured by Stelax Industries Inc.
- 44.5-mm (1.75-in) diameter carbon steel rods clad with grade 316 stainless steel, manufactured by Stelax Industries Inc.

Conventional load transfer devices are installed in JPCP sections adjacent to the experimental pavement sections.

State Monitoring Activities

Traffic data will be recorded using a Peek series 3000 ADR traffic classifier. IDOT obtained baseline FWD deflection data after the pavement was constructed and will monitor its performance on at least a semi-annual basis.

Interim Project Status, Results & Findings

Data has been collected on a semi-annual basis for the past three years. The cumulative ESALs are provided in table 12. Results of deflection testing are illustrated in the following figures.

Table 12. Data Collection Date and Cumulative ESALs (Gawedzinski 2004).

Date	Cumulative ESALs
8/1/00	0
5/1/01	20,780
10/1/01	50,036
4/25/02	62,701
10/2/02	76,872
4/3/03	93,982
10/3/03	125,533

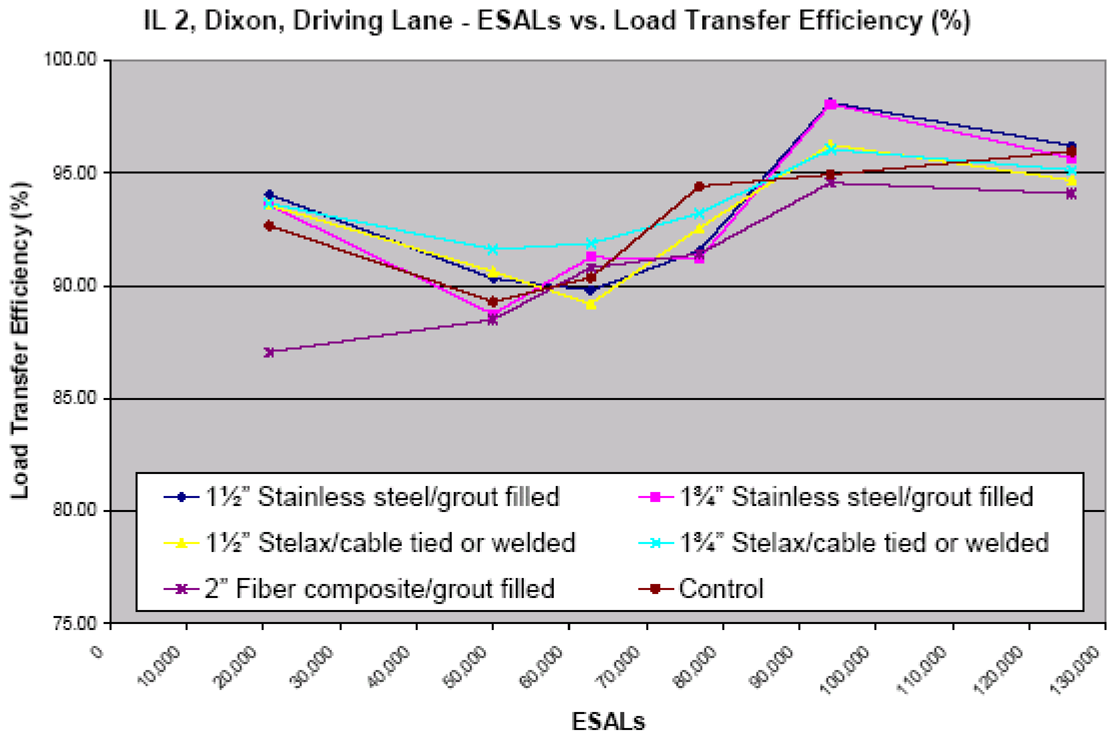

Figure 16. Driving Lane Load Transfer Efficiency vs. ESALs (Gawedzinski 2004).

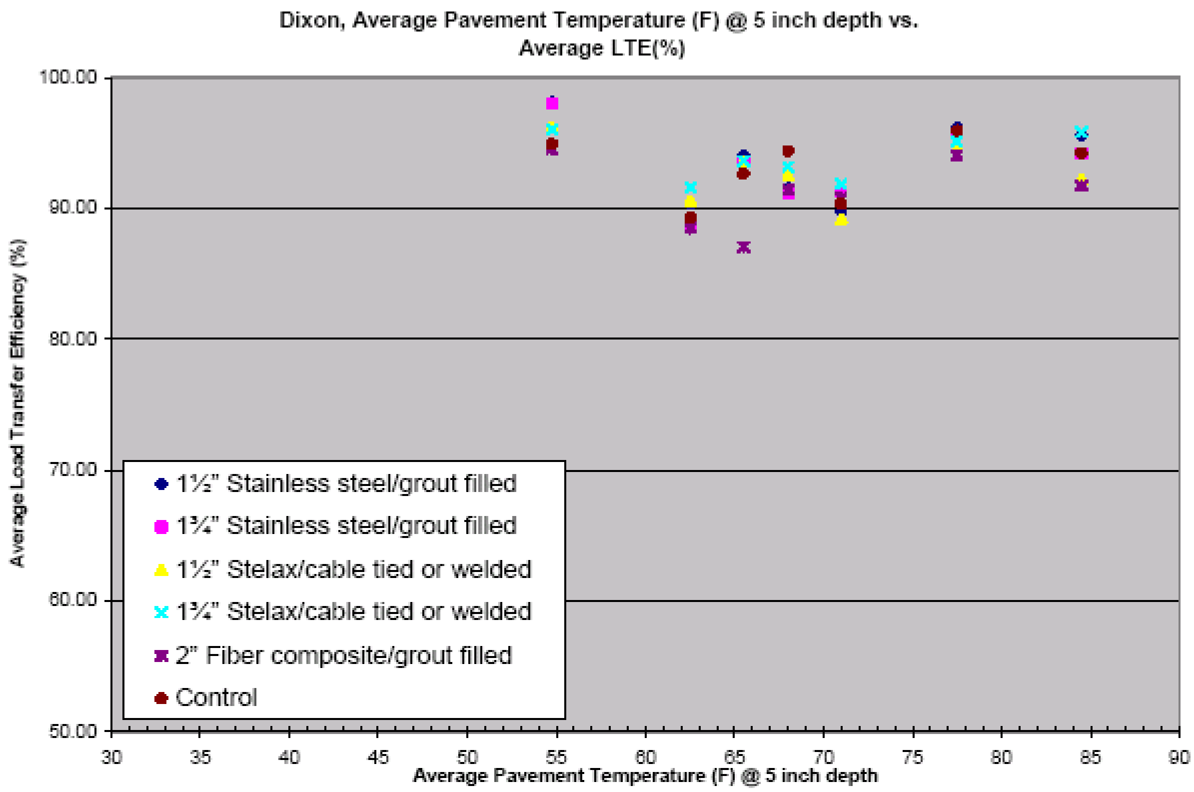

Figure 17. Average Load Transfer Efficiency vs. Average Pavement Temperature (Gawedzinski 2004).

Current Observations (Gawedzinski 2004)

At the time of construction, all of the test joints were to remain unsealed. Visual observation of the joints show all of the joints performing well with slight spalling possibly due to the pavement being cut too early. None of the joints show accumulation of incompressible material in the joint or any significant spalling due to the joints “locking up.” Additional monitoring will continue. The LTE% and joint deflection graphs show behavior expected with relatively new pavements.

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References

Gawedzinski, M. 2000. *TE-30 High Performance Rigid Pavements Illinois Project Review*. Illinois Department of Transportation, Springfield, IL.

Gawedzinski, M. 2004. *TE-30 High Performance Concrete Pavements: An Update of Illinois Projects*. Illinois Department of Transportation, Springfield, IL.

CHAPTER 11. IOWA 2 (U.S. Route 65, Des Moines)

Introduction

The Iowa Department of Transportation's second TE-30 project consists of an evaluation of alternative dowel bar materials and spacings. The experimental project was constructed in 1997 on the U.S. 65 Bypass near Des Moines (Cable and McDaniel 1998b). Figure 17 shows the location of this project.

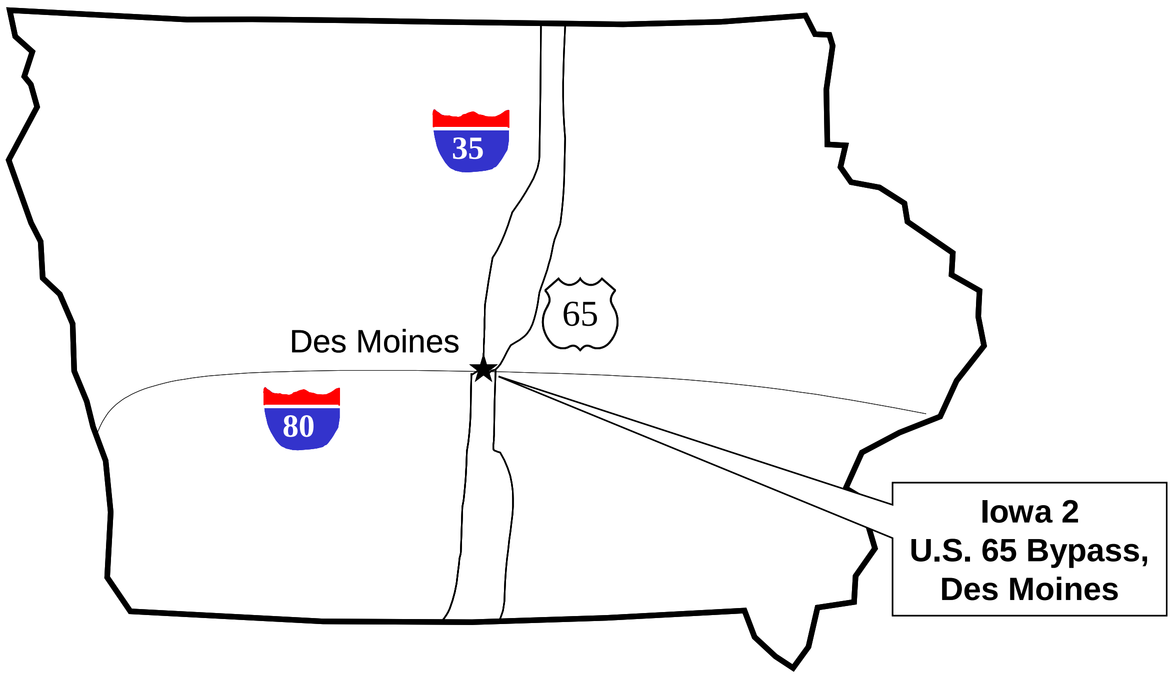

Figure 17. Location of IA 2 project.

Study Objectives

Because of the susceptibility of steel dowel bars to corrosion, the Iowa DOT has expressed interest in the use of alternative dowel bar materials to provide load transfer across transverse joints in concrete pavements. Therefore, one of the goals of this project is the comparative study of concrete pavement joints containing fiber reinforced polymer (FRP) dowel bars, stainless steel dowel bars, and conventional epoxy-coated steel dowel bars under the same design criteria and field conditions (Cable and McDaniel 1998b). Another goal of the project is the investigation of the transverse joint load transfer characteristics of alternative dowel bar spacings (Cable and McDaniel 1998b). This evaluation is a 5-year study being performed through the combined efforts of the Iowa Department of Transportation and the Iowa State University.

Project Design and Layout

This project was constructed in 1997 on the northbound lanes of the U.S. 65 Bypass near Des Moines. The basic design for the project is a 305-mm (12-in) JPCP on a 152-mm (6-in) granular base course (Cable and McDaniel 1998b). Transverse joints are located at 6.1-m (20-ft) intervals and are skewed 6:1 in the counterclockwise direction (Cable and McDaniel 1998b). Both transverse and longitudinal joints are sealed with a hot-poured sealant. Number 5 tie bars, 914 mm (36 in) long and spaced at 762-mm (30-in) intervals, were mechanically inserted by the paver across the longitudinal centerline joint (Cable and McDaniel 1998b).

The shoulder for the JPCP is a 203-mm (8-in) asphalt concrete (AC) layer, paved 2.4 m (8 ft) wide on the outside edge and 1.6 m (6 ft) on the inside edge (Cable and McDaniel 1998b). Longitudinal subdrains are located under the outside shoulder and adjacent to the edge of the outside driving lane (Cable and McDaniel 1998b).

Four different load transfer systems are included in the study: a fiber composite dowel bar manufactured by Hughes Brothers, a fiber composite dowel bar manufactured by RJD Industries, a Type 316L solid stainless steel dowel bar, and a conventional epoxy-coated steel dowel bar (Cable and McDaniel 1998b). The Hughes Brothers dowel bar is 48 mm (1.88 in) in diameter, whereas the other dowel bars are 38 mm (1.5 in) in diameter. The required diameters for the alternative dowel bars were determined from laboratory testing and experimental research performed by the manufacturers (Cable and McDaniel 1998b).

A standard spacing of 305 mm (12 in) was used for each load transfer system included in the study. In addition, sections were constructed using a spacing of 203 mm (8 in) for the alternative dowel bar materials. The experimental design matrix for this project is shown in table 9, and the layout of the test sections is shown in figure 18. The dowel bar spacing configurations used on this project are illustrated in figure 19.

Table 9. Experimental design matrix for IA 2.

	305-mm (12-in) JPCP 6.1-m (20-ft) Joint Spacing (skewed)			
	203-mm (8-in) Dowel Spacing		**305-mm (12-in) Dowel Spacing**	
	38-mm (1.5-in) Diameter Dowel	**48-mm (1.88-in) Diameter Dowel**	**38-mm (1.5-in) Diameter Dowel**	**48-mm (1.88-in) Diameter Dowel**
Fiber Composite Dowel Bars (*Hughes Brothers)*		Section 1 (440 ft)		Section 2 (417 ft)
Fiber Composite Dowel Bars (*RJD Industries)*	Section 3 (100 ft)		Section 4 (80 ft)	
Stainless Steel Dowel Bars	Section 5 (222 ft)		Section 6 (556 ft)	
Epoxy-Coated Steel Dowel Bars			Section 8 (477 ft)	

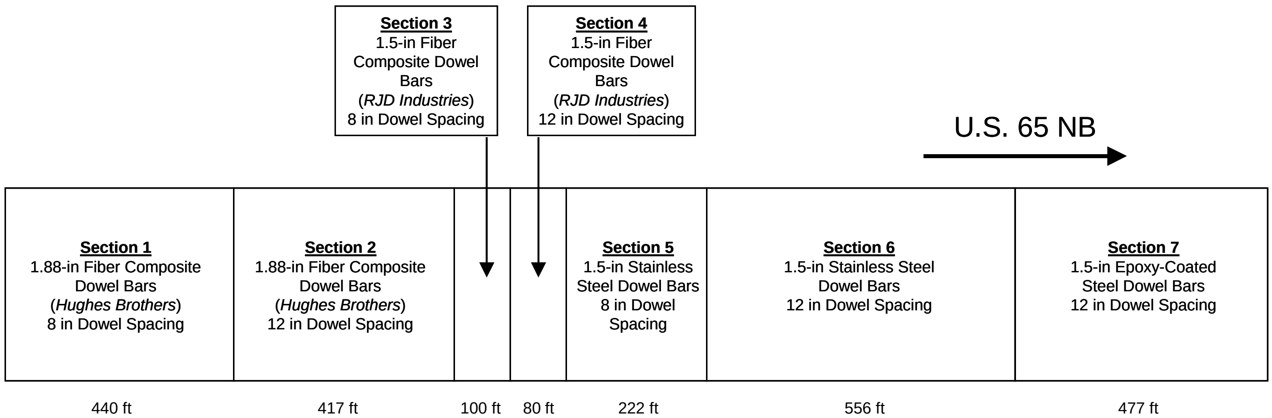

Figure 18. Layout of IA 2 project.

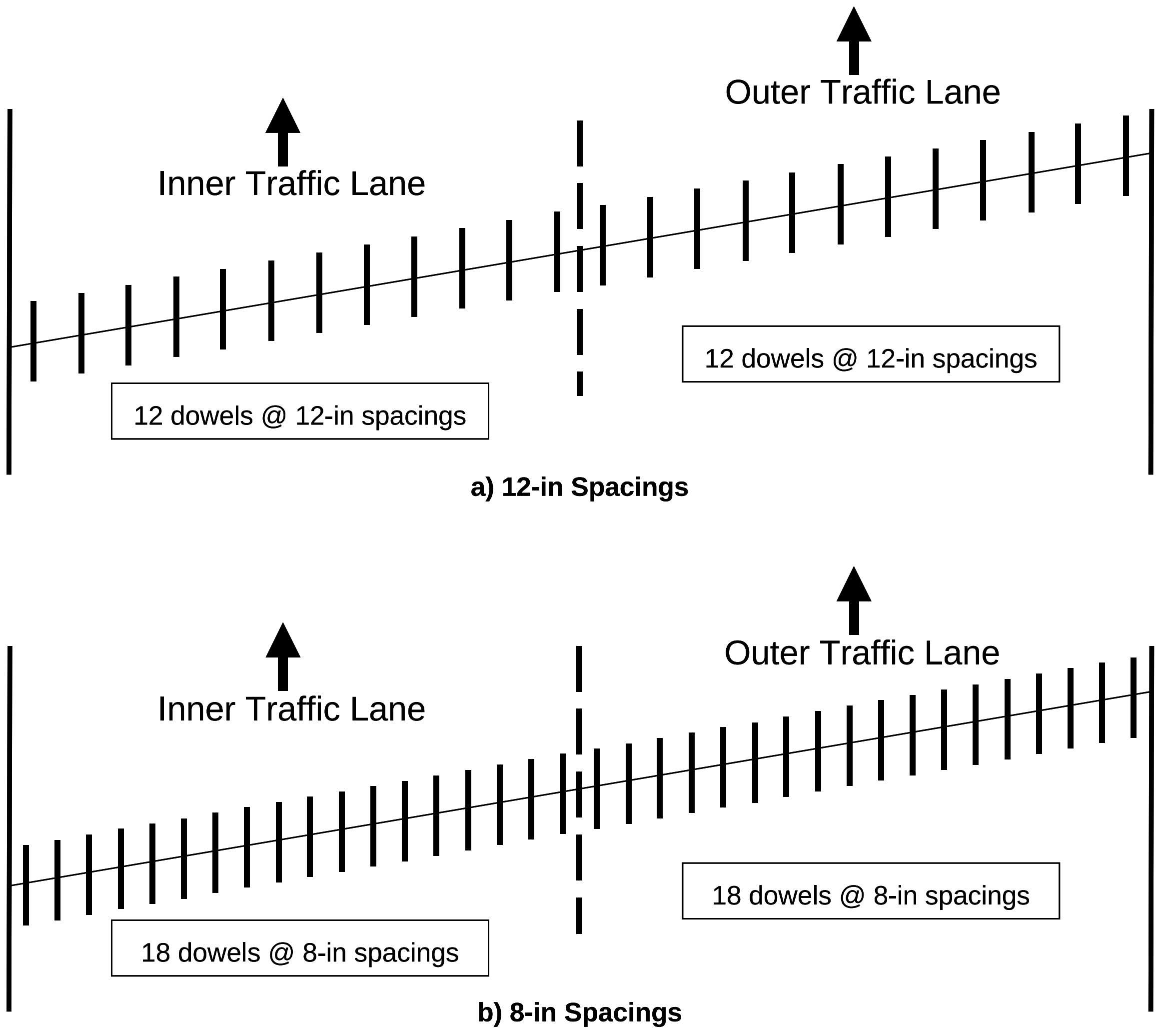

Figure 19. Illustration of dowel bar spacing configurations on IA 2.

Fiber composite tie bars were also provided by the fiber composite dowel bar manufacturers for installation in their respective test sections. However, these fiber composite tie bars had a tendency to "float" to the top of the surface during or immediately after their placement (Cable and McDaniel 1998b). This was attributed to either an incompatibility of the automatic tie bar inserter to the smaller diameter of the fiber composite tie bars or to the lighter weight of the fiber composite bars themselves (Cable and McDaniel 1998b). After several bars surfaced in succession, the epoxy-coated steel tie bars were used on the remainder of the project.

State Monitoring Activities

The performance of these test sections was monitored under a 5-year monitoring program (from the Fall of 1997 through the Spring of 2003) being conducted jointly by the Iowa DOT and the Iowa State University (Cable and Porter 2003). The following monitoring activities were conducted (Cable and Porter 2003):

- Visual distress survey using LTPP procedures. As part of these surveys, joint openings were monitored using PK nails placed along joints in each section, and joint faulting was measured using a Georgia Digital Faultmeter.
- Deflection testing using a Dynatest Falling Weight Deflectometer (FWD). Within each section, deflection testing was performed at three joints and at three center slab locations per lane. Testing was performed twice a year, once in March or April (to represent a "weak" foundation condition) and once in August or September (to represent a "strong" foundation condition).

In addition, ground penetrating radar (GPR) was used to establish the location (depth and orientation) of dowel bars and tie bars (Cable and Porter 2003). At the end of 5 years, selected joints in each section were cored and the condition of each dowel bar type was inspected (Cable and Porter 2003).

Preliminary Results/Findings

During the construction of the project, several items were noted to be of importance to future installations of alternative dowel bars in concrete pavements (Cable and McDaniel 1998b):

- The original method of securing the fiber composite and stainless steel dowel bars to the basket was inadequate. To address this, plastic zip ties were fastened around each basket brace loop and end of dowel to hold them in place. Any excess tie length was cut or turned down to prevent surface finishing problems.
- The placement of the stainless steel dowels required three to five people to handle the baskets. Future use of stainless steel dowels will require "x" braces welded to the basket to prevent side sway and collapse during handling.
- Nails were attached to the bottom of the fiber composite tie bars to facilitate their location using both cover meters and GPR.
- As stated previously, the fiber composite tie bars, placed using the automatic tie bar inserter on the paver, were susceptible to "floating" to the surface. If this is a continuing problem, the placement of these bars in tie bar baskets or the use of conventional epoxy-coated tie bars may be required.

Final Results/Findings

Project test sections were tested twice a year, beginning in the Fall of 1997, with the final tests in the Spring of 2002. Testing could not be performed in the fall of 2000. The results of the FWD testing were interpreted through calculating load transfer efficiency. The results of the load transfer analysis are illustrated in figure 20 (Cable and Porter 2003). In figure 20, the dowel bars are labeled according to their material and spacing: standard epoxy (std. epoxy), stainless steel (S.S.), fiber composite (FRP). Figure 21 displays the overall average faulting over the period of research (Cable and Porter 2003). Figure 22 illustrates the changes in joint openings over the research period (Cable and Porter 2003). Visual surveys of this project resulted in only minor corner cracking being noted immediately after construction. There are no visible signs of pavement distress that can be associated with joint reinforcement or typical highway loading over the five years of surveys (Cable and Porter 2003).

The following summaries and conclusions have been reached based on the data gathered during the study (Cable and Porter 2003):

- All dowel materials tested are performing equally in terms of load transfer, joint movement, and faulting over the five-year analysis period.
- Stainless steel dowels do provide load transfer performance equal to or greater than epoxy-coated steel dowels in this study on the average over five years.
- FRP dowels of the sizes tested in this research should be spaced no greater than 8 inches (203 mm) apart to gain load transfer performance at the same level as epoxy-coated steel dowels at 12-inch (305 mm) spacing.
- No deterioration due to road deicers was found on any of the dowel materials retrieved in the 2002 coring operation.

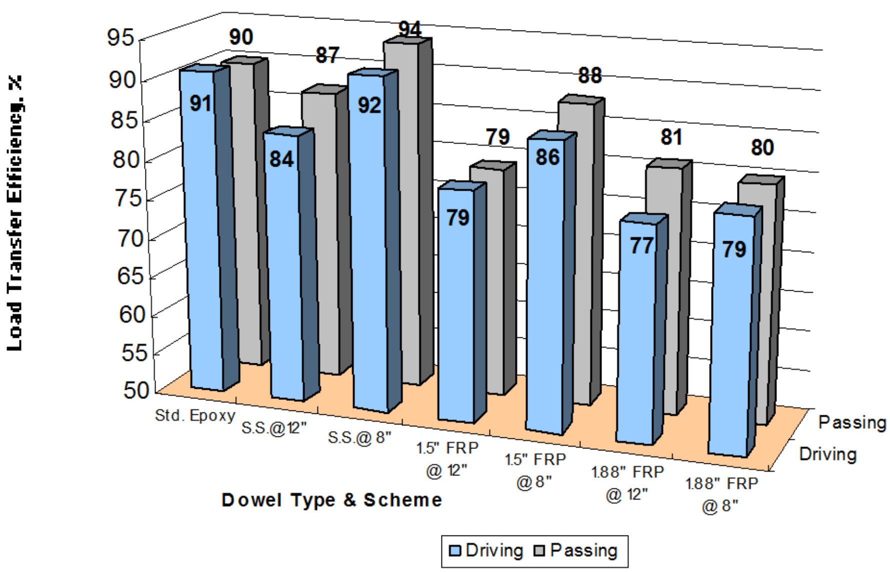

Figure 20. Average Load Transfer Efficiency.

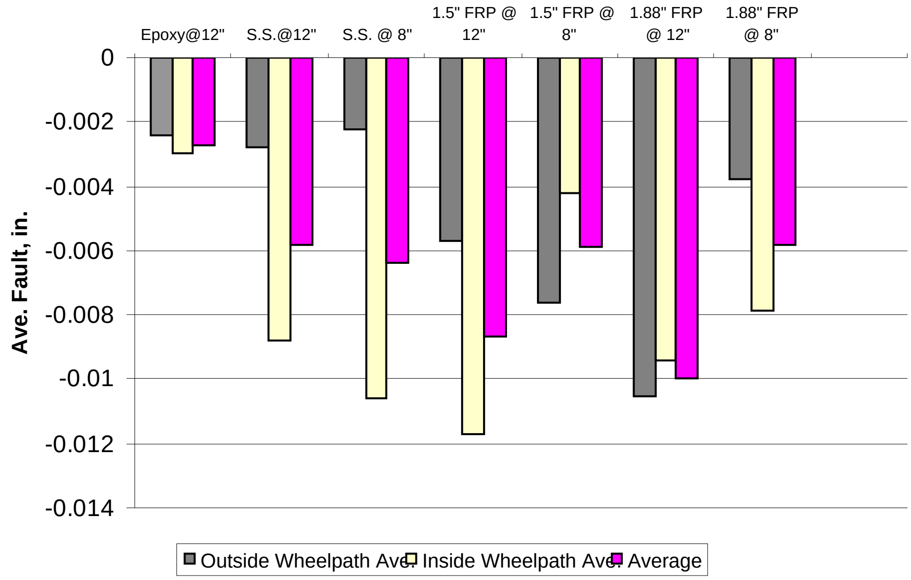

Figure 21. Average Faulting Over Research Period.

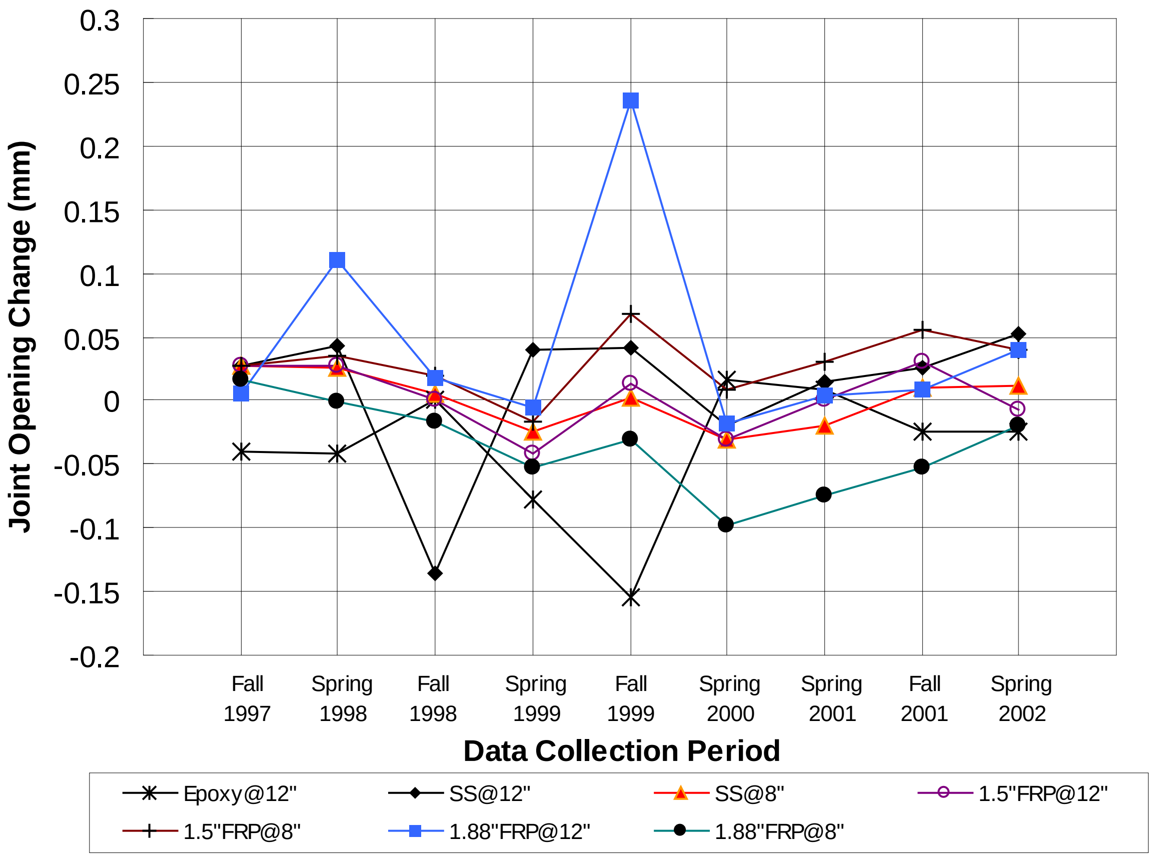

Figure 22. Joint Opening Trends.

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Reference

Cable, J. K. and L. L. McDaniel. 1998b. *Demonstration and Field Evaluation of Alternative Portland Cement Concrete Pavement Reinforcement Materials*. Iowa DOT Project HR-1069. Iowa Department of Transportation, Ames, IA.

Cable, J. K. and M. L. Porter. 2003. Demonstration and Field Evaluation of Alternative Portland Cement Concrete Pavement Reinforcement Materials. Iowa DOT Project HR-1069. Iowa State University, Ames, IA.

CHAPTER 27. OHIO 1, 2, AND 3 (U.S. Route 50, Athens)

Introduction

Under the TE-30 program, the Ohio Department of Transportation (ODOT) constructed three experimental pavement projects on U.S. 50, approximately 8 km (5 mi) east of the city of Athens (see figure 45). The projects incorporate a variety of experimental design features, including high-performance concrete mixtures utilizing ground granulated blast furnace slag (GGBFS) (Ohio 1), alternative dowel bar materials (Ohio 2), and alternative joint sealing materials (Ohio 3) (Ioannides et al. 1999; Sargand 2000; Hawkins et al. 2000). Although each project was funded separately under the TE-30 program, they are all located on the same section of roadway and share many of the same design and construction attributes, as well as the same traffic and environmental loadings; therefore, these projects are all described together in this chapter.

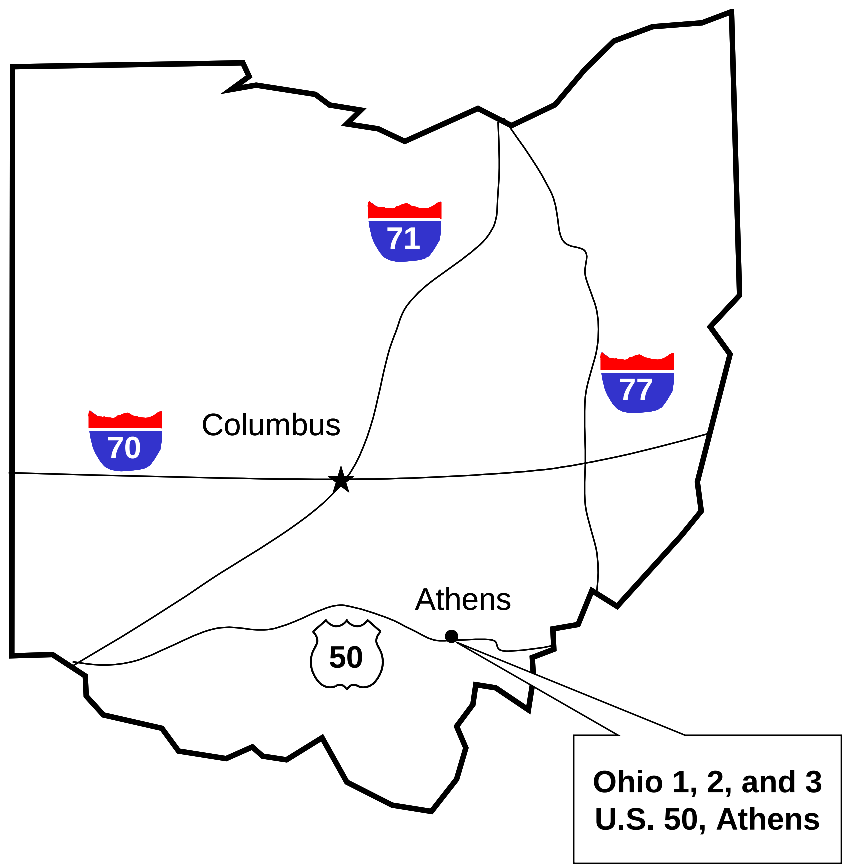

Figure 45. Location of OH 1, 2, and 3 projects.

Study Objectives

The study objectives for the overall U.S. 50 pavement project may be broken out by each specific study. For OH 1, the evaluation of GGBFS, the primary objective is to evaluate the effectiveness of GGBFS as a partial cement replacement in PCC pavements. The expectation of adding GGBFS to a concrete mix is increased workability, increased durability, and increased long-term strength.

For OH 2, the evaluation of alternative dowel bar materials, the general purposes of the study are to evaluate dowel response under a variety of loading and environmental conditions and to compare the measured responses of different types of dowel bars (Sargand 2000). Specific objectives include the following (Sargand 2000):

- Instrument standard steel and fiberglass dowels for the monitoring of strain induced by curing, changing environmental conditions, and applied dynamic forces.
- Record strain measurements periodically over time to determine forces induced in the dowel bars during curing and during changing environmental conditions.
- Record strain measurements in the dowel bars as dynamic loads are applied with the FWD.
- Evaluate strain histories recorded for the in-service pavement.

For OH 3, the evaluation of joint sealing materials, the objectives are to (Ioannides et al. 1999):

- Assess the effectiveness of a variety of joint sealing practices employed after the initial sawing of joints, and to examine their repercussions in terms of reduced construction times and life-cycle costs.
- Identify those materials and procedures that are most cost effective.
- Determine the effect of joint sealing techniques on pavement performance.

Project Design and Layout

General Design Information

The U.S. 50 project is a 10.5-km (6.5-mi) segment of highway that was reconstructed and expanded to a new four-lane divided facility. The eastbound lanes of the project were constructed in the fall of 1997, and the westbound lanes were constructed in the fall of 1998 (Ioannides et al. 1999).

The 20-year design traffic loading for this pavement is approximately 11 million ESAL applications. The subgrade over the project site is predominantly a silty clay material (Ioannides et al. 1999).

The cross-sectional design for the projects is a 254-mm (10-in) JRCP placed over a 102-mm (4-in) open-graded base course. The open-graded base course in the eastbound direction is a “New Jersey” type nonstabilized base, whereas the open-graded base course in the westbound direction is a “Iowa” type nonstabilized base (Ioannides et al. 1999). A 152-mm (6-in) crushed aggregate subbase is located beneath the open-graded bases, and is topped with a bituminous prime coat to prevent migration of fines into the open-graded layers (Ioannides et al. 1999). Table 21 provides the actual project gradations for these materials. A 102-mm (4-in) underdrain was placed at both the outside and inside edges of the pavement to collect infiltrated moisture from the open-graded bases (Ioannides et al. 1999).

Table 21. Comparison of actual base and subbase gradations used on Ohio U.S. 50 project.

Sieve Size	**Total Percent Passing**		
	New Jersey Open-Graded Base (EB)	**Iowa Open-Graded Base (WB)**	**Crushed Aggregate Subbase (EB/WB)**
2 in			100
1½ in	100		
1 in		100	
#8	12	30	25
#16	6	19	18
#30	4	15	14
#40	4	12	13
#50	4	9	12
#100	3	6	10
#200	3.2	5.6	9.8

The slabs are reinforced with smooth welded wire fabric (WWF) to control random cracking (Sargand 2000). Wire style designation W8.5 x W4—6x12 was specified, meaning that the longitudinal wires have a cross sectional area of 54.8 mm^2 (0.085 in^2) and are spaced 152 mm (6 in) apart, and the transverse wires have a cross-sectional area of 25.8 mm^2 (0.04 in^2) and are spaced 305 mm (12 in) apart. This style designation translates to a longitudinal steel content of 0.14 percent.

The transverse joints are spaced at fixed 6.4-m (21-ft) intervals and contain 38-mm (1.5-in) diameter, 457-mm (18-in) long, epoxy-coated dowel bars on 305-mm (12-in) centers (Sargand 2000). However, some of the joints within the alternative dowel bar project contain either fiberglass dowels or stainless steel tubes filled with concrete (Sargand 2000). Transverse joints were sealed with a preformed compression sealant except for the joints within the joint sealant project. The longitudinal centerline joint is tied with 16-mm (0.62-in) diameter, 760-mm (30-in) long, deformed bars spaced at 760-mm (30-in) intervals (Ioannides et al. 1999).

Plain concrete shoulders were paved separately from the mainline pavement. These were tied to the mainline pavement using 16-mm (0.62-in) diameter, 76-mm (30-in) long, deformed tie bars. The outside shoulder is 3 m (10 ft) wide and the inside shoulder is 1.2 m (4 ft) wide (Ioannides 1999).

Project Layout Information

As described previously, the U.S. 50 project actually includes three projects, one evaluating GGBFS, one evaluating alternative dowel bar materials, and one evaluating joint sealant materials. In addition, a control section that does not contain GGBFS is located at the western end of the project. The general layout of these projects is shown in figure 46. More detailed information on each project is provided in the following sections.

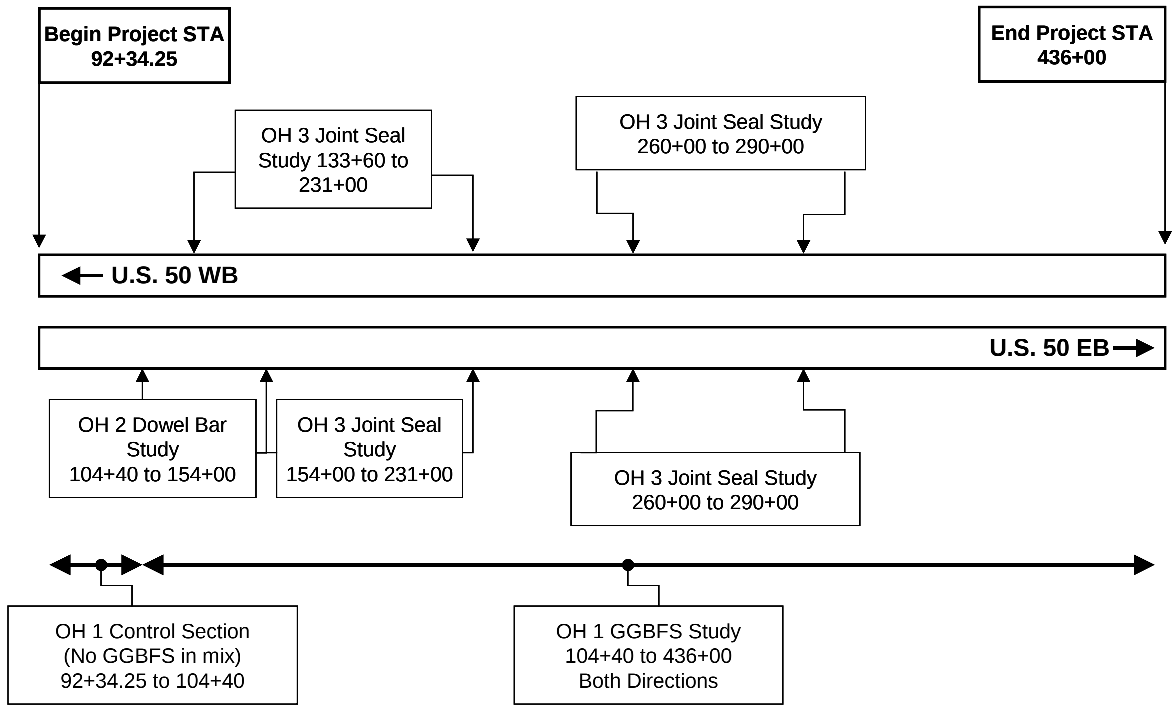

Figure 46. Layout of experimental projects on Ohio U.S. 50.

OH 1, Evaluation of Ground Granulated Blast Furnace Slag

The entire 10.5-km (6.5-mi) length of the U.S. 50 project was constructed using a high-performance concrete mix. The mixture consists of a Type I cement with GGBFS replacing 25 percent of the cement (Sargand 2000). An AASHTO #8 gravel (0.13 mm [0.5 in] top size) was used for the coarse aggregate and a natural sand was used for the fine aggregate (Sargand 2000). A w/c of 0.44 was used in the mix design. The complete PCC mix design is shown in table 22.

Table 22. Concrete pavement mix design used on Ohio U.S. 50 project.

PCC Mix Design Component	Quantity
Natural Sand	1437 lb/yd^3
AASHTO #8 Aggregate	1374 lb/yd^3
Type I Cement	412 lb/yd^3
Water	236 lb/yd^3
GGBFS	138 lb/yd^3
Water Reducer	11 oz/yd^3
Air Entraining Agent	16.5 oz/yd^3
Design Air	8%
Design Slump	3 in

Samples from the concrete mix used in the actual paving operation were tested in the laboratory and showed a 28-day compressive strength of 27.6 MPa (4000 lbf/in^2) and a 28-day modulus of rupture of 2.76 MPa (400 lbf/in^2) (Sargand 2000). The 28-day static modulus of elasticity was 25.92 GPa (3,760,000 lbf/in^2) (Sargand 2000).

As previously mentioned, a control pavement section that does not contain GGBFS in the concrete mix is located at the western end of the project, between stations 92+35.4 and 104+40. Other than the mix design, the design of the control section is the same as the GGBFS section.

OH 2, Evaluation of Alternative Dowel Bars

Three types of dowel bars were used in the dowel bar project: epoxy-coated steel dowel bars, fiberglass dowel bars (manufactured by RJD Industries, Inc.), and stainless steel tubes filled with concrete. The diameter of the steel and fiberglass dowels bars is 38 mm (1.5 in), while the stainless steel tubes have an outer diameter of 38 mm (1.5 in) and an inner diameter of 34 mm (1.35 in) (Sargand 2000). All bars are 457 mm (18 in) long.

Most of the U.S. 50 project contains conventional epoxy-coated steel dowel bars. However, three specific test sections, each incorporating one of the load transfer devices under study, were set up near the western-most limits of the project in the eastbound direction to instrument dowel response and to compare the performance of the different load transfer devices. Each test section is made up of six consecutive joints, with the middle two joints containing instrumented dowel bars (see figure 47). The concrete-filled stainless steel bars were not instrumented because the thin wall thickness did not permit the necessary installation operation to protect the lead wires of the gages (Sargand 2001).

Three dowel bars within each joint are instrumented. The instrumented bars are located at distances of 152 mm (6 in), 762 mm (30 in), and 1980 mm (78 in) from the outside edge of the pavement, as shown in figure 48 (Sargand 2000). Each instrumented dowel bar contained a uniaxial strain gauge on the top and the bottom of the bar, and one 45-degree rosette on the side. The uniaxial gauges measure environmental and dynamic strains while the rosette gauges measure only dynamic strains (Sargand 2000).

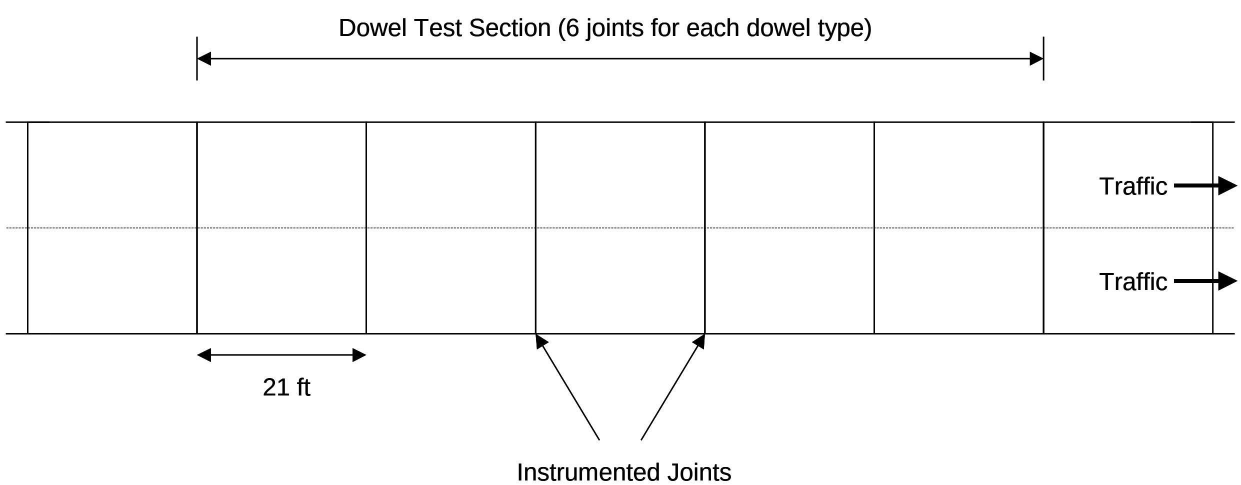

Figure 47. Layout of dowel test sections on Ohio U.S. 50 project.

Two thermocouple units were also installed near each instrumented joint to measure temperatures in the concrete slab. One unit housed three sensors that measure temperatures at depths of 102, 178, and 254 mm (4, 7, and 10 in) from the surface of the slab, and the second unit consists of a single sensor measuring temperatures at a depth of 25 mm (1 in) below the surface of the slab (Sargand 2000).

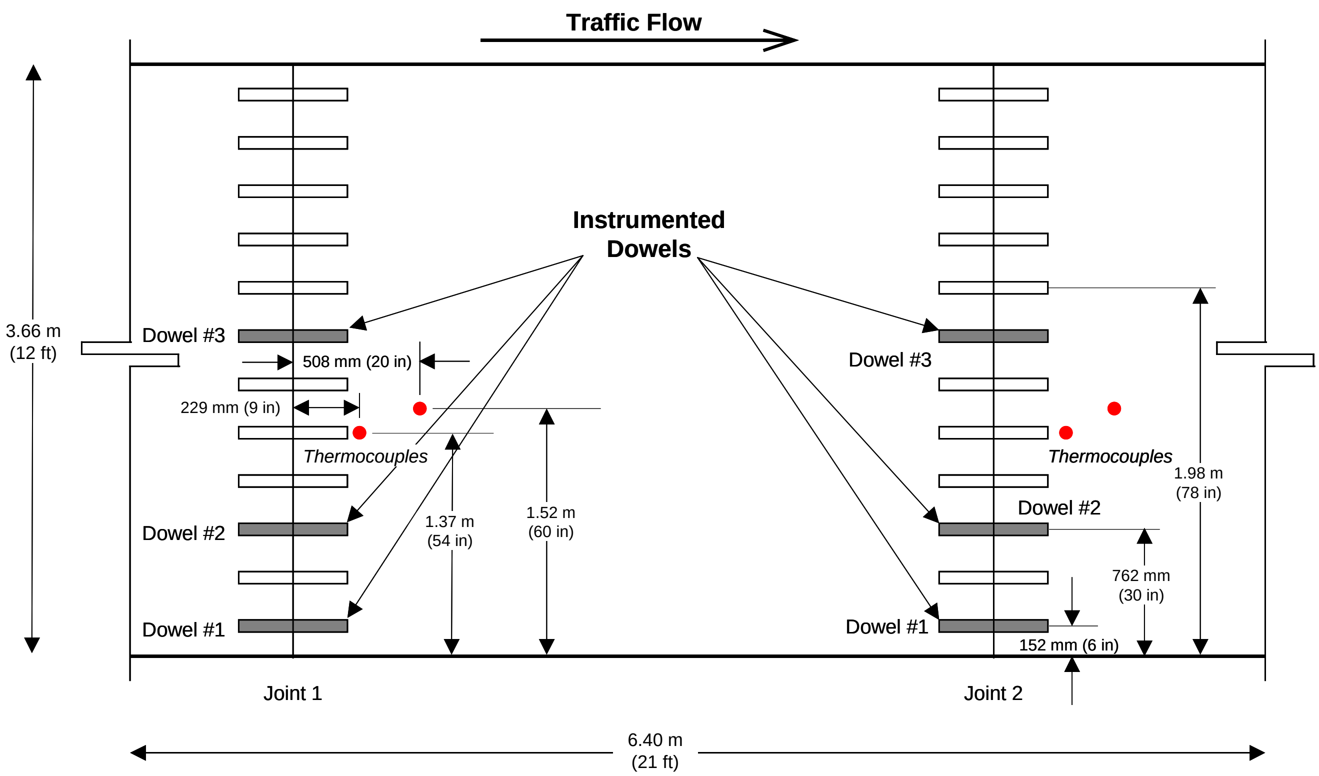

Figure 48. Dowel instrumentation layout for Ohio U.S. 50 project (Sargand 2000).

OH 3, Evaluation of Joint Sealing Materials

The joint sealant evaluation is conducted in selected segments of both the eastbound and westbound directions of U.S. 50. A total of nine different joint sealants are evaluated (including four silicone sealants, two hot-poured sealants, and three compression seals), each of which is installed in a unique joint channel configuration. In addition, several pavement sections containing no sealant are included in the study.

Table 23 summarizes the location of the different sealant materials in each direction, as well as the joint channel configuration (see figure 49) used for each material (Hawkins, Ioannides, and Minkarah 2000). The westbound sections each represent replicate sealant sections of those in the eastbound lanes, with the exception of the Watson Bowman WB-687 in the eastbound lanes, which was replicated using the Watson Bowman WB-812 in the westbound lanes (Ioannides et al. 1999). The eastbound lanes were sealed in October and November of 1997, whereas the westbound lanes were sealed in December 1998 (silicone and compression seals) and April 1999 (hot-poured sealants) (Ioannides et al. 1999).

State Monitoring Activities

The Ohio DOT, in conjunction with researchers from several state universities, monitored the performance of these pavements for 5 years. Annual condition surveys and profile measurements were conducted, along with special FWD testing on the instrumented joints. In addition, detailed joint sealant evaluations following SHRP procedures were performed annually on a selected samples of each sealant material.

Table 23. Sealant materials used in joint sealant study on Ohio U.S. 50 project (Hawkins, Ioannides, and Minkarah 2000).

Sealant Material	Sealant Type	Begin Station	End Station	Joint Configuration	Section Length, ft	No. of Joints
Eastbound Direction						
TechStar W-050	Preformed	154+00	160+00	5	600	29
No Sealant	—	160+00	166+00	6	600	29
Dow 890-SL	Silicone	166+00	172+00	3	600	29
Crafco 444	Hot-Pour	172+00	188+00	1	1600	76
Crafco 903-SL	Silicone	188+00	194+00	1	600	29
Watson Bowman WB-687	Preformed	194+00	200+00	5	600	27
Crafco 902 Silicone	Silicone	200+00	206+00	1	600	29
Crafco 903-SL	Silicone	206+00	213+00	4	700	33
Dow 890-SL	Silicone	213+00	219+00	4	600	29
No Sealant	—	219+00	225+00	2	600	28
Delastic V-687	Preformed	225+00	231+00	5	600	29
Crafco 221	Hot-Pour	260+00	266+00	1	600	29
Dow 890-SL	Silicone	266+00	272+00	1	600	28
Dow 888	Silicone	272+00	284+00	1	1200	57
Dow 888	Silicone	284+00	290+00	1	600	29
Westbound Direction						
TechStar W-050	Preformed	133+60	139+60	5	600	29
No Sealant	—	139+60	166+00	2	2640	126
Dow 890-SL	Silicone	166+00	172+00	3	600	29
Crafco 221	Hot-Pour	172+00	188+00	1	1600	76
Crafco 903-SL	Silicone	188+00	194+00	1	600	29
Crafco 903-SL	Silicone	194+00	200+00	1	600	29
Dow 890-SL	Silicone	200+00	206+00	1	600	28
Crafco 444	Hot-Pour	206+00	213+00	1	700	33
Dow 888	Silicone	213+00	219+00	1	600	28
Delastic V-687	Preformed	219+00	225+00	5	600	29
Watson Bowman WB-812	Preformed	225+00	231+00	5	600	28
Dow 888	Silicone	260+00	266+00	1	600	29
Crafco 903-SL	Silicone	266+00	272+00	4	600	28
Dow 890-SL	Silicone	272+00	284+00	4	1200	57
No Sealant	—	284+00	290+00	6	600	29

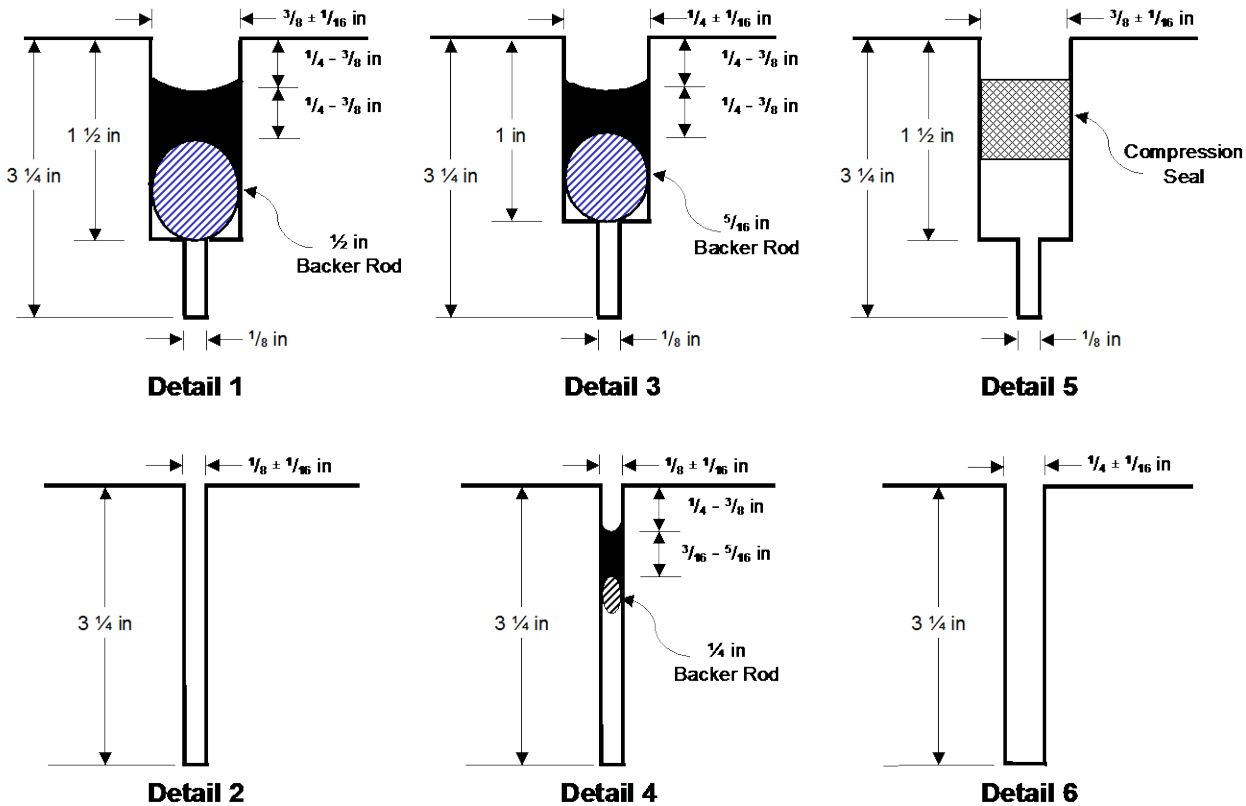

Figure 49. Joint channel configurations used in sealant study on Ohio U.S. 50 project (Hawkins, Ioannides, and Minkarah 2000).

Results/Findings

Performance results are available in the final reports for these sections. This information is presented in the following sections for each specific study.

OH 1, Evaluation of Ground Granulated Blast Furnace Slag

The final report, *Application of High Performance Concrete in the Pavement System, Structural Response of High Performance Pavements,* March 2002 provides the results from this study. Several factors related to the performance of the HPC pavement containing 25 percent GGBFS have been evaluated with the following results.

- Temperature gradients generated between the top and bottom of concrete slabs during the cure period can have a significant impact on the development of early cracks. HPC pavement sections placed in October, 1997 experienced gradients of 10 degrees C, and developed cracking within eighteen hours of placement. One HPC and one standard pavement section placed in October, 1998 experienced gradients of only 5 degrees C, and did not develop cracking. The higher temperature gradient in 1997 resulted from a cold front shortly after placement.
- Large values of strain recorded with the vibrating wire strain gages and maturity measurements indicated that the HP 1 and HP 2 sections could be expected to crack, as

was observed in the field. HP 3 constructed one year later of the same concrete mix but during a period of warmer weather did not develop cracks. In this case, both strain and maturity data collected in the field indicated a low probability of cracking.

- Results from HIPERPAV also suggested that sections HP 1 and HP 2 would crack, while HP 3 would not. Predicted strength curves were calculated for the placements, in addition to those provided by the standard HIPERPAV prediction model.
- Section HP 3 had less initial warping than did section SP (standard ODOT paving concrete). Sections HP 1 and 2 developed cracking, precluding effective curling measurement of these slabs.

Based on the laboratory results and field data obtained in this study, the following conclusions were derived (Sargand 2002):

- Temperature gradients generated between the surface and bottom of concrete slabs during the curing process can have a significant impact on the formation of early cracks.
- Section HP3 had less initial warping than did section SP constructed with standard ODOT class C concrete.
- FWD data indicated that, under similar loading conditions, the HP3 section experienced slightly less deflection at joints than the SP section.
- With limited data available, it was suggested that the moisture in the base at sealed and unsealed joints was similar. In some cases, however, moisture under sealed conditions was observed to be slightly higher, indicating that joint seals might trap moisture under the pavement.
- During FWD tests the deflection at sealed joints was generally higher than at unsealed joints.

OH 2, Evaluation of Alternative Dowel Bars

An analysis of the strains in both the fiberglass and steel dowel bars under environmental and dynamic loading was conducted (ORITE 1998; Sargand 2000; Sargand 2001). Major findings from that analysis include (Sargand 2000; Sargand 2001):

- In addition to transferring dynamic load across PCC pavement joints, dowel bars serve as a mechanism to reduce the curling and warping of slabs due to curing and temperature and moisture gradients in the slabs.
- Steel and fiberglass dowels both experienced higher moments from environmental factors than from dynamic loading. The dynamic bending stresses induced by a 56.9 kN (12,800 lb) load were considerably less than the environmental bending stresses induced by a 3 ^{o}C (5.4 ^{o}F) temperature gradient.
- Steel bars induced greater environmental bending moments than fiberglass bars.
- Significant stresses were induced by steel dowel bars early in the life of this pavement as it cured late in the construction season under minimal temperature and thermal gradients

in the slab. Concrete pavements paved in the summer under more severe conditions may reveal even larger environmental stresses.

- Both types of dowels induced a permanent bending moment in the PCC slabs during curing, the magnitude of which is a function of bar stiffness.
- Curling and warping during the first few days after concrete placement can result in large bearing stresses being applied to the concrete around the dowels. This stress may exceed the strength of the concrete at that early age and result in some permanent loss of contact around the bars.
- Steel bars transferred greater dynamic bending moments and vertical shear stresses across transverse joints than fiberglass bars of the same size.

Given these findings, it is concluded that the effects of environmental cycling and dynamic loading both must be included in the design and evaluation of PCC pavement joints (Sargand 2001). Because of the high bearing stresses that can be generated in concrete surrounding dowel bars, this parameter should be considered in dowel bar design, especially during the first few days after placement of concrete (Sargand 2001).

It is noted that these results are based on the analysis of the instrumented steel and fiberglass dowel bars only. The stainless steel tubes were not instrumented for the reason stated earlier.

OH 3, Evaluation of Joint Sealing Materials

The results from this experiment, through the 2001 performance evaluation have resulted in several observations (Ioannides et al. 1999; Hawkins, Ioannides, and Minkarah 2000):

- The silicone and hot-poured sealants in the eastbound lanes are in fair to poor condition, typically suffering from full-depth adhesion failure.
- The worst of the sealed sections were those with a narrow joint width of 3 mm (0.12 in). In these installations, the sealant material had overflowed and run onto the pavement surface.
- There is a significant difference in the performance of the same joint seal materials from EB (constructed in '97) and WB (constructed in '98). This difference is attributed to improvements in installation temperatures, experience, and equipment.
- The joints in this experiment were cleaned only by water- and air-blasting, even when the sealant manufacturers recommended sand blasting. This suggests that some of the adhesion loss may be due to an inadequate cleaning process.
- Both the Watson Bowman and the Delastic compression seals have performed by far best overall in both directions. In the WB direction, the silicones have performed best, but were poor in the EB. The performance of the hot pour materials is very different, being far better in WB in general. However, the Crafco 221 material did relatively well in one EB test section. The TechStar compression seal, however, has developed significant adhesion failure and has sunk into the joint.

- The compression seals have performed by far best overall in both directions. In the WB direction, the silicones have performed best, but were poor in the EB. The performance of the hot pour materials is very different, being far better in WB in general. However, the Crafco 221 material did relatively well in one EB test section.
- Hot pour material appears to have performed better when installed within the manufacturer's recommended temperature range. No specific temperature range is recommended for the silicone materials.
- Roughness measurements made using PSI, IRI, and Mays meter do not provide any conclusive trends relating to pavement performance.
- Assessment of joint seal efficiency has little relationship to pavement condition, at this time. It is recommended to reseal the EB sites, except for the two compression seals for continued performance monitoring.
- The Techstar W-050 material performed poorly in both directions, and is considered unsuitable for pavement applications.
- Currently, the unsealed sections seem to have more spalling, corner, and midslab cracking distress than others, although there is no conclusive pavement performance related trends as yet.

A summary of estimated joint sealant costs on this project is provided in table 24 (Ioannides et al. 1999). These costs are based solely on the material costs themselves and do not include the costs of backer rods, adhesives, or labor.

Table 24. Summary of sealant costs on Ohio U.S. 50 project (Ioannides et al. 1999).

Material	Unit Cost	Estimated Cost/Joint
Dow 890-SL	$48.00/gal	$12.27
Crafco 903-SL	$36.00/gal	$9.50
Dow 888	$42.00/gal	$10.74
Crafco 902	$39.00/gal	$9.97
Crafco 444	$10.50/gal	$2.68
Crafco 221	$0.25/lb	$0.64
Watson Bowman WB-812	$1.03/ft	$43.26
Watson Bowman WB-687	$0.72/ft	$30.24
Delastic V-687	$0.66/ft	$27.72
TechStar V-050	$8.65/ft	$363.30

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CHAPTER 36. WISCONSIN 2 (Highway 29, Owen) AND WISCONSIN 3 (Highway 29, Hatley)

Introduction

In the summer of 1997, WisDOT constructed two experimental concrete pavement projects on Highway 29 to investigate the constructability and cost effectiveness of alternative concrete pavement designs (Crovetti 1999; Crovetti and Bischoff 2001). Constructed with partial funding from the TE-30 program, one project (designated WI 2) is located in the eastbound lanes of Highway 29 between Owen and Abbotsford, while the other project (designated WI 3) is located in both lanes of Highway 29 between Hatley and Wittenberg (see figure 60). The WI 3 test sections are also part of FHWA's ongoing Strategic Highway Research Program (SHRP) study. Because of the similarities and complementary design of these two projects, they are considered together in this chapter.

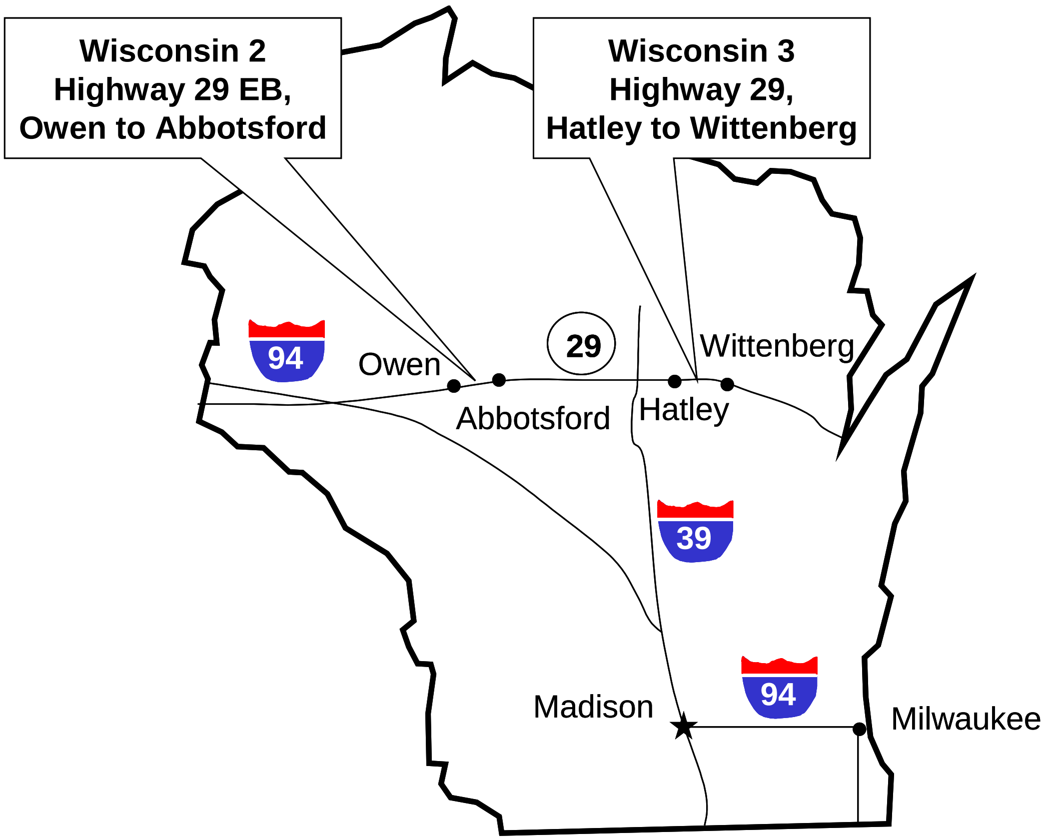

Figure 60. Location of WI 2 and WI 3 projects.

Study Objectives

The overall objective of these projects is to evaluate the constructability and cost-effectiveness of alternative concrete pavement designs (Crovetti 1999). Among the different concrete pavement designs and design features being investigated in these projects are (Crovetti 1999):

- Reduced number of dowel bar across transverse joints.
- Alternative dowel bar materials for transverse joint load transfer.
- Variable thickness pavement cross section.

Project Design and Layout

Wisconsin 2

The WI 2 project is located only in the eastbound lanes of Highway 29. It was constructed in September 1997 and includes both alternative dowel bar materials and alternative dowel bar layouts (Crovetti 1999; Crovetti and Bischoff 2001):

- Alternative Dowel Bar Materials
  - Standard epoxy-coated steel dowel bars.
  - Solid stainless steel dowel bars, manufactured by Avesta Sheffield.
  - Fiber-reinforced polymer (FRP) composite dowel bars, manufactured by Glasforms.
  - FRP composite dowel bars, manufactured by Creative Pultrusions.
  - FRP composite dowel bars, manufactured by RJD Industries.
  - Stainless steel tubes filled with mortar, manufactured by Damascus Bishop.
- Alternative Dowel Bar Layouts
  - Standard dowel layout (dowels spaced at 305-mm [12-in] intervals).
  - Alternative dowel layout 1 (three dowels in each wheelpath).
  - Alternative dowel layout 2 (four dowels in outer wheelpath, three in all other wheelpaths).
  - Alternative dowel layout 3 (four dowels in outer wheelpath, three in all other wheelpaths, one dowel at outer edge).
  - Alternative dowel layout 4 (three dowels in all wheelpaths, one dowel near outer edge).

The alternative dowel bar layouts are illustrated in figure 61. These layouts were selected to reduce dowel bar requirements while still maintaining standard placement locations used in Wisconsin (Crovetti 2001).

The nominal pavement design for these pavement sections is a 275-mm (11-in) JPCP with skewed variable joint spacing of 5.2-6.1-5.5-5.8 m (17-20-18-19 ft) (Crovetti 1999). The dowel bars were 38 mm (1.5 in) in diameter and were placed using an automated dowel bar inserter (DBI). The transverse joints were left unsealed.

The pavement was constructed over existing base materials that were salvaged from the in-place structure, including 230 mm (9 in) of existing dense-graded, crushed aggregate subbase and 125 mm (5 in) of existing dense-graded, crushed aggregate base. An additional 50 mm (2 in) of new dense-graded aggregate base was placed prior to the PCC paving.

Figure 62 shows the approximate layout of the eleven test and two control sections included in the WI 2 project, using the section nomenclature adopted by the researchers. Nominal 161-m (528-ft) long pavement segments generally consisting of twenty-nine joints were selected from within each test section for long term monitoring (Crovetti 1999). Table 30 provides the experimental design matrix for the project.

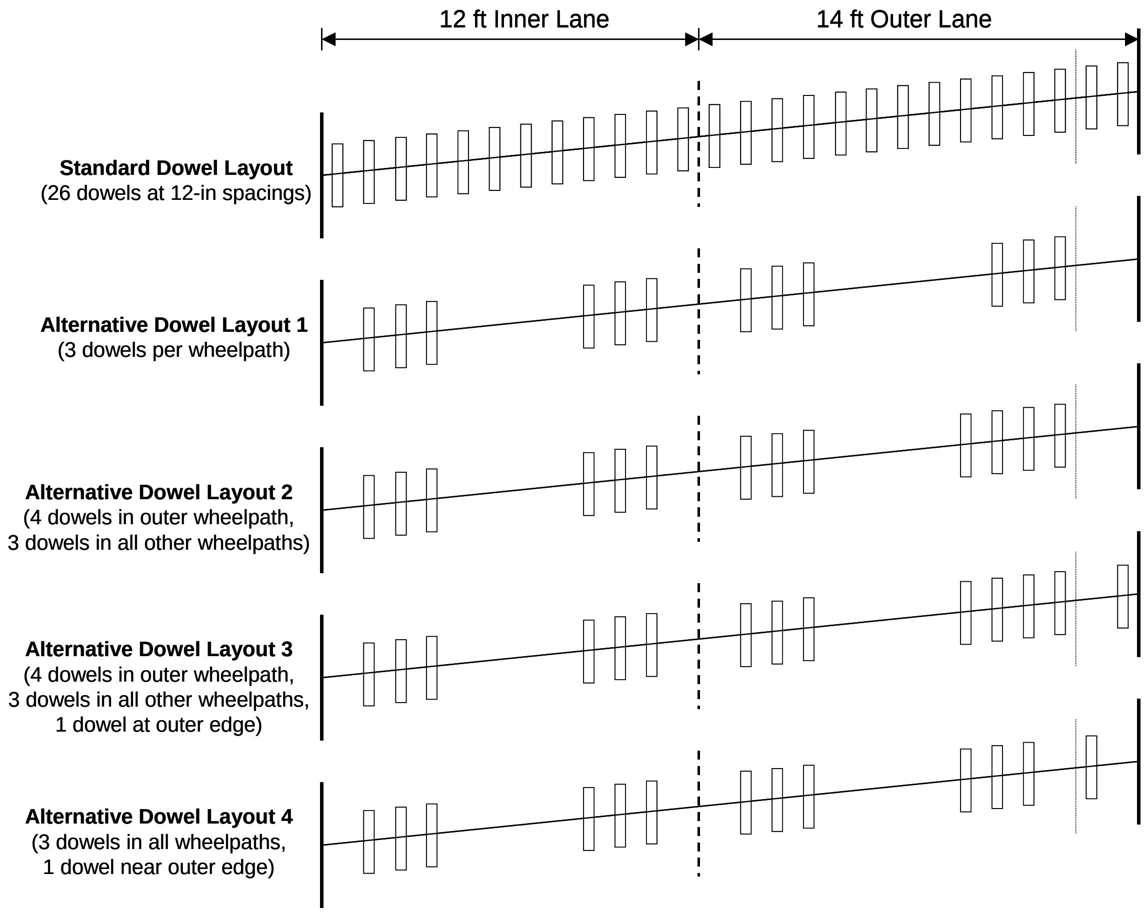

Figure 61. Alternative dowel bar layouts used on WI 2.

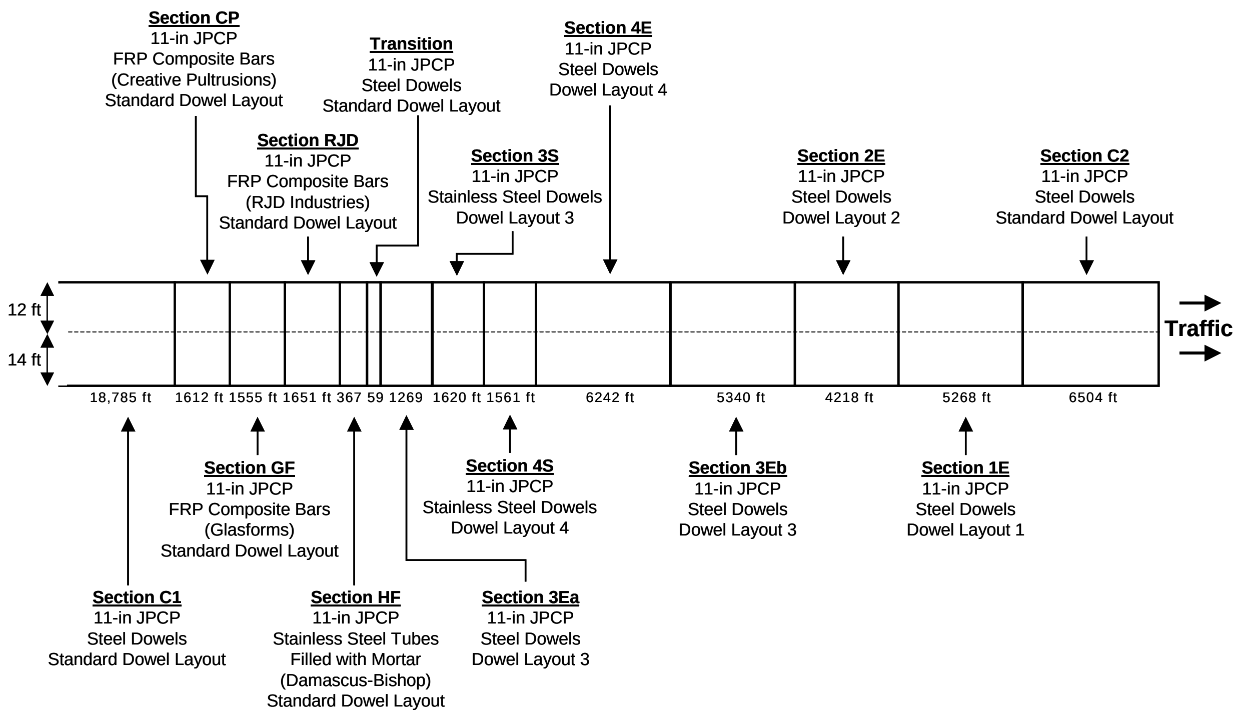

Figure 62. Approximate layout of WI 2 test sections.

Table 30. Experimental design matrix for WI 2.

	11 in JPCP 17-20-18-19 ft Joint Spacing				
	Standard Dowel Layout	**Alternative Dowel Layout 1**	**Alternative Dowel Layout 2**	**Alternative Dowel Layout 3**	**Alternative Dowel Layout 4**
Standard Epoxy-Coated Steel Dowels	Section C1 Section C2	Section 1E	Section 2E	Section 3Ea Section 3Eb	Section 4E
Solid Stainless Steel Dowels *(Avesta Sheffield)*				Section 3S	Section 4S
FRP Composite Dowel Bars *(Creative Pultrusions)*	Section CP				
FRP Composite Dowel Bars *(Glasforms)*	Section GF				
FRP Composite Dowel Bars *(RJD Industries)*	Section RJD				
Stainless Steel Tubes Filled with Mortar *(Damascus-Bishop)*	Section HF				

Wisconsin 3

The westbound lanes of the WI 3 project were constructed in June 1997, whereas the eastbound lanes were constructed in October 1997 (Crovetti 1999). The project includes the evaluation of a variable thickness cross section, an alternative dowel bar layout, and alternative dowel bar materials. The variable thickness cross section uses a 275 mm (11 in) thickness at the outside edge of the outer lane that then tapers to a thickness of 200 mm (8 in) at the far edge of the inner lane (see figure 63). The goal is the more efficient use of materials in areas subjected to greater traffic loading, resulting in more cost-effective designs.

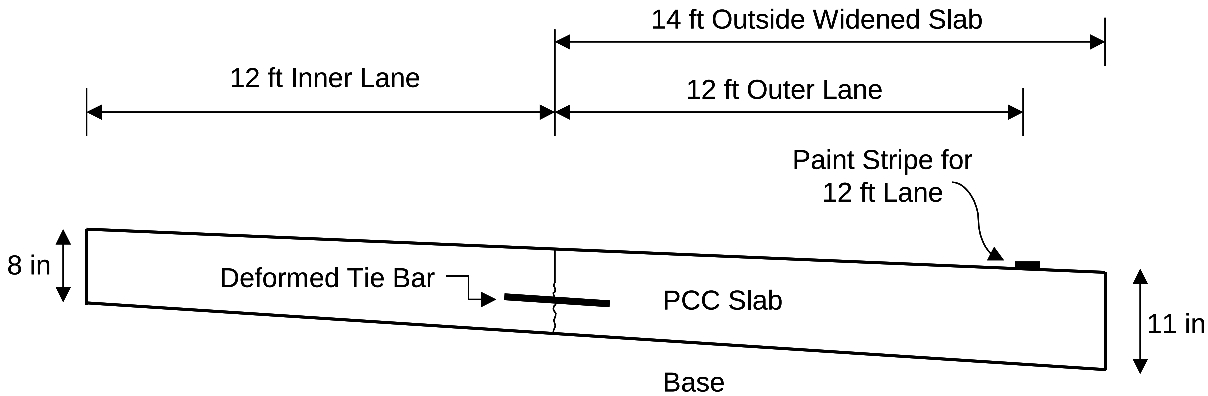

Figure 63. Variable cross section used on WI 3.

The following alternative dowel bar materials are also included on the WI 3 project (Crovetti 1999):

- Standard epoxy-coated dowel bars.
- FRP composite dowel bars, manufactured by MMFG.
- FRP composite dowel bars, manufactured by Glasforms.
- FRP composite dowel bars, manufactured by Creative Pultrusions.
- FRP composite dowel bars, manufactured by RJD Industries.
- Solid stainless steel dowel bars, manufactured by Slater Steels.

The nominal pavement design for these pavement sections is a 275-mm (11-in) JPCP with a uniform joint spacing of 5.5 m (18 ft). However, as previously described, one section has a variable thickness cross section, varying from 275 mm (11 in) for the outer lane, and then tapering to 203 mm (8 in) at the edge of the inner lane. The pavement rests on a 150-mm (6-in) crushed aggregate base course, and the transverse joints contain 38-mm (1.5-in) diameter dowels and are not sealed.

A total of six sections are included in the WI 3 project. The approximate layout of the WI 3 sections being monitored is shown in figure 64. All dowel bars were placed on baskets prior to paving (Crovetti 2001). It is noted that within the section incorporating various FRP composite dowel bars (Section FR), some of the composite dowel bars were improperly distributed between the 3.7-m (12-ft) and 4.3-m (14-ft) baskets, resulting in different manufacturers' bars being placed across some of the inner and outer traffic lanes (Crovetti 1999). The location of the different manufacturers' dowel bars is shown by lane in the blowup illustration in figure 64.

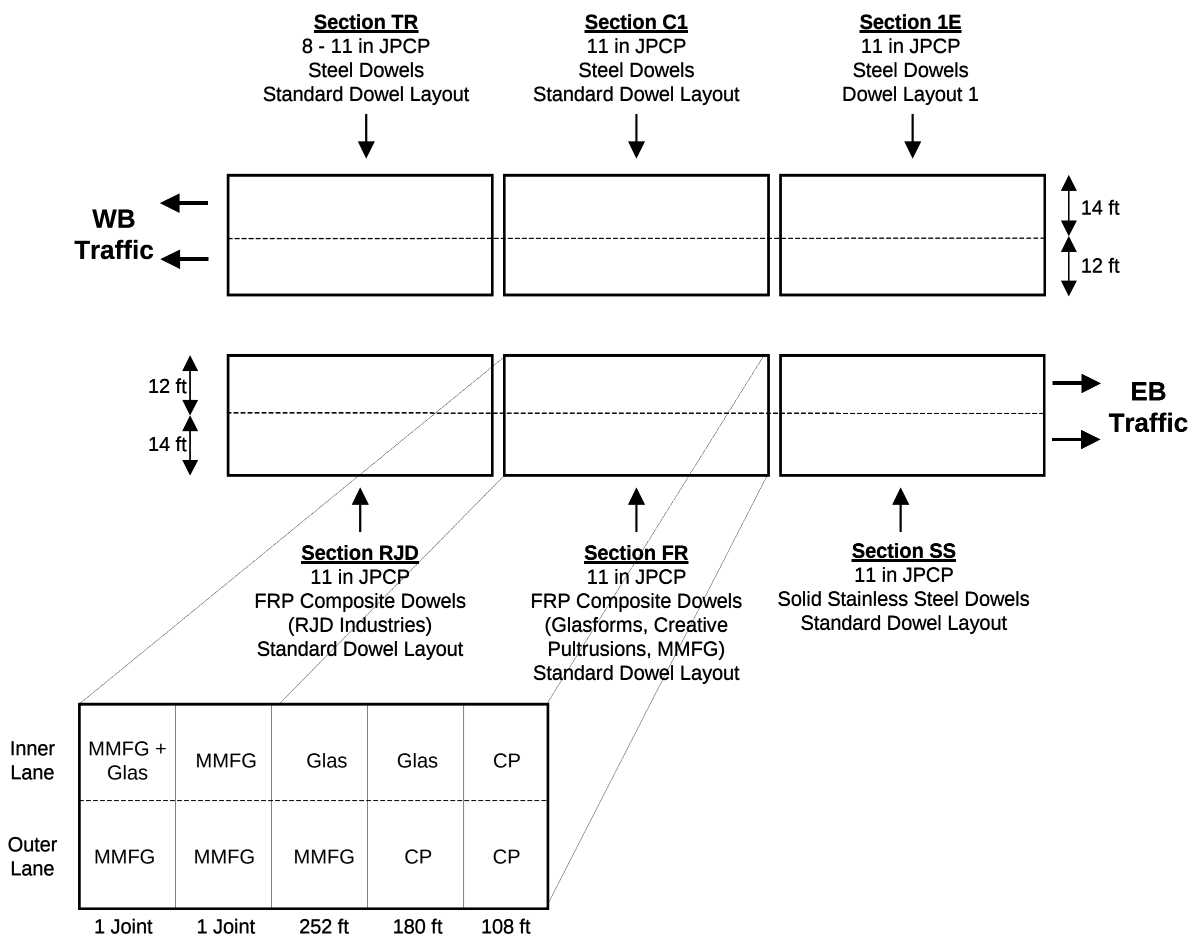

Figure 64. Approximate layout of WI 3 monitoring sections.

The experimental design matrix for the WI 3 project is shown in table 31. Most of the dowel materials are placed in the standard dowel layout, although one section is placed in alternative dowel layout 1. As previously mentioned, all of these sections are included in the SHRP study, and the SHRP code is provided in table 31 for each section.

Table 31. Experimental design matrix for WI 3.

	11-in JPCP 18-ft Joint Spacing		**8- to 11-in JPCP 18-ft Joint Spacing**
	Standard Dowel Layout	**Alternative Dowel Layout 1**	**Standard Dowel Layout**
Standard Epoxy-Coated Steel Dowels	Section C1 (SHRP 550259)	Section 1E (SHRP 550260)	Section TR (SHRP 550263)
Solid Stainless Steel Dowels *(Slater Steels)*	Section SS (SHRP 550265)		
FRP Composite Bars *(MMFG, Glasforms, Creative Pultrusions)*	Section FR (SHRP 550264A)		
FRP Composite Dowel Bars *(RJD Industries)*	Section RJD (SHRP 550264B)		

State Monitoring Activities

WisDOT, in conjunction with Marquette University, is monitoring the performance of these pavement test sections. These monitoring activities include (Crovetti 1999; Crovetti and Bischoff 2001):

- Dowel bar location study—conducted 2 months after construction.
- FWD testing—conducted immediately prior to paving, immediately after paving, and after 6 and 12 months of trafficking.
- Distress surveys—conducted immediately after paving and after 6 and 12 months of trafficking. The distress surveys are being conducted over a nominal 161-m (528-ft) pavement segment selected from within each test section.
- Ride quality surveys—conducted using a pavement profiler and measured on the sections after approximately 1 and 3 years of service.

Continued monitoring of these sections, in the form of FWD testing, distress surveys, and ride quality surveys, will continue through 2004 (Crovetti and Bischoff 2001).

Preliminary Results/Findings

Even though these sections are only 3 years old, some significant findings have been revealed through their early monitoring. These findings are described in the following sections by type of monitoring activity.

Construction Monitoring

A dowel bar inserter (DBI) was used during the construction of WI 2. The DBI easily accommodated the various types of dowel bar materials used in the study, and the DBI also accommodated the various dowel layout patterns with minimal disruption to the paving operations (Crovetti 1999).

Dowel Bar Location Study

With the purpose of determining the depth, longitudinal position, and transverse position of each dowel bar, a dowel bar location study was performed on the WI 2 project 2 months after construction using an impact echo device (Crovetti 1999). A summary of the results from the study are provided in table 32 (Crovetti 1999). Generally, it appears that the dowel bars are slightly deeper than the mid-depth of the slab (140 mm [5.5 in]), and that some vertical skewing of the dowels occurred across the joint. It should be noted that dowel depth data were inconclusive for the stainless steel tubes and the solid stainless steel dowels, and that the device could not provide exact longitudinal and transverse positions of each dowel end (Crovetti 1999).

Table 32. Summary of dowel bar location study results from WI 2 (Crovetti 1999).

Test Section	No. of Joints Tested	Average Depth, West Side of Joint, in	Average Depth, East Side of Joint, in	Average Depth Variation, in
C1 (epoxy-coated steel dowel)	1	6.04	5.86	0.18
CP (FRP composite dowel)	2	6.17	5.97	0.21
GF (FRP composite dowel)	5	6.12	6.00	0.47
RJD (FRP composite dowel)	7	6.04	6.05	0.20

FWD Testing

FWD testing has been conducted several times since the construction of these test sections. Table 33 summarizes the backcalculated k-value and concrete elastic modulus, as well as the total joint deflection (defined as the sum of the deflections from both the loaded and unloaded sides of the joint) obtained from the FWD testing (Crovetti 1999). Generally, the test results are fairly consistent over time, although greater variability was noticed in the June 1998 tests for both directions, presumably because of higher slab temperature gradients (Crovetti 1999). Apparent increases in total joint deflections may be due to FWD testing conducted in the early morning when upward slab curling is likely.

Table 33. Summary of FWD test results for WI 2 and WI 3 projects (Crovetti 1999).

Property	WI 2			WI 3				
	EB lanes			EB lanes			WB lanes	
	Oct 97	Jun 98	Nov 98	Oct 97	Jun 98	Nov 98	Jun 98	Nov 98
Dynamic k-value, $lbf/in^2/in$	312	255	254	364	324	324	255	222
PCC Elastic Modulus, lbf/in^2	3,560,000	3,870,000	4,820,000	3,970,000	5,990,000	6,060,000	5,290,000	6,130,000
Total 9000-lb Joint Deflection, mils	8.96	7.77	8.18	6.70	5.56	8.48	6.23	7.11

Transverse joint load transfer efficiencies were also measured on all test sections using the FWD. Figure 65 illustrates the average transverse joint load transfer for the outermost wheelpath of the WI 2 project, while figure 66 illustrates the average transverse joint load transfer for the outermost wheelpath of the WI 3 project (Crovetti 1999). For WI 2, the late season tests (October 1997 and November 1998) indicate significantly reduced LTE in the composite doweled sections and in dowel layout 1 as compared to the control sections (Crovetti 1999). However, LTE measured in the summer do not indicate any significant differences within the test sections, probably because of the increased aggregate interlock brought about by the closing of the joints due to the warmer temperatures (Crovetti 1999).

For WI 3, figure 66 shows that the FRP composite dowel sections and dowel layout 1 experience a reduction in LTE in the November 1998 test results; there is also a slight reduction in the LTE of the stainless steel section (Crovetti 1999). However, LTE measured in June 1998 do not indicate any significant differences between the test sections.

Distress Surveys

Distress surveys were conducted for both WI 2 and WI 3 in June and December 1998. Some joint distress (spalling, chipping, and fraying of the transverse joints) was observed and is primarily attributable to the joint sawing operations that dislodged aggregate particles near the joint faces (Crovetti and Bischoff 2001). However, this joint spalling has not yet progressed to the point to be considered as low severity based on the Wisconsin DOT Pavement Distress guidelines (Crovetti and Bischoff 2001). Other than the minor joint spalling, no transverse faulting, slab cracking, or other surface distress has been observed to date (Crovetti and Bischoff 2001).

Ride Quality Surveys

Figure 67 presents the average international roughness index (IRI) measurements in the outer lane of the WI 2 and WI 3 pavement sections (Crovetti and Bischoff 2001). These measurements were recorded in the summer of 1998 and the winter of 2000. Although there is some variability in the data, most of the test sections are performing comparably to the control sections (Crovetti and Bischoff 2001).

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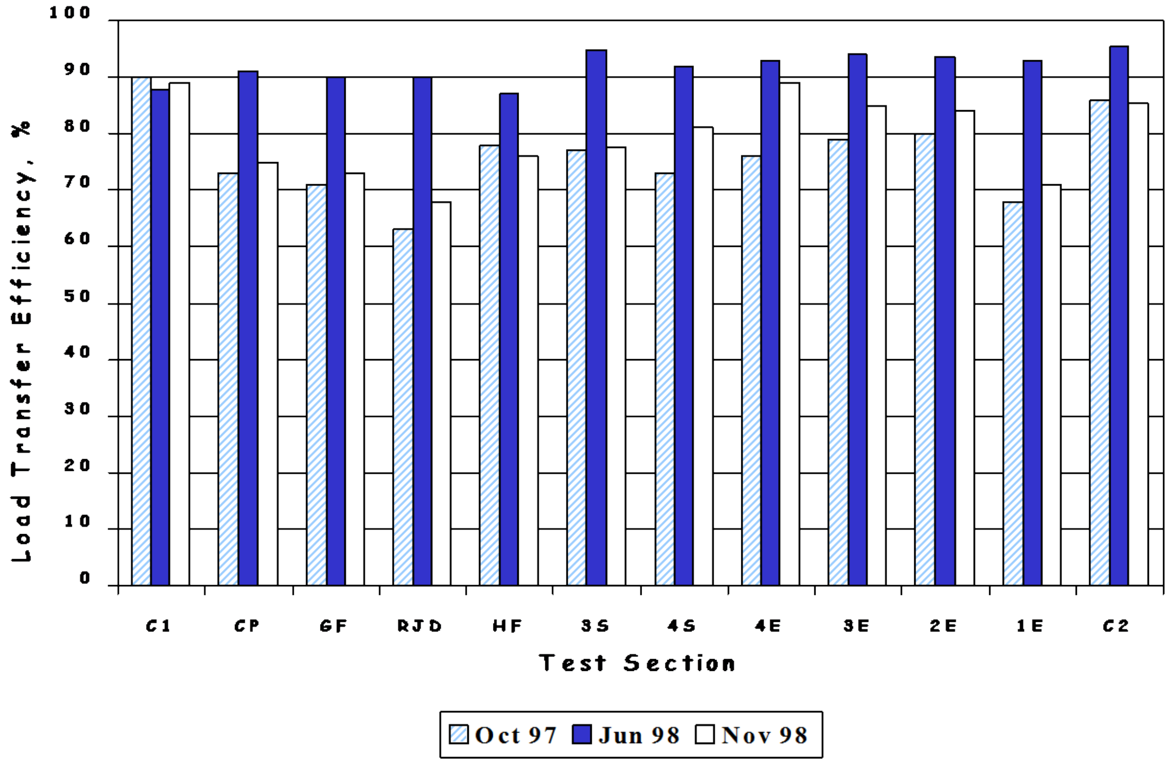


Figure 65. Transverse joint load transfer for outermost wheelpath on WI 2 (Crovetti 1999).

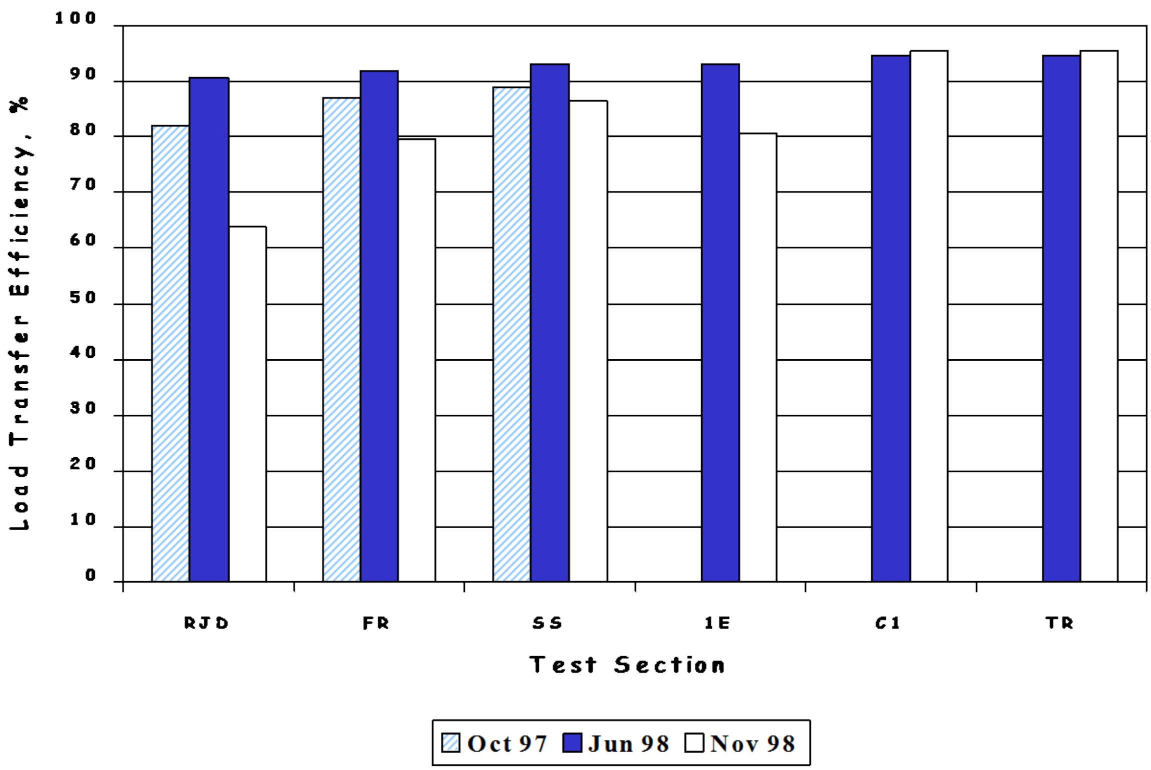


Figure 66. Transverse joint load transfer for outermost wheelpath on WI 3 (Crovetti 1999).

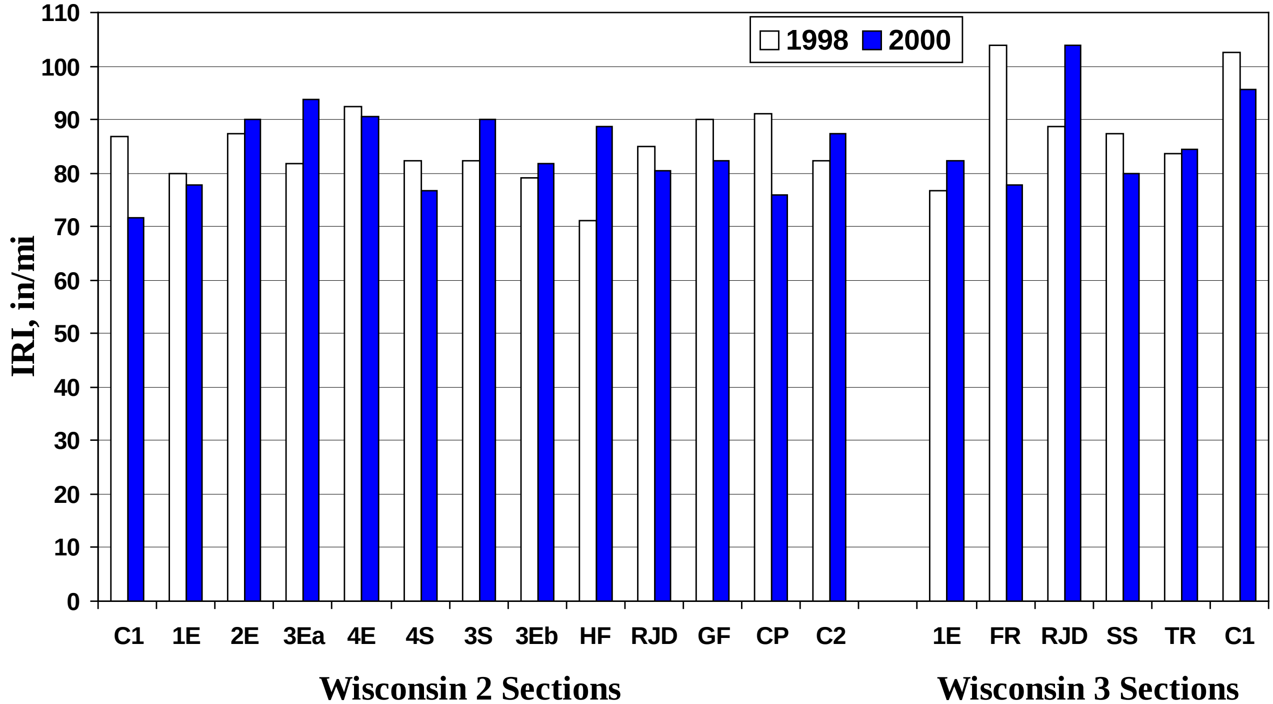


Figure 67. Average IRI values in the outer traffic lanes of WI 2 and WI 3 pavement sections (Crovetti and Bischoff 2001).

References

Crovetti, J. A. 1999. *Cost Effective Concrete Pavement Cross-Sections*. Report No. WI/SPR 12-99. Wisconsin Department of Transportation, Madison, WI.

Crovetti, J. A. and D. Bischoff. 2001. "Construction and Performance of Alternative Concrete Pavement Designs in Wisconsin." *Preprint Paper No. 01-2782*. 80th Annual Meeting of the Transportation Research Board, Washington, DC.

APPENDIX E
REPORT ON WISCONSIN FIELD TESTING ACTIVITIES

1.0 Introduction

This report presents the results of testing conducted in support of pooled fund study TPF-5(188), "Evaluation of Fiber Reinforced Composite Dowel Bars and Stainless Steel Dowel Bars." Study objectives of TPF-5(188) include coring of alternative dowel bars, coring of epoxy-coated dowel bars that have been in service for at least 15 years, and chloride analysis of cores for all types of dowel bars. Eight separate projects constructed in Wisconsin were included in this study. Table 1 provides a listing of these projects providing location, year of construction, and included dowel bar materials.

Table 1: Wisconsin Projects Listing

Project Location	Year of Construction	Included Dowel Bar Materials
STH 67 – Waukesha Co.	1976	Epoxy Coated Steel
IH 43 – Sheboygan Co.	1979	Epoxy Coated Steel
IH 94 – Dunn Co.	1984	Epoxy Coated Steel
USH 18/151 – Dane Co.	1989	Epoxy Coated Steel
STH 29 – Brown Co.	1989	Epoxy Coated Steel
STH 16 – Waukesha Co.	1990	Epoxy Coated Steel
STH 29 – Chippewa Co.	1994	Epoxy Coated Steel
STH 29 – Clark Co.	1997	Epoxy Coated Steel Solid Stainless Steel Mortar-Filled Stainless Steel Tubes Fiber Reinforced Polymer (FRP)

Field testing included falling weight deflectometer (FWD) testing (4 project locations) and the extraction of 6-inch diameter cores at selected dowel locations (all project locations). Extracted cores were transported to Marquette University for documentation and preparation of samples for chloride content testing. Chloride testing was conducted by Giles Engineering Associates, Inc. per ASTM C 1152, *Test Method for Acid-Soluble Residue in Mortar and Concrete*.

STH 67 – Waukesha County

This portion of STH 67 includes the southbound lanes constructed in 1976 from Pabst Road to Valley Road/CTH B in the Town of Oconomowoc. The project was overlaid prior to coring in November, 2009. Coring was conducted on November 24, 2009. A total of six cores were extracted from two sets of three consecutive joints randomly selected within the project limits. No FWD testing was conducted prior to the coring operations.

IH 43 – Sheboygan County

This portion of IH 43 includes the southbound lanes constructed in 1979 before and after the bridge structure over STH 23 in the City of Sheboygan. Coring was conducted on November 19, 2009. A total of six cores were extracted from two sets of three consecutive joints randomly selected within the project limits. Due to traffic control restrictions, cores were extracted near the median shoulder of the passing lane. No FWD testing was conducted prior to the coring operations.

IH 94 – Dunn County

This portion of IH 94 includes the eastbound lanes constructed in 1984 from Airport Rd/390th Street to Wilson Creek Structure, which is just west of STH 25 interchange near the City of Menomonee. Coring was conducted on October 19, 2009. A total of six cores were extracted from two sets of three consecutive joints randomly selected within the project limits. FWD testing was conducted prior to the beginning of coring operations. However, equipment problems only allowed for the testing at the approach joint location on five consecutive joints.

USH 18/151 – Dane County

This portion of USH 18/151 includes the eastbound lanes constructed in 1989 from Cave of the Mounds Road to STH 78 near the City of Mount Horeb. This portion of USH 18/151 includes unsealed random skewed transverse joints and both open graded (Section 13) and dense graded (Section 14) aggregate base layer sections. FWD testing and coring was conducted on June 16, 2009. FWD testing was conducted prior to the beginning of coring operations at five consecutive approach joint locations in each section. A total of six cores were extracted from three consecutive joints within section 13, with cores extracted from both the center lane and outside wheelpath locations. A total of three cores were extracted from two consecutive joints within section 14, with cores extracted from both the center lane and outside wheelpath locations. Coring equipment problems precluded the removal of the remaining three cores targeted for extraction.

STH 29 – Brown County

This portion of STH 29 includes the eastbound lanes constructed in 1989 from Triangle Drive to Sunlite Drive just west of the City of Green Bay. This portion of STH 29 includes neoprene sealed (Section 3) and unsealed (Section 4) random skewed transverse joints over an open graded base layer. FWD testing and coring was conducted on June 30, 2009. FWD testing was conducted prior to the beginning of coring operations at six consecutive approach joint locations in each section. A total of three cores were extracted from three consecutive joints within each section, with cores extracted from the outside wheelpath location.

STH 16 – Waukesha County

This portion of STH 16 includes the eastbound lanes constructed in 1990 between Gifford Rd/Brown St to CTH P near the Town of Oconomowoc. Coring was conducted on April 20, 2009. A total of three cores were extracted from three consecutive joints randomly selected within the project limits. No FWD testing was conducted prior to the coring operations.

STH 29 – Chippewa County

This portion of STH 29 includes the eastbound lanes constructed in 1994 between CTH X and STH 27 near the Town of Cadott. Coring was conducted on October 20, 2009. A total of six cores were extracted from two sets of three consecutive joints randomly selected within the project limits. No FWD testing was conducted prior to the coring operations due to equipment problems.

STH 29 – Clark County

This portion of STH 29 includes the eastbound lanes constructed in 1997 between Owens and Abbotsford, specifically from CTH X/Cardinal Ave to CTH E near the Town of Curtiss. This portion of STH 29 includes a widened outer lane (14 ft paved width striped at 12 ft) with unsealed random skewed transverse joints over a dense graded base layer. Dowels materials include standard epoxy coated steel (CTL), solid stainless steel (SS), mortar-filled stainless steel tubes (HF), and fiber reinforced polymer (FRP) materials manufactured by Creative Pultrusions (CP), Glasforms (GF) and RJD Industries (RJ). FWD testing and coring was conducted on October 21, 2009. FWD testing was conducted prior to the beginning of coring operations at eight consecutive approach and leave joint locations in each section. The FWD joint testing was conducted in the outer wheel path (over dowel 4) and the results were used to select joints for coring. A total of four cores were extracted from two joints within the control (CTL) section, with cores extracted from the outside edge (dowel 3) and left centerlane location (dowel 10). These positions correspond to dowel locations 1 and 8 for a standard 12 ft wide pavement lane. A total of two cores were extracted from one joint within each of the remaining five sections (SS, HF, CP, GF, RJ), with cores extracted from the outside edge (dowel 3) and left centerlane location (dowel 10).

2.0 Core Documentation and Sample Preparation

All collected pavement cores were transported to Marquette University for documentation and chloride content sample preparation. Photographs were taken of the cores, dowels and joint faces to document the condition of each. Notes on the condition of the concrete and dowels materials were recorded. Attachments A through H provide summary sheets for each collected core specimen.

After documentation, chloride content specimens were prepared using a rotary hammer drill. The normal protocol included the following steps:

1- The core sample was separated and the joint faces brushed to remove loose materials, typically composed of fine soil particles. The core section was then positioned in a stainless steel catch pan.
2- A clean, dry ¼” masonry bit was positioned on a mortar area slightly above the dowel level on one of the joint faces and penetrated for approximately ½“ to produce a powdered sample of sufficient size.
3- The powdered materials were transferred from the catch pan and passed through a No. 20 sieve. The materials passing were captured in a brass sieve pan and mixed to ensure homogeneity. A 5g sample was obtained and transferred to a clean plastic sample bag.
4- The masonry bit, sieve and catch pans were brushed clean and Steps 2 and 3 were repeated on the other joint face to obtain a second 5g sample.
5- The two 5g samples were combined to obtain a single 10g sample representing the joint face materials.
6- The catch pans and sieve were brushed clean and a second clean, dry ¼” masonry bit was positioned in a mortar area slightly above the dowel level on one of the exterior core sections and penetrated for approximately ½“ to produce a powdered sample of sufficient size.
7- The powdered materials were transferred from the catch pan and passed through a No. 20 sieve. The materials passing were captured in a brass sieve pan and mixed to ensure homogeneity. A 5g sample was obtained and transferred to a clean plastic sample bag.
8- The masonry bit, sieve and catch pans were brushed clean and Steps 6 and 7 were repeated on the other core section to obtain a second 5g sample.
9- The two 5g samples were combined to obtain a single 10g sample representing the outside core condition (approximately 3” in from the joint face)
10- The masonry bits, sieve, and catch pans were cleaned by dry brushing followed by a distilled water wash. All wetted surfaces were dried with a clean, disposable absorbent cloth and allowed to air dry before further use.

The above procedure was applied to all cores received. However, four of the specimens received from STH 67 – Waukesha Co. were in the form of completely disintegrated concrete with the extracted dowel and HMA overlay. Two of these specimens (Cores 1 & 2) contained sufficient fine materials to allow for the preparation of two 10g samples of P_{20} materials by simply dry sieving the disintegrated concrete materials. These samples were arbitrarily marked as joint face and outside samples. The remaining two disintegrated specimens (Cores 5 & 6) contained essentially clean gravel-sized materials with insufficient fine to produce test samples.

3.0 Chloride Content Testing

All prepared samples were transported to Giles Engineering Associates, Inc. Chloride testing was conducted per ASTM C 1152, *Test Method for Acid-Soluble Residue in Mortar and Concrete*. Table 2 provides a summary of the test results, including condition of the dowel bar and FWD test results, where available.

Table 2: Test Results

Highway Segment	Pavt Age	Joint # - Dowel Pos	FWD Test Results				Dowel Deterioration	Chloride Content	
			App LT%	App DT*	Lv LT%	Lv DT*		Joint Face	Outside
STH67-Waukesha	33	1-D3	n.a.	n.a.	n.a.	n.a.	Extensive*	0.0036	0.0047
STH67-Waukesha	33	2-D3	n.a.	n.a.	n.a.	n.a.	Extensive*	0.0220	0.0140
STH67-Waukesha	33	3-D3	n.a.	n.a.	n.a.	n.a.	Moderate	0.1025	0.1050
STH67-Waukesha	33	4-D3	n.a.	n.a.	n.a.	n.a.	Minor	0.0880	0.0570
I 43-Sheboygan	30	1-D3	n.a.	n.a.	n.a.	n.a.	Moderate	0.1050	0.0880
I 43-Sheboygan	30	2-D3	n.a.	n.a.	n.a.	n.a.	Minor	0.1500	0.1075
I 43-Sheboygan	30	3-D3	n.a.	n.a.	n.a.	n.a.	Extensive	0.0930	0.0920
I 43-Sheboygan	30	4-D3	n.a.	n.a.	n.a.	n.a.	Extensive	0.0845	0.1000
I 43-Sheboygan	30	5-D3	n.a.	n.a.	n.a.	n.a.	None	0.1300	0.1400
I 43-Sheboygan	30	6-D3	n.a.	n.a.	n.a.	n.a.	Extensive	0.1725	0.1125
I 94 - Dunn	25	3-D3	87	8.2	n.a.	n.a.	Extensive	0.1700	0.1475
I 94 - Dunn	25	4-D3	86	8.9	n.a.	n.a.	Minor	0.0087	0.2300
I 94 - Dunn	25	5-D3	87	8.4	n.a.	n.a.	Moderate	0.1700	0.1450
I 94 - Dunn	25	40-D3	n.a.	n.a.	n.a.	n.a.	Moderate	0.1600	0.2150
I 94 - Dunn	25	41-D3	n.a.	n.a.	n.a.	n.a.	Extensive	0.2150	0.2600
I 94 - Dunn	25	42-D3	n.a.	n.a.	n.a.	n.a.	Moderate	0.2325	0.2450
STH29-Brown	20	S3-Jt4-D3	94	14.4	n.a.	n.a.	Extensive	0.0520	0.0240
STH29-Brown	20	S3-Jt5-D3	93	14.9	n.a.	n.a.	None	0.1200	0.0480
STH29-Brown	20	S3-Jt6-D3	90	16.1	n.a.	n.a.	MOD	0.0790	0.0480
STH29-Brown	20	S4-Jt1-D3	88	11.5	n.a.	n.a.	MOD	0.0695	0.0360
STH29-Brown	20	S4-Jt2-D3	93	13.7	n.a.	n.a.	MOD	0.1125	0.0580
STH29-Brown	20	S4-Jt3-D3	95	12.9	n.a.	n.a.	Minor	0.0720	0.0415
USH18/151-Dane	20	13-JT1-D10	91	8.2	n.a.	n.a.	Minor	0.1075	0.1425
USH18/151-Dane	20	13-JT1-D3	n.a.	n.a.	n.a.	n.a.	Minor	0.1400	0.1300
USH18/151-Dane	20	13-JT2-D10	94	13.1	n.a.	n.a.	None	0.2000	0.1550
USH18/151-Dane	20	13-JT2-D3	n.a.	n.a.	n.a.	n.a.	Minor	0.2000	0.1650
USH18/151-Dane	20	13-JT3-D10	91	7.5	n.a.	n.a.	None	0.1625	0.1450
USH18/151-Dane	20	13-JT3-D3	n.a.	n.a.	n.a.	n.a.	Minor	0.1300	0.1225
USH18/151-Dane	20	14-JT1-D10	91	8.9	n.a.	n.a.	NA	0.1625	0.1250
USH18/151-Dane	20	14-JT1-D3	n.a.	n.a.	n.a.	n.a.	NA	0.0820	0.1500
USH18/151-Dane	20	14-JT2-D10	89	6.4	n.a.	n.a.	NA	0.0990	0.1225
STH16-Waukesha	19	1-D3	n.a.	n.a.	n.a.	n.a.	Extensive	0.0950	0.0510
STH16-Waukesha	19	2-D3	n.a.	n.a.	n.a.	n.a.	Moderate	0.1075	0.0910
STH16-Waukesha	19	3-D3	n.a.	n.a.	n.a.	n.a.	NA	0.0940	0.0350
STH29-Chippewa	15	1-D3	n.a.	n.a.	n.a.	n.a.	None	0.4000	0.1950
STH29-Chippewa	15	2-D3	n.a.	n.a.	n.a.	n.a.	Minor	0.3125	0.1125
STH29-Chippewa	15	3-D3	n.a.	n.a.	n.a.	n.a.	Moderate	0.3125	0.2000
STH29-Chippewa	15	4-D3	n.a.	n.a.	n.a.	n.a.	Minor	0.3050	0.0910
STH29-Chippewa	15	5-D3	n.a.	n.a.	n.a.	n.a.	Minor	0.4000	0.1500
STH29-Chippewa	15	6-D3	n.a.	n.a.	n.a.	n.a.	Minor	0.3650	0.1500
STH 29-Clark	12	CTL-JT6-D10	n.a.	n.a.	n.a.	n.a.	Minor	0.1225	0.1600
STH 29-Clark	12	CTL-JT6-D3	90	9.4	91	9.2	None	0.0780	0.0820
STH 29-Clark	12	CTL-JT7-D10	n.a.	n.a.	n.a.	n.a.	Extensive	0.1675	0.1725
STH 29-Clark	12	CTL-JT7-D3	87	9.8	80	10.4	None	0.1500	0.1450
STH 29-Clark	12	CP-JT3-D10	n.a.	n.a.	n.a.	n.a.	None*	0.1475	0.1075
STH 29-Clark	12	CP-JT3-D3	61	9.6	66	10	None*	0.0440	0.0720
STH 29-Clark	12	GF-JT7-D10	n.a.	n.a.	n.a.	n.a.	None*	0.1525	0.0900
STH 29-Clark	12	GF-JT7-D3	78	10.5	74	10.7	None*	0.1275	0.0380
STH 29-Clark	12	RJ-JT1-D10	n.a.	n.a.	n.a.	n.a.	Minor	0.1475	0.1250
STH 29-Clark	12	RJ-JT1-D3	75	10.6	69	11	None*	0.1125	0.0970
STH 29-Clark	12	HF-JT8-D10	n.a.	n.a.	n.a.	n.a.	None*	0.0980	0.1350
STH 29-Clark	12	HF-JT8-D3	85	9.7	86	9.5	None*	0.1250	0.0880
STH 29-Clark	12	SS-JT5-D10	n.a.	n.a.	n.a.	n.a.	None*	0.1275	0.1475
STH 29-Clark	12	SS-JT5-D3	50	9.0	60	8.8	None*	0.1075	0.0800

App DT* and Lv DT* = total joint deflection in mils normalized to a load of 9,000 lb.

None* = Fiber Reinforced Polymer Dowel, Extensive* indicates samples where no intact concrete was available.

NA indicates core sample where no dowel bar was provided.

4.0 Discussion of Results

The aggregate results of the chloride content tests were compared against levels of dowel bar deterioration as noted for the epoxy coated steel dowels. Figure 1 provides a bar chart of the overall average and median results comparing dowel deterioration to chloride concentration levels. As shown, the overall average and median chloride concentrations are higher for the joint face samples than for the outside core samples for all test bins. However, there is no direct correlation between dowel deterioration and chloride concentrations; in fact overall average and median chloride concentrations are greatest for samples with none to minor dowel deterioration.

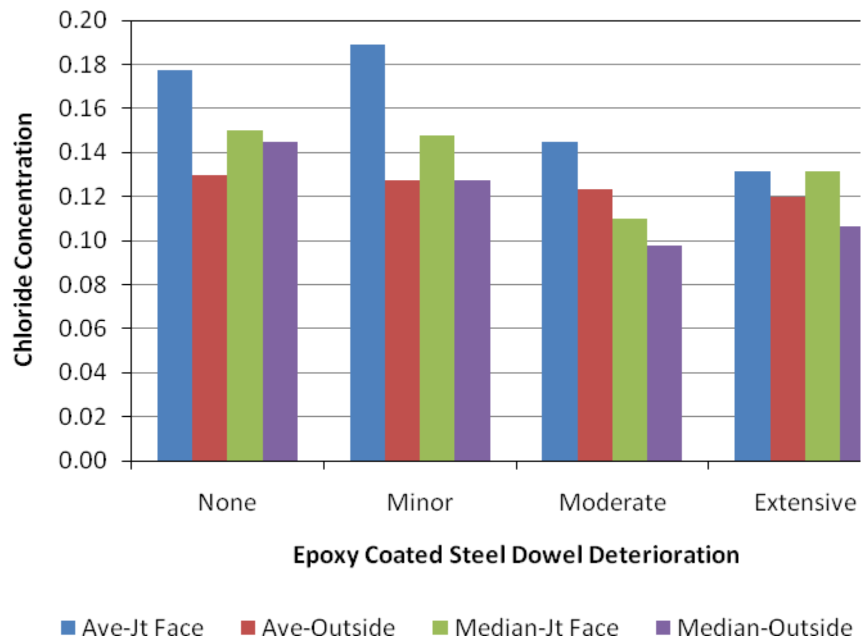


Figure 1: Chloride Concentration vs Dowel Deteioration

The effect of joint sealant and pavement age on chloride concentration levels is presented in Figures 2 and 3, with each data point representing and individual core sample test result. As shown, there appears to be little correlation between chloride concentration levels and pavement age for the unsealed joints. For those joints with neoprene sealants, chloride concentrations levels are generally lower than comparable measures for unsealed joints and these levels also increase with pavement age.

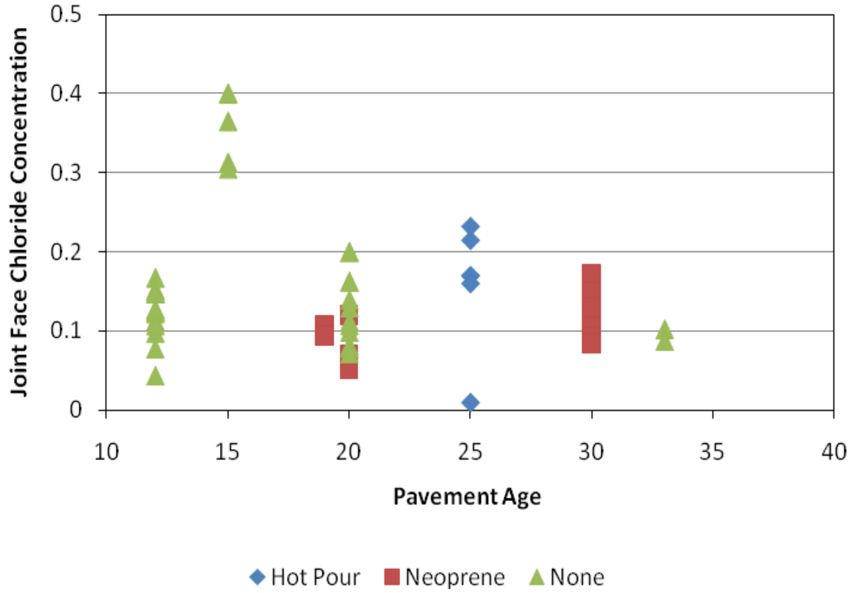


Figure 2: Joint Face Chloride Concentration vs Pavement Age

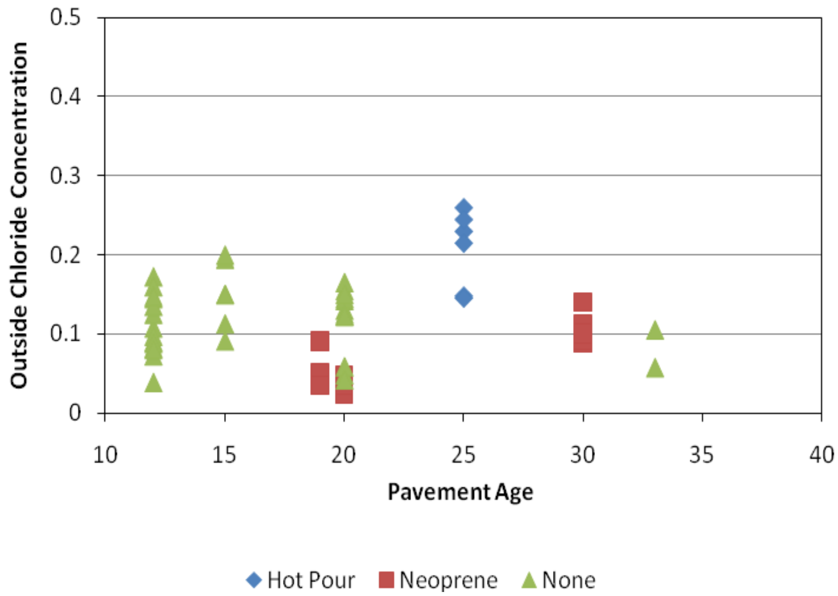


Figure 3: Outside Chloride Concentration vs Pavement Age

Figure 4 presents the chloride concentration ratio versus pavement age, with the concentration ratio computed as:

Chloride Concentration Ratio = Joint Face Concentration / Outside Concentration

As shown, the concentration levels generally decrease with age, confirming the expected trend that the chloride ions penetrate further into the concrete over time.

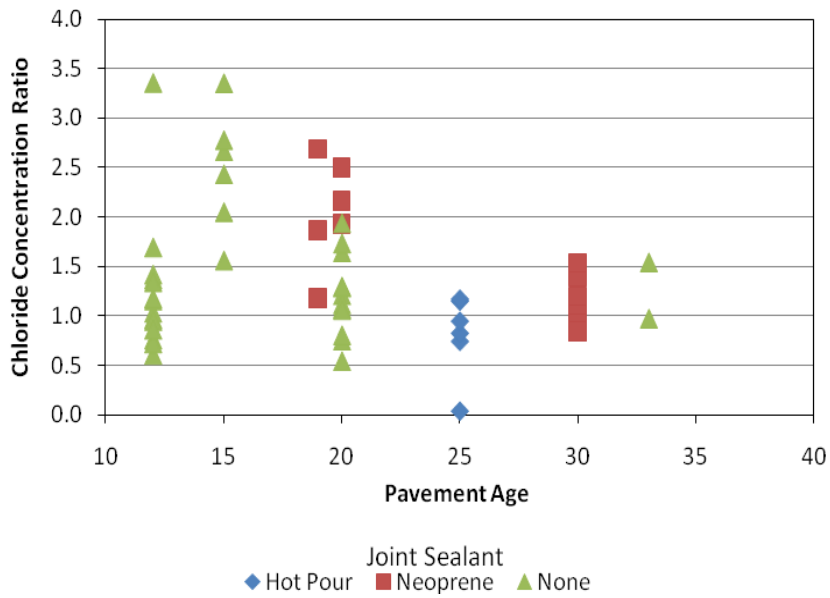


Figure 4: Chloride Concentration Ratio vs Pavement Age

ATTACHMENT A

Core Summary Sheets

STH 67 – Waukesha County

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH67-Waukesha-C1**	
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	Unable to see	
Corrosion of dowel at concrete interface	Extensive	
Abrasion of the dowel surface at the crack face	Extensive	
Dowel Deterioration	Extensive	
Dowel Type	Epoxy Coated	
PCC Deterioration	Complete deterioration, only AC layer intact.	
PCC Stain		
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: All the pieces of the PCC were collected in a bag. The dowel bar is loose and the AC layer by itself.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH67-Waukesha-C2**	        
Core Intact, Y or N	No, PCC all desintegratedNA	
Type of Joint Sealant	NA	
Socketing around dowel	NA	
Corrosion of dowel at concrete interface	Extensive	
Abrasion of the dowel surface at the crack face	Extensive	
Dowel Deterioration	Extensive	
Dowel Type	Epoxy Coated	
PCC Deterioration	Extensive	
PCC Stain	NA	
Signs of Defect in Core-holes		
Date when core was taken		
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The aggregate is all in pieces.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH67-Waukesha-C3**	         
Core Intact, Y or N	No (2 pieces)	
Type of Joint Sealant	NA	
Socketing around dowel	Minor	
Corrosion of dowel at concrete interface	Moderate in one end	
Abrasion of the dowel surface at the crack face	Moderate	
Dowel Deterioration	Moderate	
Dowel Type	Epoxy Coated	
PCC Deterioration	Moderate	
PCC Stain	Small dark spots in one side	
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		

General Comments:
AC Overlay and dowel are attached to the PCC.

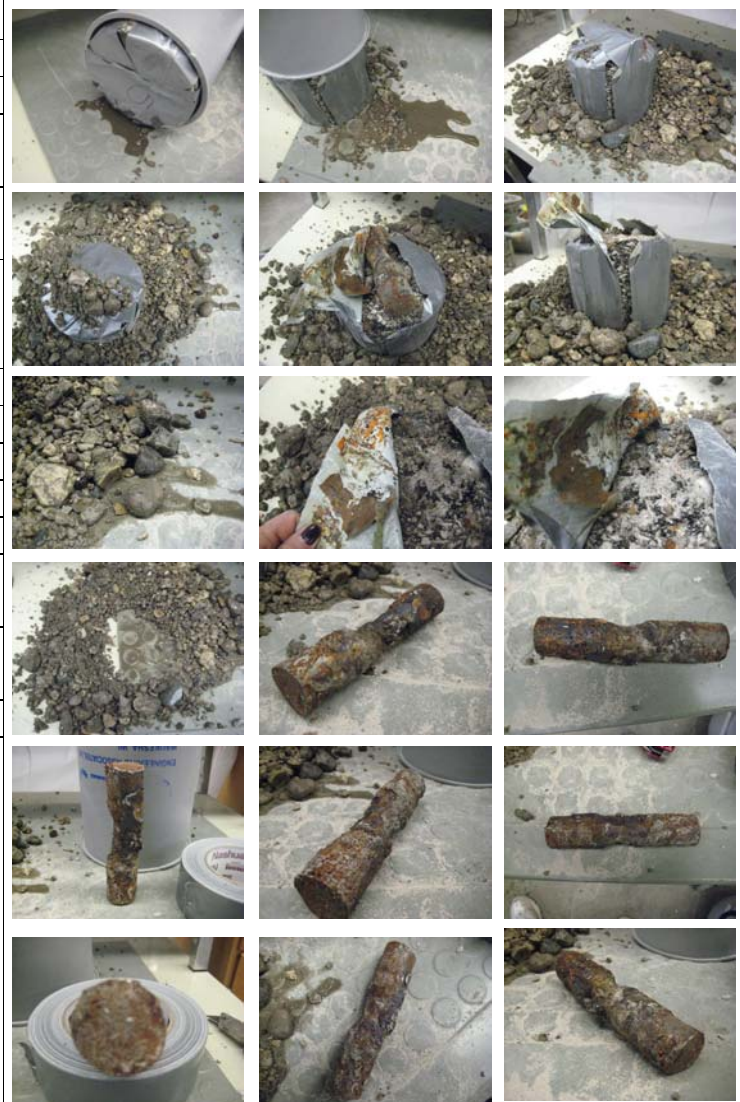
Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH67-Waukesha-C4**	
Core Intact, Y or N	Yes	
Type of Joint Sealant	NA	
Socketing around dowel	Yes	
Corrosion of dowel at concrete interface	Minor	
Abrasion of the dowel surface at the crack face	Minor, some orange color spots at ends	
Dowel Deterioration	Minor	
Dowel Type	Epoxy Coated	
PCC Deterioration	No	
PCC Stain	Yes, dark and brown colors at dowel placement areas	
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The AC Overlay layer is attached to PCC as well as the dowel.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH67-Waukesha -C5**	
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	Unable to see	
Corrosion of dowel at concrete interface	Extensive	
Abrasion of the dowel surface at the crack face	Extensive	
Dowel Deterioration	Extensive	
Dowel Type	Epoxy Coated	
PCC Deterioration	Extensive	
PCC Stain	Unable to see	
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		

General Comments:
The PCC core is disintegrated and there is only the layer of AC as an overlay. The dowel is loose and the PCC is gravel material. See photos.
The core was wrapped with duck tape. After unwrapping, there was a lot of water running through the sample as seen in photos,

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH67-Waukesha-C6**	            
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	Extensive	
Corrosion of dowel at concrete interface	Extensive	
Abrasion of the dowel surface at the crack face	Extensive	
Dowel Deterioration	Extensive	
Dowel Type	Epoxy Coated	
PCC Deterioration	Yes, complete disintegration	
PCC Stain		
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The core has an AC overlay layer. The PCC portion is disintegrated completely. After the duck tape was removed, water was found on the dowel and around that area. The water seemed to come from condensation (all wrapped with duck tape) from the coring process.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

ATTACHMENT B

Core Summary Sheets

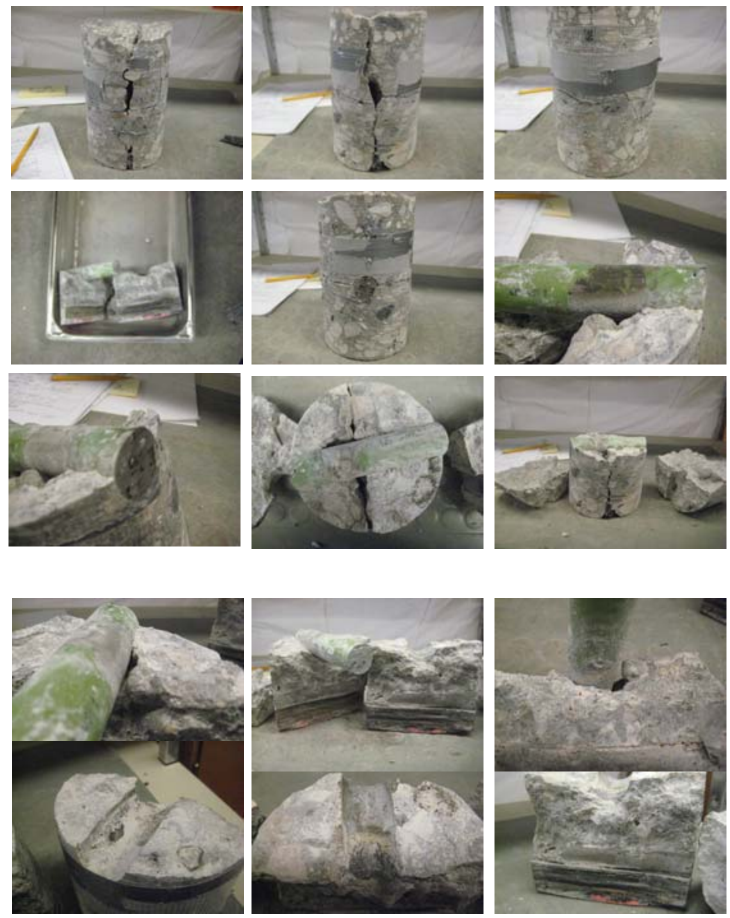
IH 43 – Sheboygan County

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**IH43-Sheboygan-Jt1-D11**	
Core Intact, Y or N	No (3 Pieces)	
Type of Joint Sealant	Neoprene	
Socketing around dowel	Yes, moderate	
Corrosion of dowel at concrete interface	Yes, minor	
Abrasion of the dowel surface at the crack face	Extensive at joint	
Dowel Deterioration	Moderate	
Dowel Type	Epoxy Coated	
PCC Deterioration	Yes in some areas	
PCC Stain	Yes, dark spots	
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The core came in 3 big pieces; the dowel is still attached to one of the big pieces.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**IH43-Sheboygan-Jt2-D11**	
Core Intact, Y or N	No (3 pieces)	
Type of Joint Sealant	Neoprene	
Socketing around dowel	Yes, minor	
Corrosion of dowel at concrete interface	Minor	
Abrasion of the dowel surface at the crack face	Minor	
Dowel Deterioration	Minor	
Dowel Type	Epoxy Coated	
PCC Deterioration	No	
PCC Stain	Yes, starting to show some yellowish spots, there is one dark area	
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The bottom was opened with a hammer to be able to see the dowel. There are a total of 4 pieces.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description
Core Number	**IH43-Sheboygan-Jt3-D11**
Core Intact, Y or N	No (3 pieces)
Type of Joint Sealant	Neoprene
Socketing around dowel	Extensive
Corrosion of dowel at concrete interface	Extensive
Abrasion of the dowel surface at the crack face	Extensive
Dowel Deterioration	Extensive
Dowel Type	Epoxy Coated
PCC Deterioration	Moderate inside core
PCC Stain	Yes, very dark in areas, some pieces from inside core are missing.
Signs of Defect in Core-holes	
Date when core was taken	NA
Dowel/Concrete Slab Interface	
Base Material Noted	

General Comments:
Part of the PCC pieces are attached to bar and dowel attached to bigger PCC piece.

Photos
























Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**IH43-Sheboygan-Jt4-D11**	
Core Intact, Y or N	Yes	
Type of Joint Sealant	Neoprene	
Socketing around dowel	Yes, some at the joint	
Corrosion of dowel at concrete interface	Extensive	
Abrasion of the dowel surface at the crack face	Extensive	
Dowel Deterioration	Extensive at joint	
Dowel Type	Epoxy Coated	
PCC Deterioration	No	
PCC Stain	Yes, at dowel and joint below dowel, see photos.	
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The core was broken with the electrical hammer to be able to see the dowel. There are 4 big pieces of PCC.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**IH43-Sheboygan-Jt5-D11**	
Core Intact, Y or N	No	
Type of Joint Sealant	Neoprene	
Socketing around dowel	Yes, some at the joint see photos	
Corrosion of dowel at concrete interface	No	
Abrasion of the dowel surface at the crack face	No	
Dowel Deterioration	No (at least at exposed areas of dowel)	
Dowel Type	Epoxy Coated	
PCC Deterioration	Minor	
PCC Stain	No	
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The dowel is still attached to main piece of PCC. There are a total of 3 big pieces, a lot of small pieces from the joint area. The PCC seems to be porous and crumbled. See photos.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**IH43-Sheboygan-Jt6-D11**	
Core Intact, Y or N	No	
Type of Joint Sealant	Neoprene	
Socketing around dowel	Yes	
Corrosion of dowel at concrete interface	Extensive	
Abrasion of the dowel surface at the crack face	Extensive	
Dowel Deterioration	Extensive	
Dowel Type	Epoxy Coated	
PCC Deterioration	Yes	
PCC Stain	Yes, some dark areas	
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: There is a lot of loose aggregate. See photos.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

ATTACHMENT C

Core Summary Sheets

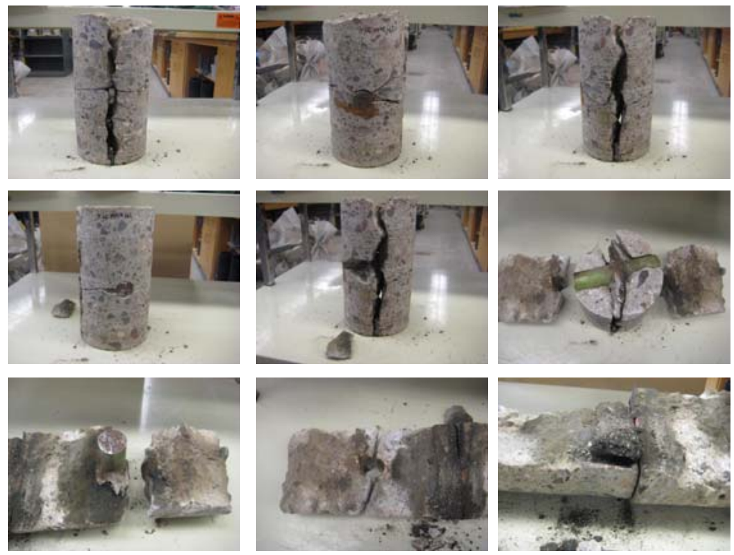
IH 94 – Dunn County

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	**Description**	**Photos**
Core Number	**IH94-Dunn-Jt3-D3**	
Core Intact, Y or N	No	
Type of Joint Sealant	Neoprene	
Socketing around dowel	Some slightly	
Corrosion of dowel at concrete interface	Extensive	
Abrasion of the dowel surface at the crack face	Extensive	
Dowel Deterioration	Extensive	
Dowel Type	Epoxy Coated	
PCC Deterioration	Moderate	
PCC Stain	Dark colors around the dowel	
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: There are 4 big pieces, one medium piece attached to corroded dowel. The rest of small pieces were collected in a bag.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**IH94-Dunn-Jt4**	
Core Intact, Y or N	No	
Type of Joint Sealant	Liquid Hot Poured AC	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	Minor	
Abrasion of the dowel surface at the crack face	Minor	
Dowel Deterioration	Minor	
Dowel Type	Epoxy Coated	
PCC Deterioration	Yes	
PCC Stain	Yes, dark browns	
Signs of Defect in Core-holes		
Date when core was taken	10-19-2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: PCC dark at joint and dowel area. Corrosion of dowel mainly at joint, dark browns. PCC broken into 5 pieces.		

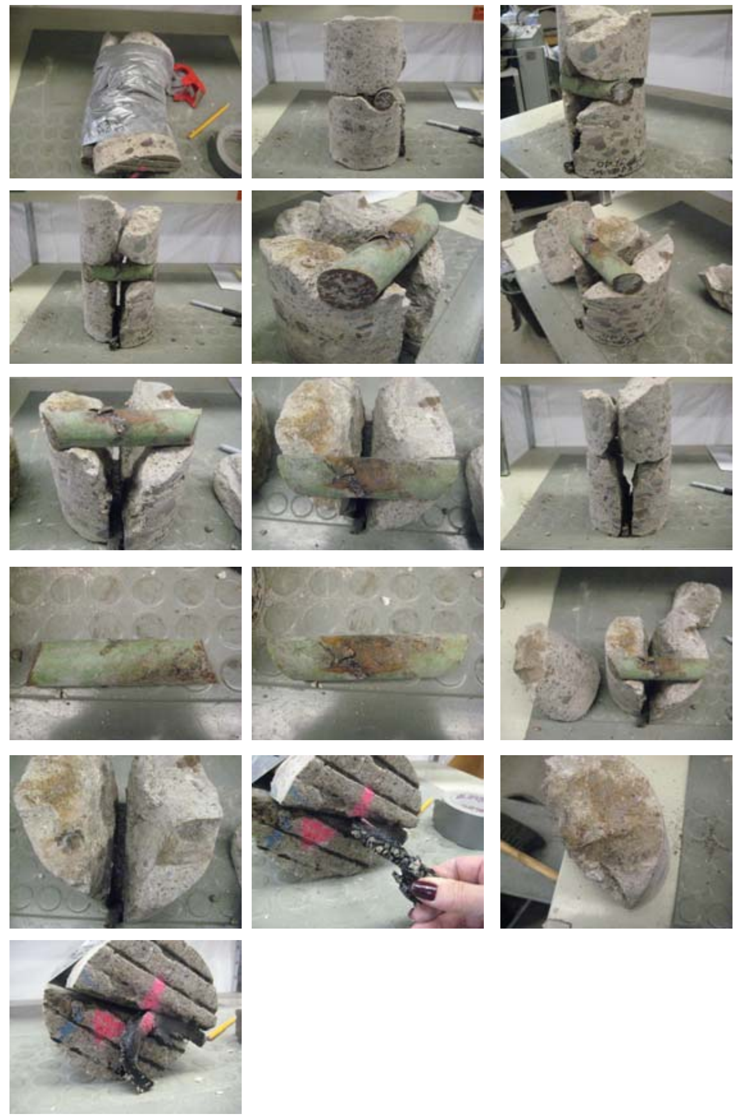
Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**IH94-Dunn-Jt5-D3**	
Core Intact, Y or N	No	
Type of Joint Sealant	Neoprene	
Socketing around dowel	Minor	
Corrosion of dowel at concrete interface	Moderate	
Abrasion of the dowel surface at the crack face	Moderate	
Dowel Deterioration	Moderate	
Dowel Type	Epoxy Coated	
PCC Deterioration	Moderate (core is crumbled)	
PCC Stain	No visible to the pieces available	
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: There are 3 big pieces, a lot of little pieces and dowel is attached to the big piece.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**IH94-Dunn-Jt40**	
Core Intact, Y or N	No (4 pieces)	
Type of Joint Sealant	Liquid Hot Poured	
Socketing around dowel	Very little	
Corrosion of dowel at concrete interface	Moderate	
Abrasion of the dowel surface at the crack face	Moderate	
Dowel Deterioration	Moderate	
Dowel Type	Epoxy Coated	
PCC Deterioration	Minor	
PCC Stain	Some spots at dowel placing area	
Signs of Defect in Core-holes		
Date when core was taken	10-19-2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: There are 4 big pieces of PCC, dowel is loose and joint sealant still attached to PCC		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**IH94- Dunn-Jt41**	
Core Intact, Y or N	No	
Type of Joint Sealant	Hot Poured Asphalt	
Socketing around dowel	Some	
Corrosion of dowel at concrete interface	Extensive	
Abrasion of the dowel surface at the crack face	Extensive	
Dowel Deterioration	Extensive	
Dowel Type	Epoxy Coated	
PCC Deterioration	Some, top half missing	
PCC Stain	YES, dark browns and yellows, rust colors	
Signs of Defect in Core-holes		
Date when core was taken	10-19-2009	
Dowel/Concrete Slab Interface	NA	
Base Material Noted	No	
General Comments: 3 Big PCC pieces, 1 medium which was very stained, and a lot of small pieces hard to collect.		

Note: All photos were taken placing Cores Upside down to facilitate balance

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**IH94-Dunn-Jt42**	
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	Moderate	
Abrasion of the dowel surface at the crack face	Moderate	
Dowel Deterioration	Moderate	
Dowel Type	Epoxy Coated	
PCC Deterioration	5 main PCC pieces, a lot of crumbled concrete especially around the dowel. High quantity of disintegrated concrete.	
PCC Stain		
Signs of Defect in Core-holes		
Date when core was taken	10-19-2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments:		

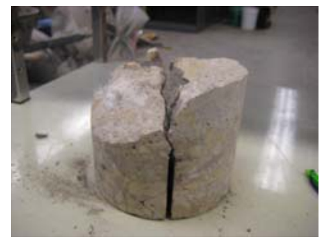
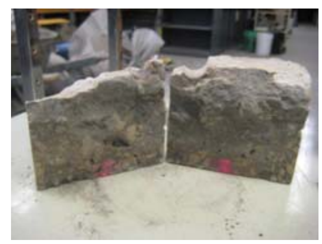
Note: All photos were taken placing Cores Upside down to facilitate balance.

ATTACHMENT D

Core Summary Sheets

USH 18/151 – Dane County

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**USH18-Dane-S13-Jt2-D3**	     
Core Intact, Y or N	No (only available top portion)	
Type of Joint Sealant	NA	
Socketing around dowel	None	
Corrosion of dowel at concrete interface	NA	
Abrasion of the dowel surface at the crack face	NA	
Dowel Deterioration	NA	
Dowel Type	NA	
PCC Deterioration	Minor at joint	
PCC Stain		
Signs of Defect in Core-holes		
Date when core was taken	6-17-2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: Dowel was not available, only top portion of core.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**USH18-Dane-S13-Jt2-D11**	
Core Intact, Y or N	Yes	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	No	
Abrasion of the dowel surface at the crack face	No	
Dowel Deterioration	One small spot, light brown	
Dowel Type	Epoxy Coated	
PCC Deterioration	Some after core was opened with electrical hammer.	
PCC Stain		
Signs of Defect in Core-holes		
Date when core was taken	6-17-2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The core was opened with an electrical hammer to be able to observe the dowel. The core was broken into 3 big pieces, 3 medium and a lot of small ones.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**USH18-Dane-S13-Jt3-D3**	           
Core Intact, Y or N	Yes	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	Yes	
Abrasion of the dowel surface at the crack face	Minor	
Dowel Deterioration	Minor	
Dowel Type	Epoxy Coated	
PCC Deterioration	No	
PCC Stain	Yes, a few colors of orange and dark brown spots	
Signs of Defect in Core-holes		
Date when core was taken	6-17-2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The core was broken with the electrical hammer to be able to observe the inside. Now there are 3 big pieces and 1 medium		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Note: All photos were taken placing Cores Upside down to facilitate balance.

Signs of Defects in Cores Taken	Description
Core Number	**USH18-Dane-S13-Jt3-D11**
Core Intact, Y or N	Yes
Type of Joint Sealant	NA
Socketing around dowel	No
Corrosion of dowel at concrete interface	Only at the ends where the core is cut
Abrasion of the dowel surface at the crack face	No
Dowel Deterioration	No
Dowel Type	Epoxy Coated
PCC Deterioration	No
PCC Stain	A few spots starting to turn yellowish
Signs of Defect in Core-holes	
Date when core was taken	6-17-2009
Dowel/Concrete Slab Interface	
Base Material Noted	

General Comments:
The core was opened with an electrical hammer. There are 3 big pieces and dowel is still attached to bigger piece. There are also 3 small pieces that belong to the area of joint/dowel.

Photos









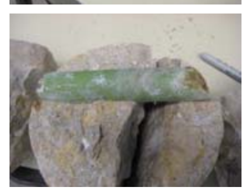








Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**USH18-Dane-S13-Jt4-D3**	            
Core Intact, Y or N	No (2 pieces)	
Type of Joint Sealant	None	
Socketing around dowel	None	
Corrosion of dowel at concrete interface	A few small spots visible	
Abrasion of the dowel surface at the crack face	No	
Dowel Deterioration	Minor, rusted at edges of cut	
Dowel Type	Epoxy Coated	
PCC Deterioration	No	
PCC Stain	Minor, only at joints due to pollution	
Signs of Defect in Core-holes		
Date when core was taken	6-17-2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: This core was also opened with electrical hammer to observe the inside. Total pieces 4.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**USH18-Dane-S13-Jt4-D11**	         
Core Intact, Y or N	Yes	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	No	
Abrasion of the dowel surface at the crack face	No	
Dowel Deterioration	No	
Dowel Type	Epoxy Coated	
PCC Deterioration	No	
PCC Stain	No	
Signs of Defect in Core-holes		
Date when core was taken	6-17-2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The core was broken with electrical hammer to be able to observe the inside of the core. The dowel was in perfect shape. The PCC gor broken in a laminated shapes.		

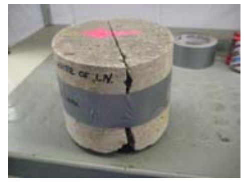
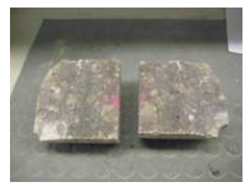
Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**USH18-Dane-S14-Jt1-D3**	     
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	Dowel missing but apparently none	
Corrosion of dowel at concrete interface	NA	
Abrasion of the dowel surface at the crack face	NA	
Dowel Deterioration	NA	
Dowel Type	NA	
PCC Deterioration	Bottom portion is missing	
PCC Stain	Dark at joint due to pollution	
Signs of Defect in Core-holes		
Date when core was taken		
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: Only 2 big pieces of PCC and no dowel to analyze.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**USH18-Dane-S14-Jt1-D11**	         
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	No Dowel available	
Abrasion of the dowel surface at the crack face	No Dowel available	
Dowel Deterioration	NA	
Dowel Type	NA	
PCC Deterioration	None	
PCC Stain	None	
Signs of Defect in Core-holes		
Date when core was taken	6-17-2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: Only the top half of the core was available. No dowel included.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

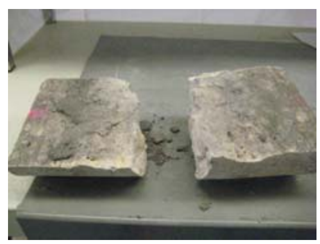
Signs of Defects in Cores Taken	Description
Core Number	**USH18-Dane-S14-Jt2-D11**
Core Intact, Y or N	No (2 pieces)
Type of Joint Sealant	NA
Socketing around dowel	No
Corrosion of dowel at concrete interface	No Dowel to evaluate
Abrasion of the dowel surface at the crack face	NA
Dowel Deterioration	NA
Dowel Type	NA
PCC Deterioration	No
PCC Stain	Yes, some spots and dark material at joint.
Signs of Defect in Core-holes	
Date when core was taken	6-17-2009
Dowel/Concrete Slab Interface	
Base Material Noted	

General Comments:
No dowel was available to evaluate.

Photos








Note: All photos were taken placing Cores Upside down to facilitate balance.

ATTACHMENT E

Core Summary Sheets

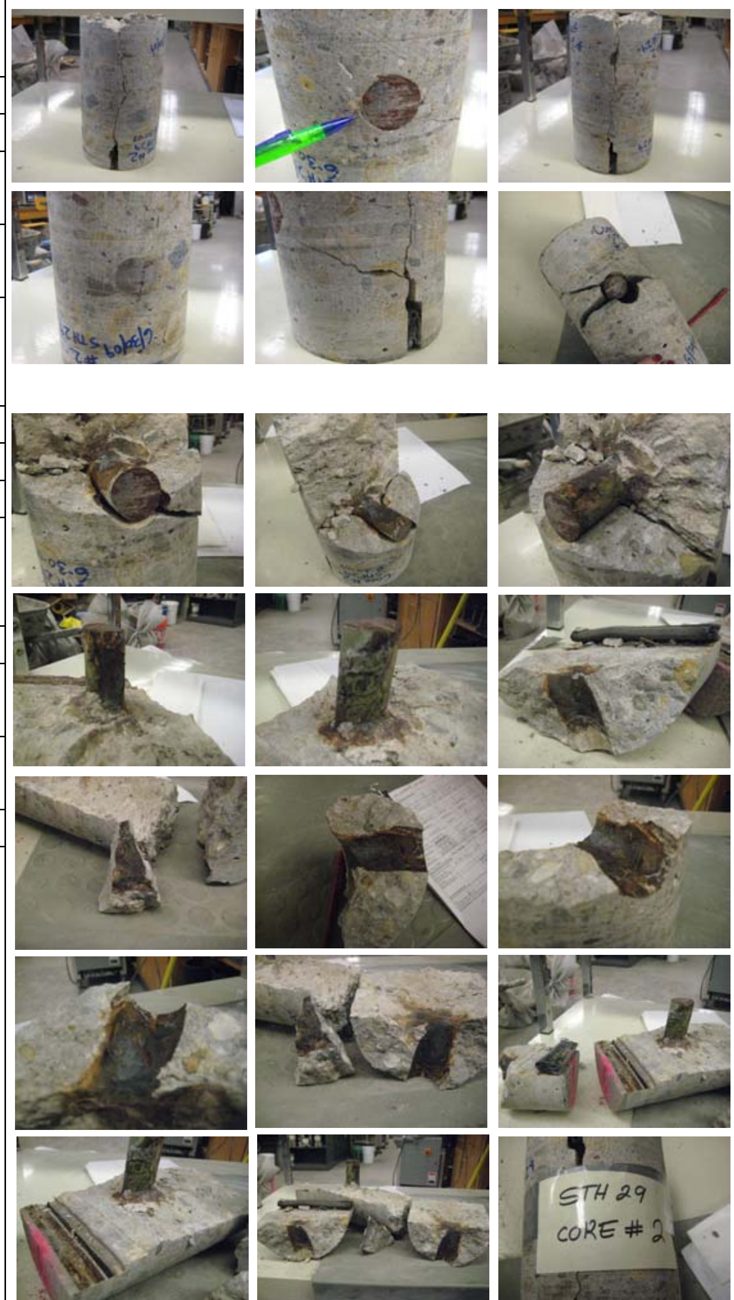
STH 29 – Brown County

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Brown-S3-Jt4-D3**	
Core Intact, Y or N	No	
Type of Joint Sealant	Neoprene	
Socketing around dowel	None	
Corrosion of dowel at concrete interface	Moderate	
Abrasion of the dowel surface at the crack face	Moderate	
Dowel Deterioration	Significantly at joint	
Dowel Type	Epoxy Coated	
PCC Deterioration	Moderate at joints and minor at bottom	
PCC Stain	Yes, colors ranging from dark browns and oranges, dark colors mainly at joint.	
Signs of Defect in Core-holes		
Date when core was taken	6-30-09	
Dowel/Concrete Slab Interface	NA	
Base Material Noted	NA	
General Comments: PCC in 4 big pieces.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Brown-S3-Jt5-D3**	
Core Intact, Y or N	Yes	
Type of Joint Sealant	Neoprene	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	Moderate	
Abrasion of the dowel surface at the crack face	Moderate	
Dowel Deterioration	Moderate	
Dowel Type	Epoxy Coated	
PCC Deterioration	No	
PCC Stain	Yes, spots of colors yellow, dark browns and dark oranges	
Signs of Defect in Core-holes		
Date when core was taken	6-30-2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: There are now a total of 4 pieces after core was opened with an electrical hammer. There are 3 big pieces and 1 medium.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Brown-S3-Jt6-D3**	
Core Intact, Y or N	No (3 pieces)	
Type of Joint Sealant	NA	
Socketing around dowel	Minor	
Corrosion of dowel at concrete interface	Yes, all around	
Abrasion of the dowel surface at the crack face	Yes, all length	
Dowel Deterioration	Moderate	
Dowel Type	Epoxy Coated	
PCC Deterioration	No	
PCC Stain	Yes, dark brown areas	
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The core was hammered to observe the rest of the dowel. There are 4 main pieces, 2 medium pieces and some small. The dowel is still attached to ¼ of PCC.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Brown-S4-Jt1-D3**	
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	Minor	
Corrosion of dowel at concrete interface	Moderate	
Abrasion of the dowel surface at the crack face	Moderate	
Dowel Deterioration	Yes	
Dowel Type	Epoxy Coated	
PCC Deterioration	Minor	
PCC Stain	Yes, dark browns and oranges by corroded areas.	
Signs of Defect in Core-holes		
Date when core was taken	6-30-2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: There are 4 big pieces and one medium. The dowel is loose.		

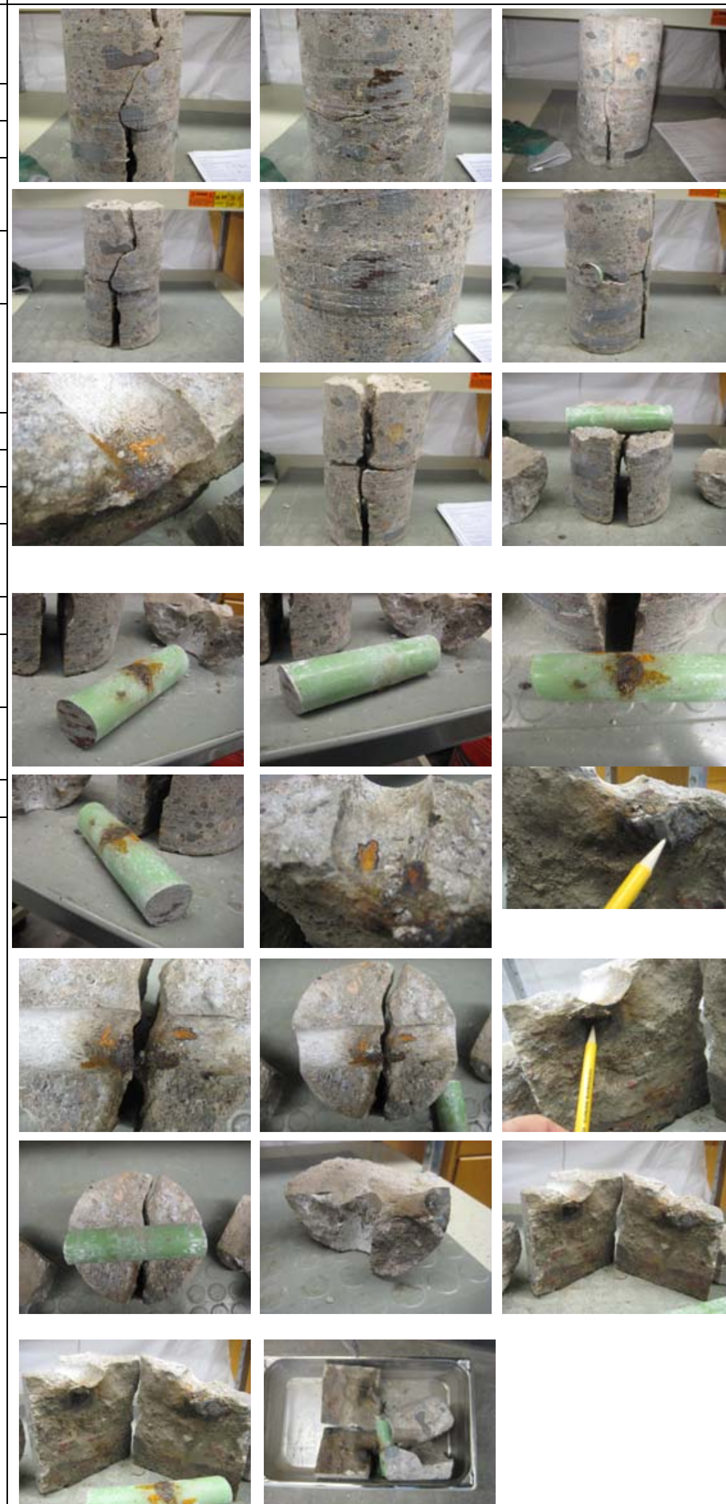
Note: All photos were taken placing Cores Upside down to facilitate balance

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description
Core Number	**STH29-Brown-S4-Jt2-D3**
Core Intact, Y or N	Yes
Type of Joint Sealant	NA
Socketing around dowel	No
Corrosion of dowel at concrete interface	Yes
Abrasion of the dowel surface at the crack face	Minor
Dowel Deterioration	Minor
Dowel Type	Epoxy Coated
PCC Deterioration	No
PCC Stain	Yes (mainly at joint, dark brown spots)
Signs of Defect in Core-holes	
Date when core was taken	6-30-2009
Dowel/Concrete Slab Interface	
Base Material Noted	

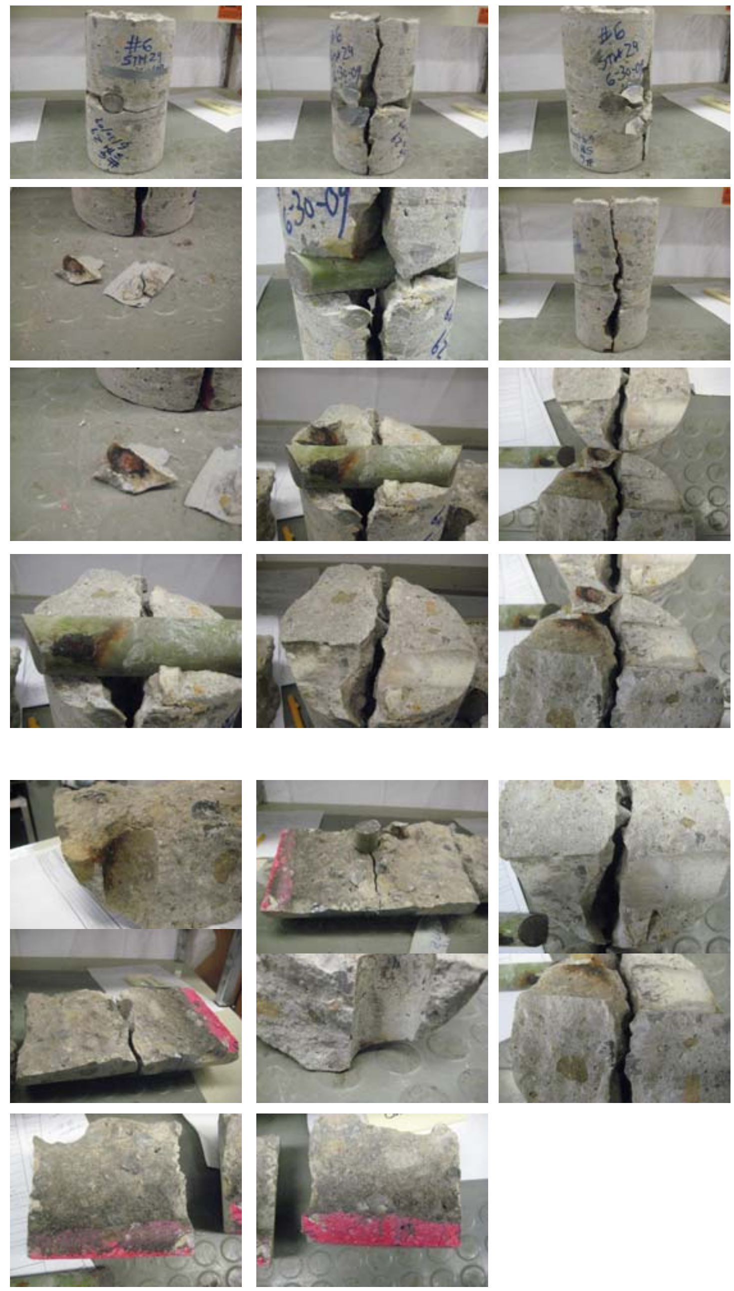
General Comments:
The core was broken with an electrical hammer to be able to see the dowel. The core consist of 4 big pieces and the dowel is loose.

Photos



Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Brown-S4-Jt3-D3**	
Core Intact, Y or N	No (4 pieces)	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	Minor	
Abrasion of the dowel surface at the crack face	Moderate	
Dowel Deterioration	Minor	
Dowel Type	Epoxy Coated	
PCC Deterioration	Minor	
PCC Stain	One spot very dark brown and orange	
Signs of Defect in Core-holes		
Date when core was taken	6-30-2009	
Dowel/Concrete Slab Interface	No	
Base Material Noted		

General Comments:
There are 2 small pieces of PCC with stain and attached to the duck tape

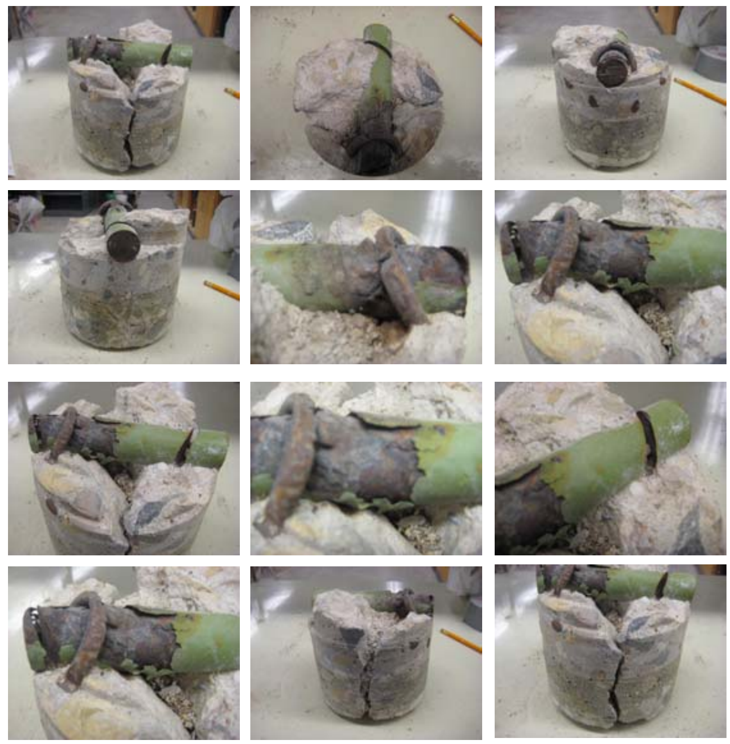
Note: All photos were taken placing Cores Upside down to facilitate balance.

ATTACHMENT F

Core Summary Sheets

STH 16 – Waukesha County

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH16-Waukesha-C1**	
Core Intact, Y or N	Yes, only available top portion of core	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	Extensive	
Abrasion of the dowel surface at the crack face	Extensive	
Dowel Deterioration	Extensive	
Dowel Type	Epoxy Coated	
PCC Deterioration	Minor	
PCC Stain	Unable to observe	
Signs of Defect in Core-holes		
Date when core was taken	4-20-09	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: PCC need to be hammered drilled to separate sides to observe joint face.		

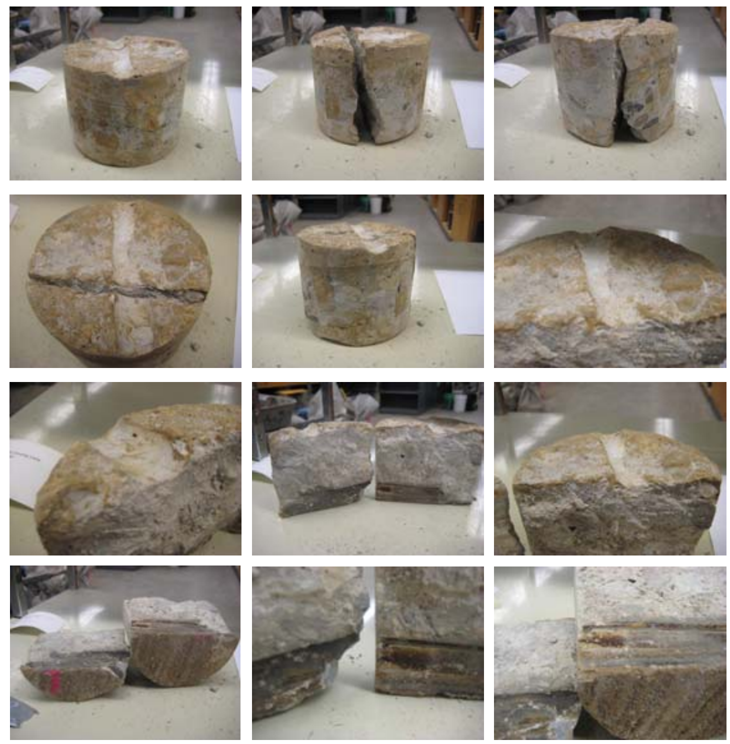
Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH16-Waukesha-C2**	
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	NA	
Corrosion of dowel at concrete interface	Moderate	
Abrasion of the dowel surface at the crack face	Moderate	
Dowel Deterioration	Moderate	
Dowel Type	Epoxy Coated	
PCC Deterioration	Minor	
PCC Stain	Unable to see	
Signs of Defect in Core-holes		
Date when core was taken	4-20-09	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: This core came in 3 pieces, 2PCC and the dowel. Unable to see any stain in PCC. The dowel was loose and moderately corroded.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH16-Waukesha-C3**	
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	NA	
Corrosion of dowel at concrete interface	NA	
Abrasion of the dowel surface at the crack face	NA	
Dowel Deterioration	NA	
Dowel Type	Dowel Missing	
PCC Deterioration	Minor	
PCC Stain	Yes, a few spots light brown where dowelsits.	
Signs of Defect in Core-holes		
Date when core was taken	4-20-09	
Dowel/Concrete Slab Interface		
Base Material Noted		

General Comments:
The core came with only 2 pieces of PCC (top portion) and dowel was missing.
Regarding the joint seal, it was missing but looking at the core, it could have been neoprene. See last 2 photos of this page.

Note: All photos were taken placing Cores Upside down to facilitate balance

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH67-Waukesha-C4**	
Core Intact, Y or N	Yes	
Type of Joint Sealant	NA	
Socketing around dowel	Yes	
Corrosion of dowel at concrete interface	Minor	
Abrasion of the dowel surface at the crack face	Minor, some orange color spots at ends	
Dowel Deterioration	Minor	
Dowel Type	Epoxy Coated	
PCC Deterioration	No	
PCC Stain	Yes, dark and brown colors at dowel placement areas	
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The AC Overlay layer is attached to PCC as well as the dowel.		

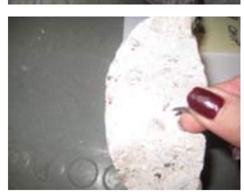
Note: All photos were taken placing Cores Upside down to facilitate balance.

ATTACHMENT G

Core Summary Sheets

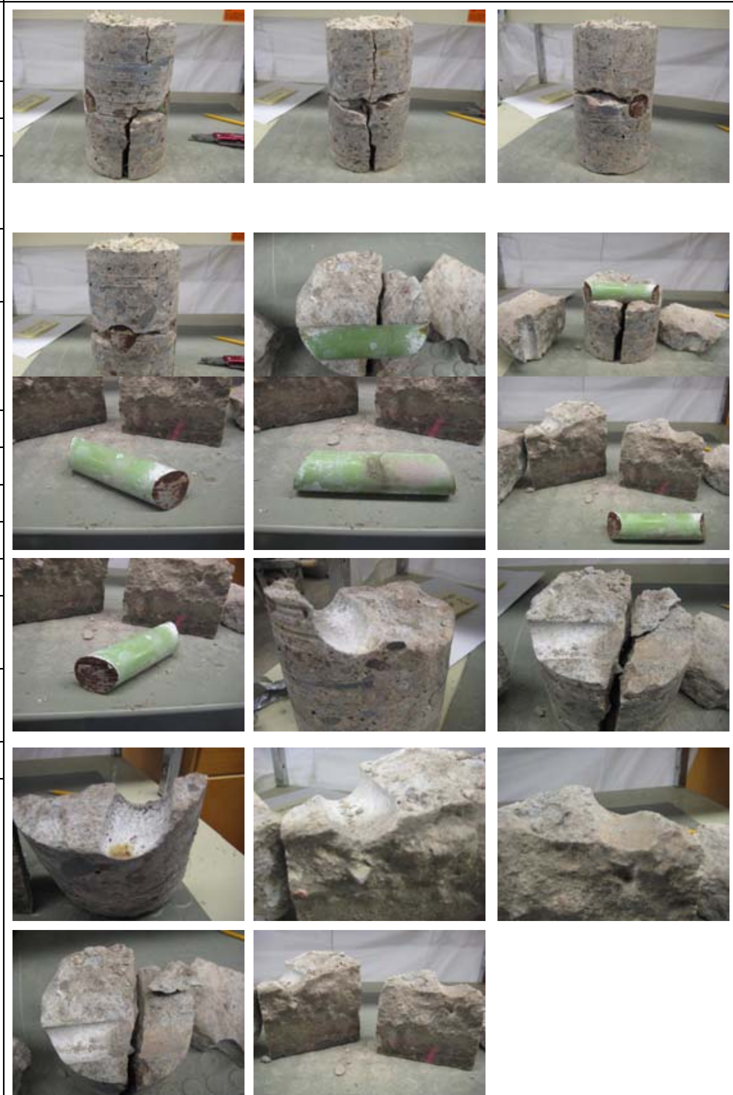
STH 29 – Chippewa County

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Chippewa-C1-D3**	               
Core Intact, Y or N	No (4 Big pieces)	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	Small orange spot	
Abrasion of the dowel surface at the crack face	Only at cut edges	
Dowel Deterioration	No	
Dowel Type	Epoxy Coated	
PCC Deterioration	Yes, minor	
PCC Stain	No stains	
Signs of Defect in Core-holes		
Date when core was taken	NA	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The dowel is still attached to one of the PCC pieces.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Chippewa-C4-D3**	
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	Minimal, there is brown spots at edges	
Abrasion of the dowel surface at the crack face	Minor	
Dowel Deterioration	Minor	
Dowel Type	Epoxy Coated	
PCC Deterioration	No	
PCC Stain	One spot light yellow	
Signs of Defect in Core-holes		
Date when core was taken	10-20-2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: There are 4 PCC big pieces and the dowel is loose.		

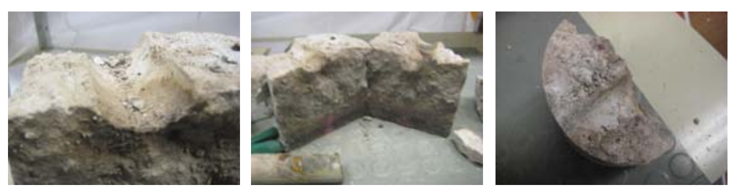
Note: All photos were taken placing Cores Upside down to facilitate balance

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description
Core Number	**STH29-Chippewa-C6-D3**
Core Intact, Y or N	No
Type of Joint Sealant	NA
Socketing around dowel	Minor
Corrosion of dowel at concrete interface	Yes, minor
Abrasion of the dowel surface at the crack face	Moderate
Dowel Deterioration	Minor
Dowel Type	Epoxy Coated
PCC Deterioration	Yes, minor
PCC Stain	Orange and dark brown spots in one area only
Signs of Defect in Core-holes	
Date when core was taken	10-20-2009
Dowel/Concrete Slab Interface	
Base Material Noted	

General Comments:
There are 4 big pieces, 1 medium and dowel is loose.

Photos



Note: All photos were taken placing Cores Upside down to facilitate balance.

ATTACHMENT H

Core Summary Sheets

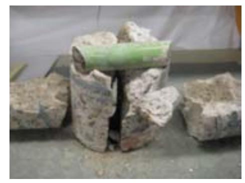
STH 29 – Clark County

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Clark-C1-Jt6-D3**	
Core Intact, Y or N	Yes	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	Minor	
Abrasion of the dowel surface at the crack face	Minor	
Dowel Deterioration	No	
Dowel Type	Epoxy Coated Steel	
PCC Deterioration	No	
PCC Stain	One spot very dark	
Signs of Defect in Core-holes		
Date when core was taken	10/21/2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The core was hammered to be able to observe the dowel and PCCC. There are now 3 big pieces.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Clark-C1-Jt6-D11**	  
Core Intact, Y or N	No	  
Type of Joint Sealant	NA	  
Socketing around dowel	No	  
Corrosion of dowel at concrete interface	No	  
Abrasion of the dowel surface at the crack face	No	
Dowel Deterioration	Yes, some oxidation at cut areas.	
Dowel Type	Epoxy Coated Steel	
PCC Deterioration	Minor	
PCC Stain	None	
Signs of Defect in Core-holes		
Date when core was taken	10/21/2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: There are 4 big pieces, 1 medium and the dowel is loose.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Clark-C1-Jt7-D3**	
Core Intact, Y or N	No (4 big pieces)	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	Only at ends where cut was cut and taped.	
Abrasion of the dowel surface at the crack face	No	
Dowel Deterioration	No	
Dowel Type	Epoxy Coated Steel	
PCC Deterioration	Minor	
PCC Stain	Yes, light browns and yelllows	
Signs of Defect in Core-holes		
Date when core was taken	10/21/2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The core is broken in 4 big pieces and the dowel is loose.		

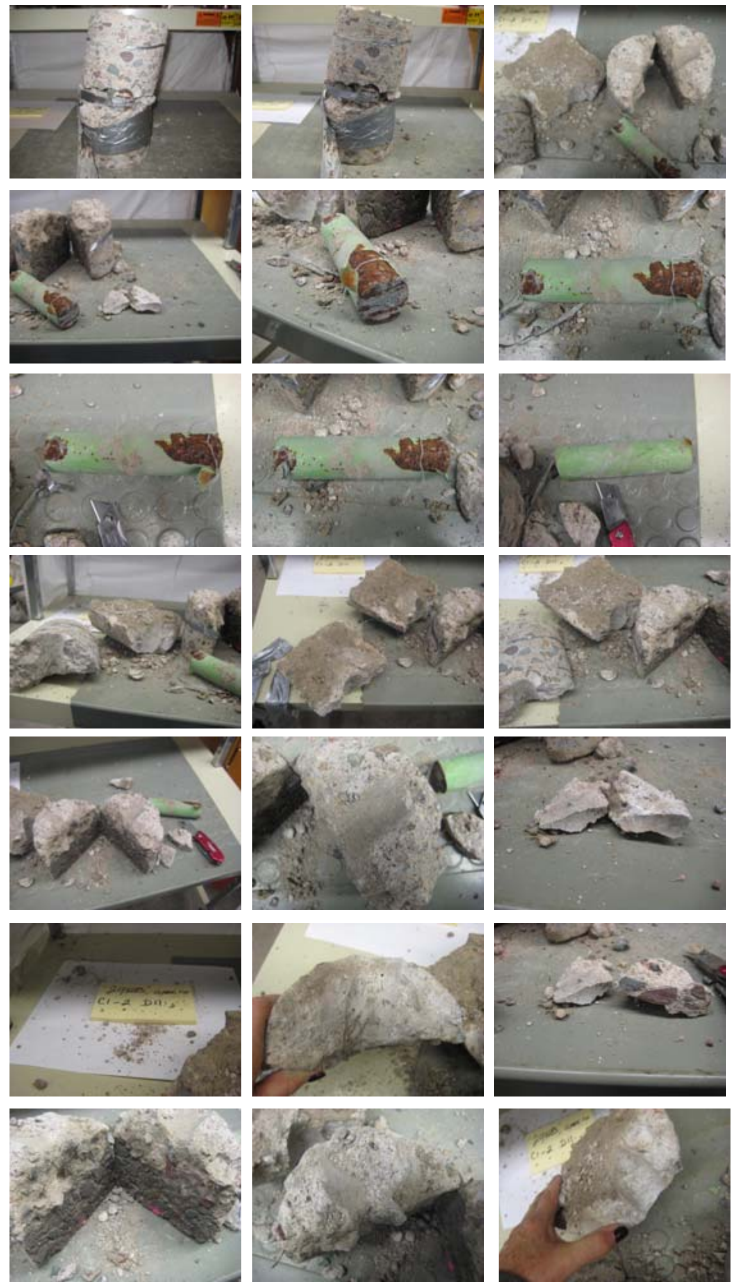
Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description
Core Number	**STH29-Clark-CI-Jt7-D11**
Core Intact, Y or N	No
Type of Joint Sealant	NA
Socketing around dowel	Minor
Corrosion of dowel at concrete interface	Minor
Abrasion of the dowel surface at the crack face	Extensive
Dowel Deterioration	Yes
Dowel Type	Epoxy Coated Steel
PCC Deterioration	Yes, core crumbled
PCC Stain	No
Signs of Defect in Core-holes	
Date when core was taken	10/21/2009
Dowel/Concrete Slab Interface	
Base Material Noted	

General Comments:
The core is broken into 4 big pieces, 3 medium, dowel is loose and a lot of small pieces.

Photos



Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Clark-CP-Jt3-D3**	
Core Intact, Y or N	No (a lot of small pieces)	
Type of Joint Sealant	NA	
Socketing around dowel	Yes, at joints, moderate	
Corrosion of dowel at concrete interface	No	
Abrasion of the dowel surface at the crack face	No	
Dowel Deterioration	No	
Dowel Type	Creative Pultrusions FRP (Fiber Reinforced Polymer)	
PCC Deterioration	Moderate	
PCC Stain	None	
Signs of Defect in Core-holes		
Date when core was taken	10-21-2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: There are 3 big pieces, 1 medium, and dowel attached to bigger piece. There are also a lot of small pieces from crumbled PCC) Dark also at joint due to pollution.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Clark-CP-Jt3-D11**	
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	No	
Abrasion of the dowel surface at the crack face	No	
Dowel Deterioration	No	
Dowel Type	Creative Pultrusions FRP (Fiber Reinforced Polymer)	
PCC Deterioration	Minor	
PCC Stain	No	
Signs of Defect in Core-holes		
Date when core was taken	10/21/2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The dowel is attached to one of the PCC pieces. A total of 8 pieces.		

Note: All photos were taken placing Cores Upside down to facilitate balance

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Clark-GF-Jt7-D3**	
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	Dark areas at joints	
Abrasion of the dowel surface at the crack face	No	
Dowel Deterioration	No	
Dowel Type	Glasforms FRP (Fiber Reinforced Polymer)	
PCC Deterioration	Moderate	
PCC Stain	Some dark areas at joint	
Signs of Defect in Core-holes		
Date when core was taken	10/21/2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: There are 4 big pieces, dowel attached to ¼ of the core. There is one medium piece and small pieces from PCC crumbled.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Clark-GF-Jt7-D11**	            
Core Intact, Y or N	No (6 pieces)	
Type of Joint Sealant	NA	
Socketing around dowel	None	
Corrosion of dowel at concrete interface	None	
Abrasion of the dowel surface at the crack face	None	
Dowel Deterioration	None	
Dowel Type	Glasforms FRP (Filled Reinforced Polymer)	
PCC Deterioration	Very manor due to the breaking of the core when extracted.	
PCC Stain	There are dark gray areas which believed to be from pollution due to lacking of joint seal.	
Signs of Defect in Core-holes		
Date when core was taken	10/21/2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: A total of 6 big PCC pieces. Dark color of PCC at joint (see last photo in this page)		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	STH29-Clark-RJ-Jt1-D3	
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	Yes, some dark spots at joint	
Abrasion of the dowel surface at the crack face	No	
Dowel Deterioration	No	
Dowel Type	RJD FRP (Fiber Reinforced Polymer)	
PCC Deterioration	Yes, some crumbled	
PCC Stain	Yes, some dark spots at dowel placement	
Signs of Defect in Core-holes		
Date when core was taken	10/21/2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: There are 4 big pieces, one medium and a lot of small pieces. The dowel is still attached to ¼ of the PCC core.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Clark-RJ-Jt1-D11**	
Core Intact, Y or N	No (5 pieces)	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	Minor in one side	
Abrasion of the dowel surface at the crack face	No	
Dowel Deterioration	Minor	
Dowel Type	RJD FRP (Fiber Reinforced Polymer)	
PCC Deterioration	Minor	
PCC Stain	Some dark brown areas	
Signs of Defect in Core-holes		
Date when core was taken	10/21/2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: The core is broken into 5 pieces and the dowel is loose.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Clark-HF-Jt8-D3**	
Core Intact, Y or N	No (6 pieces)	
Type of Joint Sealant	NA	
Socketing around dowel	No	
Corrosion of dowel at concrete interface	No	
Abrasion of the dowel surface at the crack face	No	
Dowel Deterioration	No	
Dowel Type	Mortar-Filled Stainless Steel Tube	
PCC Deterioration	Minor	
PCC Stain	Only one dark area	
Signs of Defect in Core-holes		
Date when core was taken	10/21/2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: There are 3 big pieces of PCC, 3 medium pieces and dowel is loose.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29- Clark-HF-JT8-D11**	
Core Intact, Y or N	No	
Type of Joint Sealant	None	
Socketing around dowel	None	
Corrosion of dowel at concrete interface	None	
Abrasion of the dowel surface at the crack face	None	
Dowel Deterioration	None	
Dowel Type	Mortar-Filled Stainless Steel Tube	
PCC Deterioration	None	
PCC Stain	None	
Signs of Defect in Core-holes		
Date when core was taken	10/21/2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: PCC broken in 4 pieces, dowel still attached to main PCC piece.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Clark-SS-Jt5-D3**	
Core Intact, Y or N	No	
Type of Joint Sealant	NA	
Socketing around dowel	Yes, PCC crumbling at joint	
Corrosion of dowel at concrete interface	No	
Abrasion of the dowel surface at the crack face	No	
Dowel Deterioration	No	
Dowel Type	Solid Stainless Steel	
PCC Deterioration	Yes	
PCC Stain	Some darker areas at joint. Could be due to dirt and pollution	
Signs of Defect in Core-holes		
Date when core was taken	10/21/2009	
Dowel/Concrete Slab Interface		
Base Material Noted	No	
General Comments: There are 4 big pieces of PCC and one medium. Dowel is not attached to any of the pieces.		

Note: All photos were taken placing Cores Upside down to facilitate balance.

Evaluation of Alternative Dowel Bar Materials – Documentation of Cores

Signs of Defects in Cores Taken	Description	Photos
Core Number	**STH29-Clark-SS-Jt5-D11**	       
Core Intact, Y or N	No (this core was staked inside of the core bit in big pieces)	
Type of Joint Sealant	NA	
Socketing around dowel	Unable to see the core all together	
Corrosion of dowel at concrete interface	NA	
Abrasion of the dowel surface at the crack face	None	
Dowel Deterioration	None	
Dowel Type	Solid Stainless Steel	
PCC Deterioration	Unable to see. It had to be broken to extract it from core bit.	
PCC Stain	Minor shaded areas or dark stain	
Signs of Defect in Core-holes		
Date when core was taken	10/21/2009	
Dowel/Concrete Slab Interface		
Base Material Noted		
General Comments: There are a lot of pieces due to the breaking to extract the core from core bit.		

Note: All photos were taken placing Cores Upside down to facilitate balance.