High Performance Concrete Pavements

Project Summary

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Ahmad Ardani, Colorado DOT
Mark Gawedzinski, Illinois DOT
David Lippert, Illinois DOT
Tommy Nantung, Indiana DOT
Jim Cable, Iowa State University
Mark Dunn, Iowa DOT
John Wojakowski, Kansas DOT
Dimitrios Goulias, University of Maryland
David Smiley, Michigan DOT
Tom Hines, Michigan DOT
Dave Rettner, Minnesota DOT
Curt Turgeon, Minnesota DOT
Tom Burnham, Minnesota DOT
Randy Battey, Mississippi DOT
Tim Chojnacki, Missouri DOT
Charles Goodspeed, University of New Hampshire
Shad Sargand, Ohio University
Anastasios Ioannides, University of Cincinnati
Roger Green, Ohio DOT
V. Ramakrishnan, South Dakota School of Mines and Technology
Daniel Strand, South Dakota DOT
Dave Huft, South Dakota DOT
Dan Johnston, South Dakota DOT
Celik Ozyildirim, Virginia Transportation Research Council
David Kuemmel, Marquette University
Jim Crovetti, Marquette University
Debbie Bischoff, Wisconsin DOT
Peter Kemp, Wisconsin DOT

Chapter 1. INTRODUCTION

Background

Under Test and Evaluation Project 30 (TE-30), High Performance Concrete Pavement (HPCP), the Federal Highway Administration (FHWA) is exploring the applicability of innovative portland cement concrete (PCC) pavement design and construction concepts in the United States. These innovative concepts, ranging from the use of trapezoidal cross sections to alternative dowel bar materials to fiber-reinforced concrete, all share the same TE-30 goal of providing long-lasting, economical PCC pavements that meet the specific performance requirements of their particular application.

The TE-30 program actually got its start in May 1992 when a team of State, industry, and Federal engineers participated in the U.S. Tour of European Concrete Highways (US TECH) (FHWA 1992). During that visit, the tour participants were exposed to a wealth of information on concrete pavement materials, structural design, and construction that could benefit concrete pavements in the United States. Followup visits to Germany and Austria in October 1992 (Larson, Vanikar, and Forster 1993) provided additional information that was used to construct an experimental concrete pavement in the United States consisting of a German structural design (to provide long service life) and an Austrian exposed aggregate surface (to reduce tire / pavement noise). That pavement, a 1.6-km (1-mi) test section located in the northbound lanes of I-75 (Chrysler Freeway) in downtown Detroit, was constructed in 1993 (Weinfurter, Smiley, and Till 1994).

The success of the I-75 project in incorporating European design concepts that hold the promise of long-lasting, low maintenance concrete pavements spawned a great interest in pursuing similar projects. In 1994, both the FHWA and industry agreed to pursue this effort, effectively launching the TE-30 program. Broad functional or performance criteria were established so that participating State highway agencies (SHAs) could select the area considered appropriate for improving the performance of concrete pavements in their States.

Several innovation areas for the program were suggested:

- Increasing the service life.
- Decreasing construction time.
- Lowering life-cycle costs.
- Lowering maintenance costs.
- Constructing ultra-smooth ride quality pavements.
- Incorporating recycled or waste products while maintaining quality.
- Utilizing innovative construction equipment or procedures.
- Utilizing innovative quality initiatives.

Specific target projects were later added, including joint sealing alternatives, alternative load transfer devices, durable concrete mix designs, alternative surface finishing techniques, and more cost-effective use of paving materials (such as widened lanes, trapezoidal cross sections, and two-lift construction).

In each of these applications, emphasis is given to an integrated design approach in which site influences (traffic loading, climate, and subgrade), concrete mix design, structural design, joint details, and construction are considered together to develop the appropriate pavement design. Consequently, the term “high performance” does not necessarily refer to high strength concrete, but rather to any of the materials and mix design, structural design, or construction components of the pavement that are expected to contribute to the pavement’s long-term performance.

Included Projects

The TE-30 Program has funded approximately 25 field projects since 1996. As previously noted, these projects were intended to test and evaluate innovative concrete pavement technology in “on-the-road” applications. A report (FHWA 2002) summarized the status of the projects initiated through December 2001; several topic-specific reports were also prepared based on the results of the original FHWA report (Hoerner and Smith 2002; Smith 2002).

Since the preparation of the original report, several additional field projects have been constructed and additional monitoring reports completed. Moreover, projects originally included in a similar FHWA test and evaluation project, Field Trials of Concrete Pavement Product and Process Technology, have been or are in the process of being constructed. The projects included in this report are those from the previous FHWA report (FHWA 2002) with an additional 14 projects included from the Field Trials of Concrete Pavement Product and Process Technology initiative.

The geographical distribution of the included projects is shown in Figure 1, and the projects are listed in Table 1. For each project, the table includes information on the design features evaluated, the year built, the type of concrete pavement (jointed plain concrete pavement [JPCP], jointed reinforced concrete pavement [JRCP], continuously reinforced concrete pavement [CRCP], precast post-tensioned concrete pavement (PPCP), or fiber-reinforced concrete pavement [FRCP]), and whether the project was funded as part of the TE-30 program. More detailed information on each project is provided in the relevant chapter and in Appendix A to this report.

Purpose and Overview of Report

The previous FHWA report (FHWA 2002) provides the foundation for this report. The 24 projects described in that original report have been updated, where new information was available, and an additional 15 projects have been included in this report. Most of these newer projects have been constructed and, since the TE-30 program is ongoing, it is necessary to ascertain the status of those projects that have been constructed.

It is the purpose of this report to document the current status of the TE-30 projects and the *Field Trials of Concrete Pavement Product and Process Technology* sections as well. The authors have also attempted to describe anticipated results from the TE-30 program and to recommend relevant future research activities. Chapters 2 through 39 of this report summarize each individual project included to date, describing the goals of the project, the design features being evaluated, and any preliminary results or products.

Two appendixes are included in support of this report. Appendix A provides a summary table containing general design and construction information on each of the projects. Appendix B lists references relevant to each of the projects, organized by State.

Figure 1. Location of TE-30 and related projects.

Table 1. Listing of TE-30 and Related Projects

PROJECT	TE-30?	PAVEMENT TYPE	DESIGN FEATURES EVALUATED	YEAR BUILT
California 1 I-10, El Monte	No	PPCP	Precast, post-tensioned concrete pavement	2004
Colorado 1 SH 121, Wadsworth	No	UTW	Ultrathin whitetopping	2001
Colorado 2 I-25, Loveland	No	JPCP	Precast concrete slabs for full-depth repairs	2004
Illinois 1 I-55 SB, Williamsville	No	JRCP	Alternative dowel bar materials	1996
Illinois 2 IL 59, Naperville	Yes	JRCP JPCP	Alternative dowel bar materials Sealed/unsealed joints Traffic counters	1997
Illinois 3 US 67 WB, Jacksonville	Yes	JPCP	Alternative dowel bar materials Sealed/unsealed joints	1999
Illinois 4 SR 2 NB, Dixon	No	JPCP	Alternative dowel bar materials	2000
Indiana 1 I-65 at SR 60, Clark County	Yes	JPCP	Factors to reduce curling/warping	2004
Iowa 1a IA 5, Carlisle	Yes	JPCP	PCC mixing times on PCC properties	1996
Iowa 1b US 30, Carroll	Yes	JPCP	PCC mixing times on PCC properties	1996
Iowa 2 US 65 Bypass, Des Moines	Yes	JPCP	Alternative dowel bar materials Alternative dowel bar spacings	1997
Iowa 3 US 151, Linn/Jones	Yes	JPCP	Fly-ash stabilization of PCC	2001
Iowa 4 IA 330, Jasper, Story, and Marshall Counties	No	JPCP	Elliptical steel dowel bars	2002
Iowa 5 Iowa 330, Melbourne	No	JPCP	Elliptical fiber-reinforced polymer dowel bars	2002
Iowa 6 Various locations	No	Various	Fly-ash stabilization of subgrade for PCC pavements	N/A
Iowa 7	No	Various	Total Environmental Management for Paving (TEMP)	N/A
Kansas 1 K-96, Haven	Yes	JPCP FRCP	Alternative dowel bar materials Alternative PCC mix designs (including fiber PCC) Joint sawing alternatives Joint sealing alternatives Surface texturing Two-lift construction	1997
Kansas 2 Hutchinson	Yes	JPCP	Smoothness monitoring of plastic concrete	2001
Maryland 1 US 50, Salisbury Bypass	Yes	JPCP FRCP	PCC mix design Fiber PCC	2001
Michigan 1 I-75 NB, Detroit	No	JRCP JPCP	Two-lift construction Exposed aggregate Thick foundation Alternative dowel bar materials and spacing	1993

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PROJECT	TE-30?	PAVEMENT TYPE	DESIGN FEATURES EVALUATED	YEAR BUILT
Michigan 2 M25, Port Austin I-675 Zilwaukee	No	JRCP	Precast concrete slabs for full depth repairs	2003
Minnesota 1 I-35W, Richfield	Yes	JPCP	Alternative dowel bars PCC mix design	2000
Minnesota 2 Mn/ROAD Low Volume Road Facility, Albertville	Yes	JPCP	Alternative dowel bar materials Doweled/nondoweled joints PCC mix design	2000
Minnesota 3 Mn/ROAD, Mainline Road Facility and US 169, Albertville	No	UTW	Application of ultrathin whitetopping	1997
Mississippi 1 US 72, Corinth	Yes	Resin-Modified	Alternative PCC paving material (resin-modified pavement)	2001
Missouri 1 I-29 SB, Rock Port	Yes	JPCP FRCP	Fiber PCC Slab thickness Joint spacing	1998
New Hampshire 1	Yes	N/A	HPCP definitions “Design Optimization” computer program	N/A
Ohio 1 US 50, Athens	Yes	JRCP	PCC mix design (GGBFS) Evaluation of HIPERPAV	1997-1998
Ohio 2 US 50, Athens	Yes	JRCP	Alternative dowel bar materials	1997
Ohio 3 US 50, Athens	Yes	JRCP	Sealed/unsealed joints	1997-1998
Ohio 4 US 35, Jamestown	Yes	JPCP	Evaluation of soil stiffness using nondestructive testing devices	2001
Pennsylvania 1 SR 22, Murrysville	Yes	JPCP	Evaluation of HIPERPAV	2004
South Dakota 1 US 83, Pierre	Yes	JPCP FRCP	PCC mix design Joint spacing Doweled/nondoweled joints	1996
Tennessee 1 I-65, Nashville	No	JPCP	Implementation of performance-related specifications	2004
Virginia 1 I-64, Newport News	Yes	JPCP	PCC mix design	1998-1999
Virginia 2 VA 288, Richmond	Yes	CRCP	PCC mix design Steel contents	2004
Virginia 3 US 29, Madison Heights	Yes	CRCP	PCC mix design Steel contents	2004
Washington 1 SR 395, Kennewick	No	JPCP	PCC mix design for rapid construction	2000
West Virginia 1 Corridor H, Route 219, Elkins University Avenue, Routes 857 and 119, Morgantown Route 9 between Martinsburg and Charlestown	No	JPCP JPCP CRCP	Alternative dowel bar materials, size, spacing Alternative dowel bar materials and FRP moisture diffusion FRP versus steel longitudinal reinforcing bars	2002 2002 2006?
Wisconsin 1 WI 29, Abbotsford	Yes	JPCP	Surface texturing	1997

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PROJECT	TE-30?	PAVEMENT TYPE	DESIGN FEATURES EVALUATED	YEAR BUILT
Wisconsin 2 WI 29, Owen	Yes	JPCP	Alternative dowel bar materials Alternative dowel bar spacings	1997
Wisconsin 3 WI 29, Hatley	Yes	JPCP	Alternative dowel bar materials Alternative dowel bar spacings Trapezoidal cross section	1997
Wisconsin 4 I-90, Tomah	Yes	JPCP	Alternative dowel bar materials PCC mix design	2002
FHWA 1	No	UTW	Ultrathin whitetopping repair techniques	1998-1999
Various States 1	No	JPCP	Evaluation of magnetic tomography for dowel bar location (MIT Scan-2)	2003 +

KEY: CRCP = continuously reinforced concrete pavement; FRCP = fiber-reinforced concrete pavement; FRP = fiber-reinforced polymer; GGBFS = ground granulated blast furnace slag; JPCP = jointed plain concrete pavement; JRCP = jointed reinforced concrete pavement; PCC = portland cement concrete; PPCP = precast, post-tensioned concrete pavement

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Chapter 2. CALIFORNIA 1 (I-10, EL MONTE)

Introduction

Caltrans has undertaken this project to evaluate the feasibility of using precast, post-tensioned concrete pavement for rapid replacement of a deteriorated roadway.

Study Objectives

The objectives of the project are to demonstrate the constructibility and cost effectiveness of precast, post-tensioned concrete pavements under high-volume, urban freeway conditions.

Project Design and Layout

The project includes the replacement of a 248-ft section of I-10 in Los Angeles County. A total of 31 precast concrete pavement segments were fabricated in accordance with the "Texas" design as described by Merritt, McCullough, and Burns (2003). Replacement panels were precast in segments 2.4 m (8 ft) long, 11.3 m (37 ft) wide, and 254 mm (10 in.) thick. The panels contain formed keyways along the edges normal to the direction of travel. Three types of precast panels are fabricated: base panels, joint panels, and central stressing panels. The nominal pavement thickness is 254 mm (10 in.) with a grade reversal at the outside shoulder with increased thickness at the crown. In the construction process, the panels are placed and then post tensioned in the direction of travel using stressing pockets in the slabs at the mid length of each section.

State Monitoring Activities

Under contract with FHWA, a research agency is monitoring the performance of the project for a minimum of 1 year. If funding allows, cracking smoothness and spalling evaluations will also be conducted at years 3 and 5. Other evaluation items will include expansion joint condition and vertical slab movements.

Preliminary Results/Findings

This experimental project was completed in April 2004. Precasting of the 31 panels began in December 2003 and was completed in early January 2004. The panels were then stored until the construction crew was ready for installation. The panels were placed in 2 nights followed by 4 days of post-tensioning and grouting. Currently a construction video is available from FHWA and a construction report will be completed within the next year.

Points of Contact

Kirsten Stahl
Pavement Engineer
Caltrans District 7
100 S Main Street
Los Angeles, CA 90012
(213) 897-0470
kirsten.stahl@dot.ca.gov

Jason Dietz
FHWA California Division Office
980 9th Street, Suite #400
Sacramento, CA 95814-2724
(916) 498-5886
jason.dietz@fhwa.dot.gov

References

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Chapter 3. COLORADO 1 (S.H. 121, Wadsworth, Colorado)

Introduction

The Colorado Department of Transportation (CDOT) constructed three experimental thin whitetopping test sections in 1996 and 1997. An additional 6.4-km (4-mi) test section was constructed in 2001 on S.H. 121 just south of Denver near Wadsworth, Colorado. The original three test sections were instrumented, and the resulting pavement response and performance data were used in the development of a design procedure for thin whitetopping overlays. The fourth test section was constructed to validate the design procedure that was developed.

Study Objectives

The objective of this project is to instrument, construct, load test, and monitor the performance of a thin whitetopping test section. The results will be used to validate the design procedure developed based on the test sections constructed in 1996 and 1997. More information on the test sections constructed in 1996 and 1997 can be found in the report by Tarr, Sheehan, and Okamoto (1998).

Project Design and Layout

A 10-year, 1.3 million 18-kip ESAL design was developed for the thin whitetopping test section using the CDOT design procedure. The resulting design consisted of milling to promote bonding between the existing hot mix-asphalt (HMA) and the overlay. A 152.4 mm (6-in.) PCC overlay was then placed on the remaining 140 mm (5.5 in.) of HMA. Both the longitudinal and transverse joints were sawed 1.8 m (6 ft) apart. This section contained four additional designs, which contained various combinations of overlay thicknesses and joint spacing, as shown in Table 2.

The concrete mix did contain fibers. Dowel bars were not included in any of the test sections, but tie bars were included along all longitudinal joints at 762-mm (30-in.) spacings.

The volume of traffic on this section of roadway is relatively high but the traffic consists primarily of cars and light trucks.

State Monitoring Activities

Monitoring activities included collecting strain measurements in conjunction with static load testing, performing distress surveys, recording temperature gradients, and performing shear bond strength testing on pavement cores. These activities were performed at 28 days and at 1 year after construction. Load testing was performed again in 2003. The load testing consisted of applying an 18-kip rear axle load. In 2003, a 30-kip tandem axle load was also used. Loads were applied at the corner, edges, and midpanel of the slab. Temperatures throughout the slab and asphalt were collected every 15 minutes while the load testing was being performed.

Table 2. Summary of CO 1 Project Test Sections

TEST SECTION	PCC OVERLAY THICKNESS (IN.)	HOT-MIX ASPHALT THICKNESS (IN.)	LONGITUDINAL JOINT SPACING (FT)	TRANSVERSE JOINT SPACING (FT)
1	3.9	5.0	4	4
2	4.5	5.0	6	6
3	6.5	5.0	6	9
4	6.5	5.0	6	6

Preliminary Results/Findings

The average bond shear strength was 100 lbf/in^2, and this appears to be sufficiently high to provide good performance for the overlay designs represented on State highway 121.

The strains induced by the static 18-kip single-axle load placed along the edge of the slab were converted to stress and are summarized in Table 3. The data show a reduction in stress as the slab thickness increases and the panel size decreases.

The stress predicted by the design equations was calculated for the loading conditions in the field. The predicted stresses were lower than the stresses determined using the measured strains. This is expected because the measured strains did not capture curling and warping restraint stresses.

Table 3. Ranges of Edge Stresses Measured Under an 18-Kip Single-Axle Load on the CO 1 Project Test Sections

TEST SECTION	PCC THICKNESS (IN.)	PANEL SIZE (FT)	STRESS RANGE (LBF/IN^2)
1	3.9	4 x 4	80–175
2	4.5	6 x 6	75–160
3	6.5	6 x 9	55–110
4	6.5	6 x 6	60–100

Current Project Status, Results, and Findings

A final report on the revisions to the thin whitetopping design procedure (Sheehan, Tarr, and Tayabji 2004) is available from the Colorado Department of Transportation.

Point of Contact

Ahmad Ardani
Colorado Department of Transportation
4201 East Arkansas Ave, Rm A-100
Denver, CO 80222
Ahmad.Ardani@dot.state.co.us
(303) 757-9978

References

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Chapter 4. COLORADO 2 (I-25, Loveland) and MICHIGAN 2 (M25, Port Austin, and I-675, Zilwaukee)

Introduction

An alternative to conventional cast-in-place concrete repairs is the use of precast concrete patches to address issues related to joint and slab deterioration. The use of precast panels has the potential to reduce construction time, increase long-term pavement performance, and reduce user delays. However, there are limited laboratory or field data on the construction and performance of full-depth precast concrete patches. In 2003, Michigan State University, in conjunction with the Michigan and Colorado Departments of Transportation (DOTs), initiated an FHWA-sponsored project to study the feasibility of precast panels as an alternative to cast-in-place concrete repairs. Sites in both Colorado (Figure 2) and Michigan were selected.

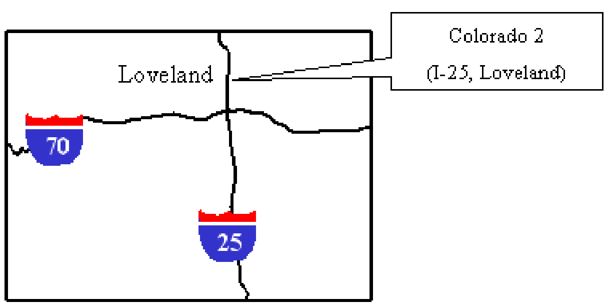

Figure 2. Location of CO 2 project.

Study Objectives

The objectives of this project are (Buch 2002):

1. Review the literature and document the known practices.
2. Conceptualize various construction alternatives as they relate to precast concrete patches.
3. Identify potential concrete pavement restoration projects along in-service concrete pavements in Colorado and Michigan and install precast concrete patches. For the purposes of comparison, control cast-in place full-depth patches will also be installed.
4. Investigate the effectiveness and efficiency of precast panels through the development of maintenance performance guidelines.
5. Recommend strategies for monitoring the "newly" installed precast patches.
6. Produce step-by-step guidelines for the construction of precast concrete panels.

Project Design and Layout

Hundreds of precast panels will be installed and monitored for performance along interstate highways in Michigan and Colorado. The construction in Michigan will include the installation of 20 precast panels, 12 along M25 (Port Austin) and 8 along I-675 (Zilwaukee). These 20 panels were slated for installation during the summer of 2003. The remaining panels will be installed on I-25 in Colorado beginning summer of 2003 and concluding in 2005. Nearly 450 panels will be installed on I-25 in Colorado.

The means of anchoring the precast panels is different between the Michigan studies and the Colorado studies. Details of each method are described in the following sections.

Michigan Concept

The concept being used in Michigan is illustrated in Figure 3 and is completed by using the following techniques:

- Mark and saw cut the perimeter (full depth) of the existing deteriorated concrete pavement area and allow time for the rest of the slab to "relax" and relieve stresses. The width of the patch should be the same as one of the standard widths for precast concrete patches.
- Cut dowel and tie bar slots in the adjoining concrete slabs.

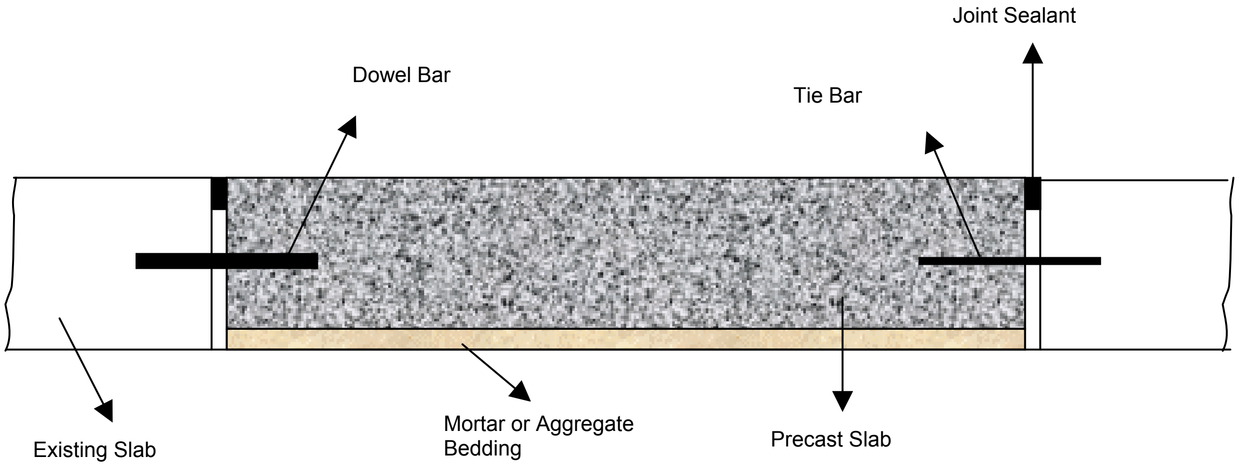

Figure 3. Precast panel design for Michigan.

- Remove the existing concrete and compact the exposed base.
- Place a bedding of aggregate or mortar (rapid setting) to adjust slab elevation if needed.
- Lower the precast concrete panel fitted with dowels and tie bars into the prepared opening. Adjust the elevation and grout the dowel bar slots. The lift holes in the precast concrete slab should also be grouted and finished.
- Seal all construction joints.
- Diamond grind when needed to restore ride quality.

Colorado Concept

The Colorado DOT is using a proprietary technology that was developed by URETEK USA, Inc. for tying the slabs. URETEK USA, Inc. has developed the Stitch-In-Time™ technology, which is a repair system for restoring load transfer to jointed, cracked, spalled or otherwise damaged concrete pavement. URETEK's Stitch-In-Time™ System uses a series of 13-mm (0.5-in.) saw-cut slots to position a 6-mm (0.25-in) thick composite insert. The slots and inserts are easily filled with sand and bonded into place with a hybrid high-density polymer. Rapid curing characteristics allow almost immediate traffic restoration. The Stitch-In-Time™ system is being used to tie the precast panels to the existing concrete pavement. (URETEK 2004). Figure 4 shows a precast panel installation in Colorado.

State Monitoring Activities

The research team will collect pre-construction information such as pavement conditions, traffic, and inventory data for the panels to be replaced (Buch 2002). The entire construction process will be documented. After panel placement, continued performance monitoring, consisting of deflection measurements (load carrying capacity and load transfer), visual distress surveys, and ride measurements will be completed for a minimum of 3 years.

Preliminary Results/Findings

Preliminary results are not available at this time, however an installation report and information video should be available by summer 2004.

Figure 4. Precast panel installation in Colorado.

Points of Contact

Ahmad Ardani, P.E.
Research Engineer
Colorado Department of Transportation
4201 E Arkansas
Denver, CO 80222
Ph: (303) 757-9978
Fax: (303) 757-9974
ahmad.ardani@dot.state.co.us

Neeraj Buch, Ph.D.
Assistant Professor
3546 Engineering Building
Department of Civil and
Environmental Engineering
Michigan State University
East Lansing, MI 48821
Ph: (517) 432-0012
Fax: (517) 432-1827
buch@egr.msu.edu

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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 5). 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.

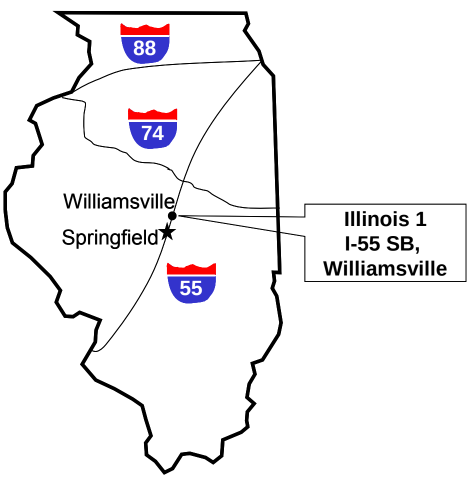

Figure 5. Location of IL 1 project.

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 6); 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).

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 762-mm (30-in.) intervals (Gawedzinski 2000).

Seven joints (excluding hinge joints) are included in the project, the layout of which is shown in Figure 7. 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, California. The final regular transverse joint in the project contains conventional epoxy-coated steel dowel bars.

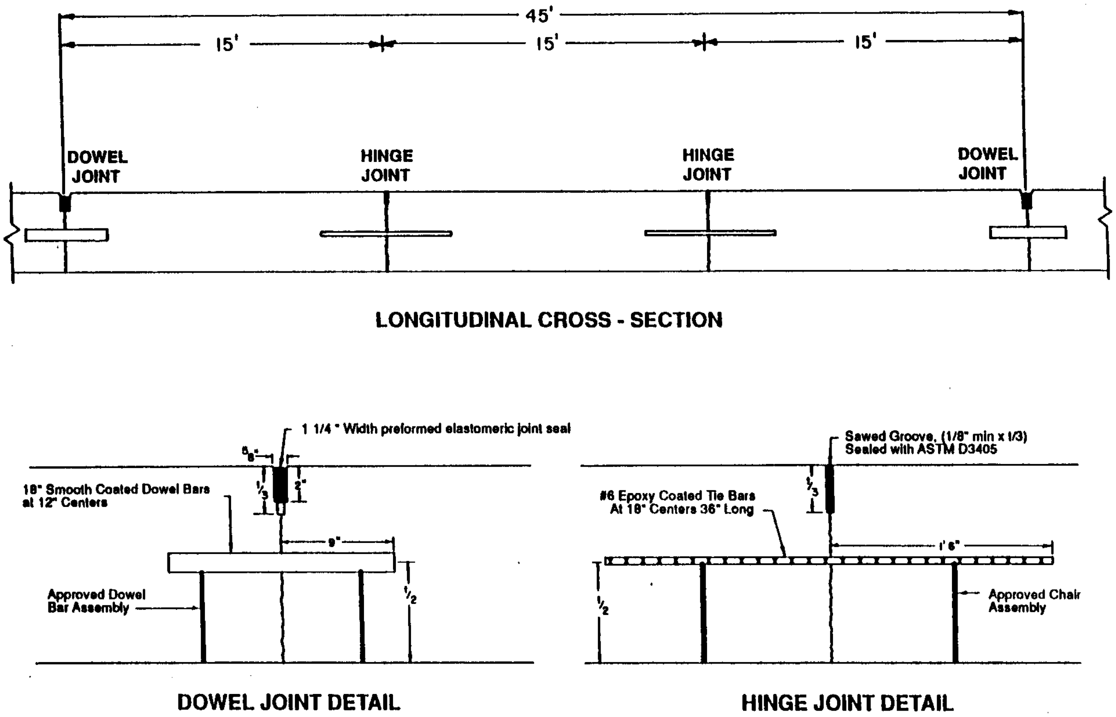

Figure 6. Illinois DOT hinge joint design (IDOT 1989).

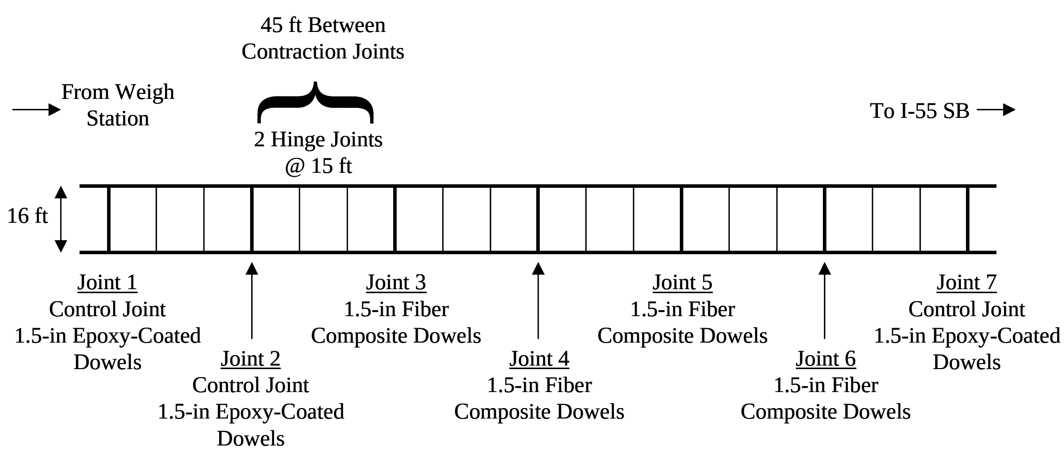

Figure 7. Layout of IL 1 project.

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 4 (Gawedzinski 2000).

Table 4. Traffic Data for IL 1 (September 1996 to September 1999) (Gawedzinski 2000)

TRUCK TYPE	NUMBER OF VEHICLES	ACCUMULATED 18-KIP ESAL APPLICATIONS
Single unit	95,623	31,324
Multiple unit	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 8 and 9 (Gawedzinski 2000). Figure 8 shows the load transfer efficiency (LTE) across each of the seven joints as a function of time, whereas Figure 9 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 8, with the conventional steel dowel bars consistently showing higher levels of load transfer then the fiber composite bars. But, as seen in Figure 9, the largest deflection is consistently shown by one of the conventional doweled joints, although the other two conventional doweled joints show consistently low deflections. LTE values less than 70 percent provide very low stress load transfer, and the results of the LTE testing suggest that many of the joints are exhibiting an unacceptable LTE level after only 7.5 years.

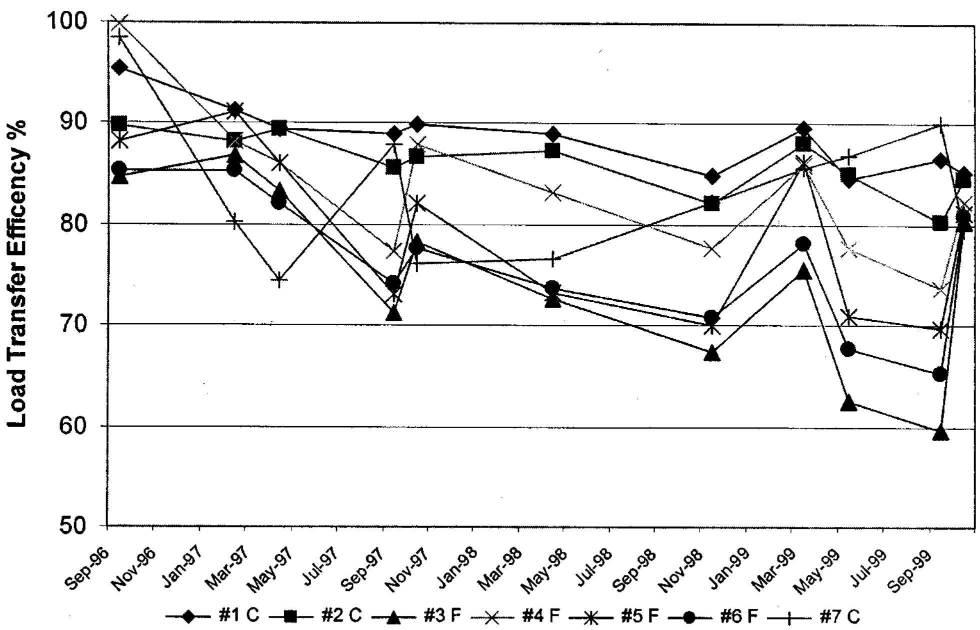

Figure 8. Load transfer efficiency on IL 1 (Gawedzinski 2000).

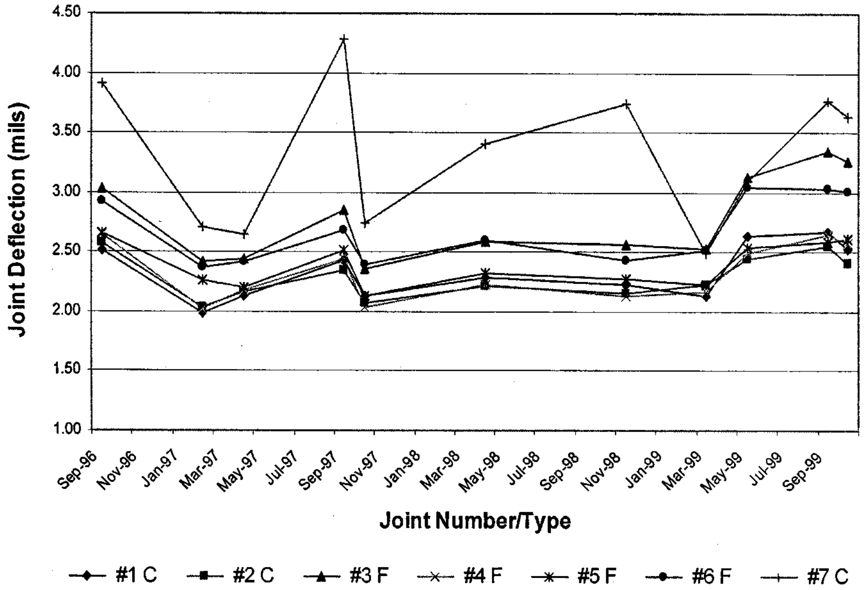

Figure 9. Maximum joint deflections on IL 1 (Gawedzinski 2000).

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, and 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 13.5-month period from January 23, 2002, to March 13, 2003. ESAL counts for the missing period 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 5 (Gawedzinski 2004).

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 15.75-mm (0.62-in.) wide joint. Over time, the preformed elastomeric joint material has been pushed deeper into the saw cut, especially in the wheelpaths. Deflection 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 wheelpaths and at the center of the lane for a total of 18 tests per joint.

Figure 10 (Gawedzinski 2004) provides a summary of the LTE verses ESALs, as measured over time. Figure 11 (Gawedzinski 2004) provides a graph of average pavement temperature at a 4-in depth verses LTE.

Current Observations (Gawedzinski 2004)

The Williamsville site is 7.5 years old and has been subjected to 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 wheelpaths. 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, but many of the joints (especially those equipped with FRP bars) are approaching (or have fallen below) the minimum acceptable LTE level of 70 percent.

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.

Points of Contact

David Lippert
(217) 782-8582

Mark Gawedzinski
(217) 782-2799
GawedzinskiMJ@dot.il.gov

Illinois Department of Transportation
Bureau of Materials and Physical Research
126 E. Ash Street
Springfield, IL 62704

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Table 5. Cumulative ESALs as of the
Day of Falling Weight Deflectometer Testing
(Gawedzinski 2004)

DATE	CUMULATIVE ESALS
9/26/96	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
6/11/03	9,841,810.9
10/2/03	10,075,492.5
10/24/03	10,103,714.9

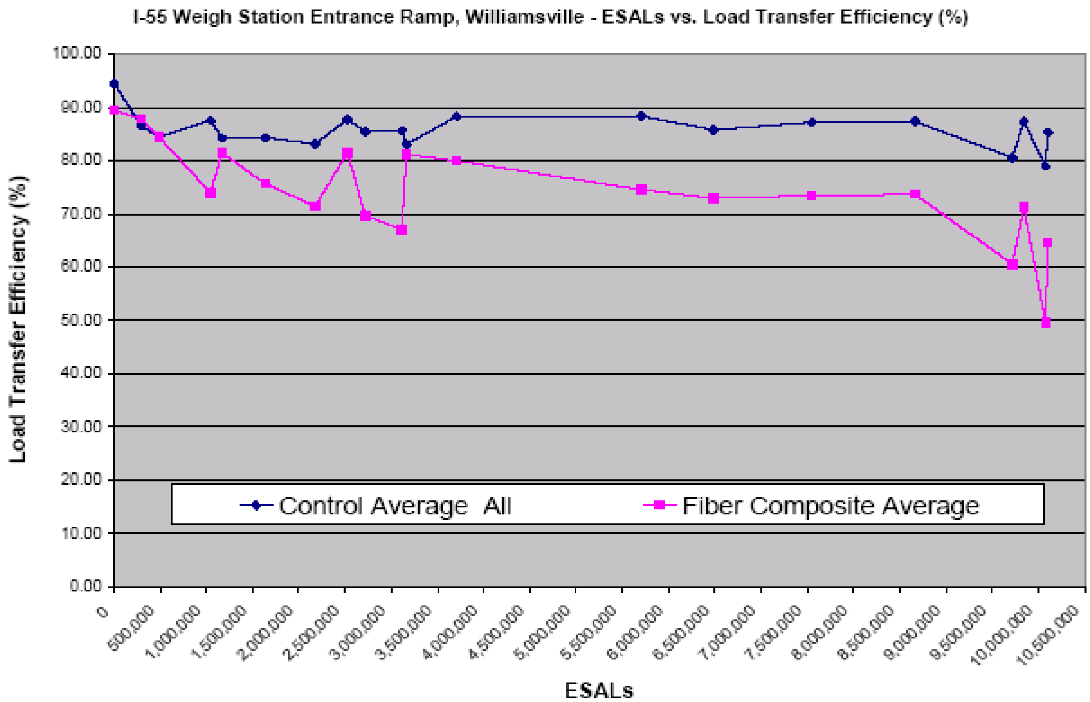

Figure 10. Load transfer efficiency vs. ESALs (Gawedzinski 2004).

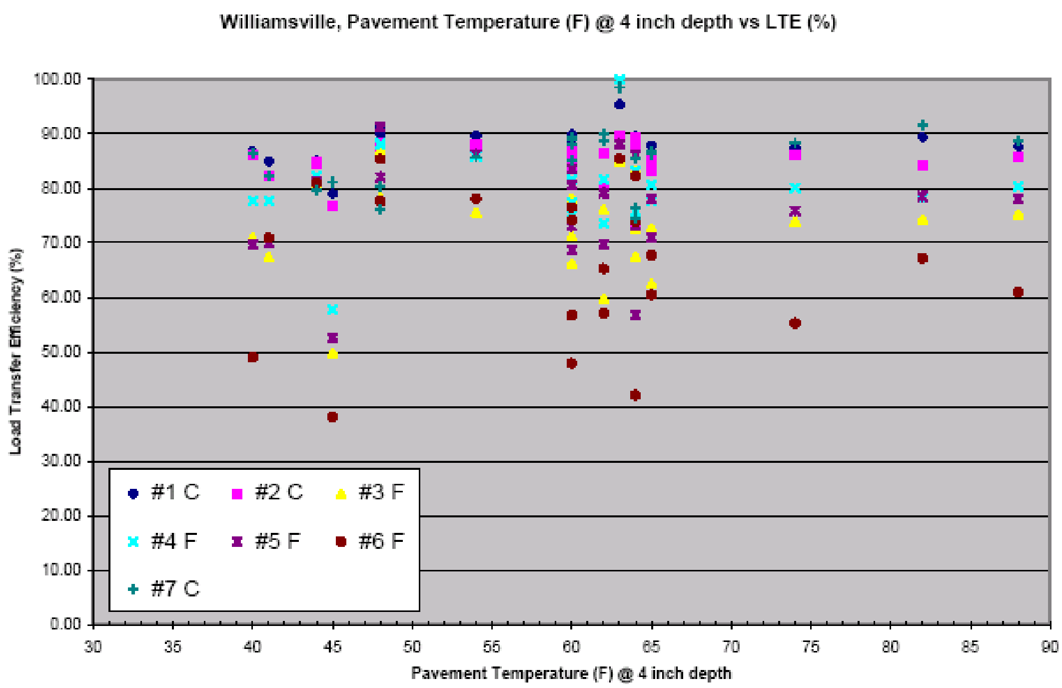

Figure 11. Load transfer efficiency vs. pavement temperature (Gawedzinski 2004).

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 12). 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).

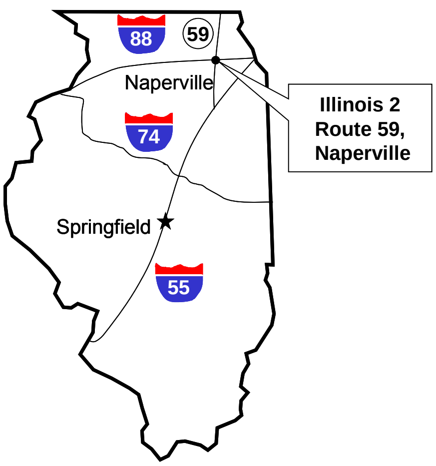

Figure 12. Location of IL 2 project.

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 6 is located beneath the aggregate base course (Gawedzinski 1997).

Table 6. Gradation of Porous Granular Embankment Subgrade 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 6). 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 13. This figure summarizes the main features included in each of the sections. The experimental design matrix for this project is shown in Table 7.

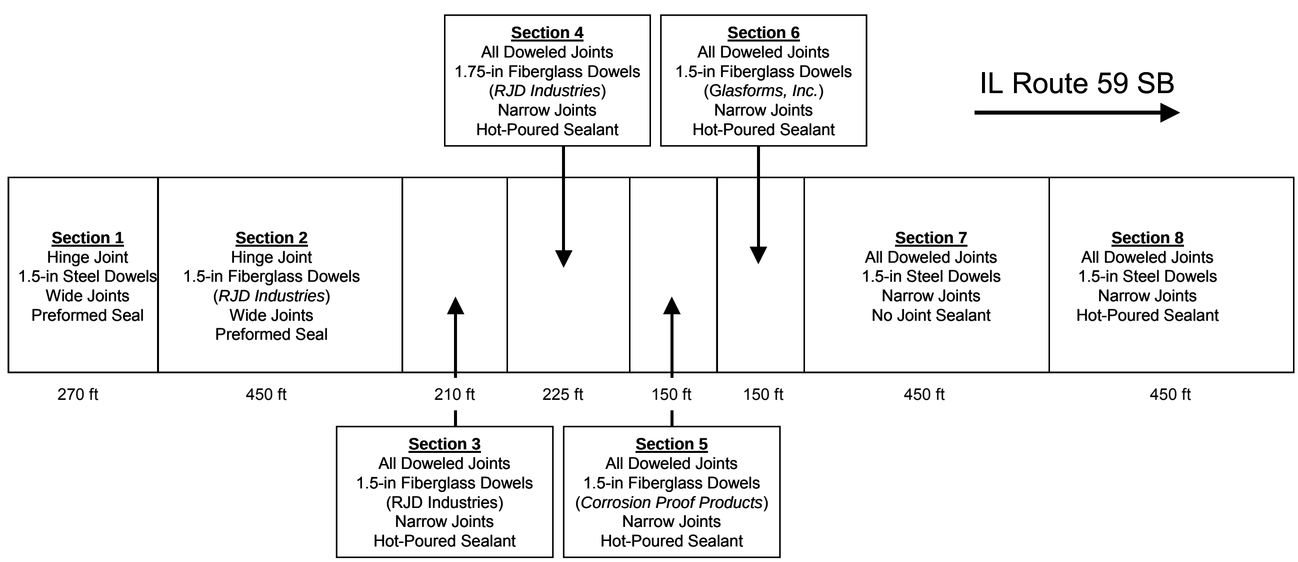

Figure 13. Layout of IL 2 project.

Table 7. 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)	

State Monitoring Activities

IDOT collects traffic data for the three southbound and three northbound lanes using two 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, because problems were encountered with the Groundhog® device no comparisons between the devices are possible (Gawedzinski 2000).

Traffic data for the three experimental southbound lanes are summarized in Table 8 (Gawedzinski 2000). The data are for the period 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).

This project is evaluated by IDOT on at least a semi-annual basis. Evaluation consists of both distress surveys and nondestructive testing using the FWD. Results from the FWD testing program are plotted in Figures 14 and 15 for sections 1 through 6 only (Gawedzinski 2000). Figure 14 shows the average load transfer for the six test sections as a function of time, whereas Figure 15 shows the average maximum joint deflection measured for the sections as a function of time. 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 temperature at the time of testing. These LTE values are considered marginal, particularly for a pavement that is only a few years old.

Table 8. 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

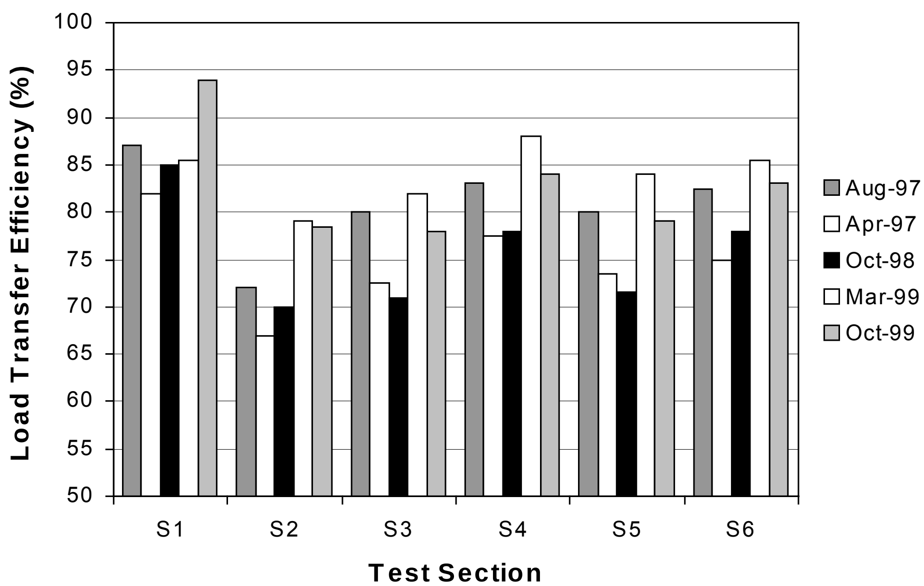

Figure 14. Load transfer efficiency on IL 2 (Gawedzinski 2000).

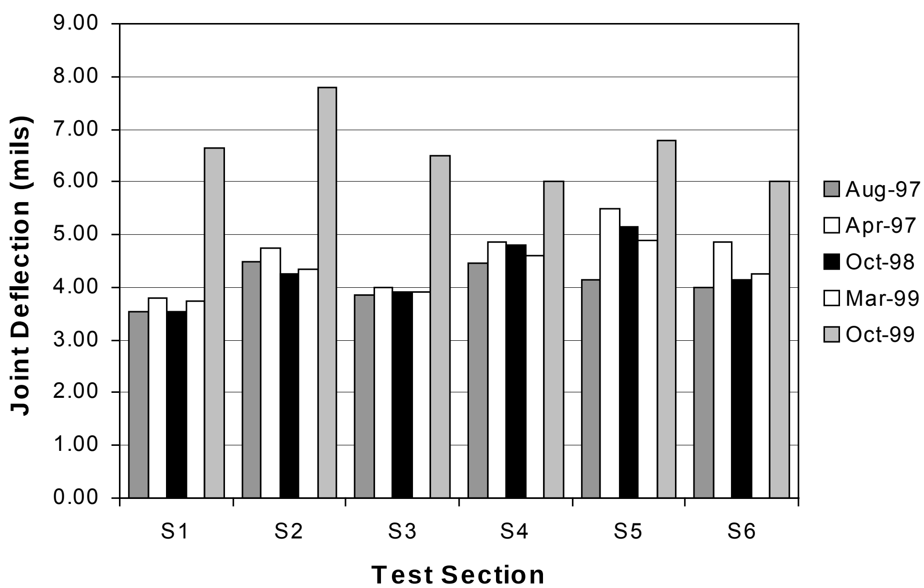

Figure 15. Maximum joint deflections on IL 2 (Gawedzinski 2000).

Figure 15 shows that the maximum deflections for all joints is increasing over time, with the maximum deflection during the October 1999 testing 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 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, and Findings

In August 2002, the Model 241Traffic 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 are used to convert single unit (SU) and multiple unit (MU) truck counts to ESALs. In May 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 0.6 km (0.4 mi) 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 9.

FWD tests are currently performed annually across all of the test sections. Certain sections were dropped from the FWD testing for a time due to traffic safety issues. These issues were resolved, and now FWD results are obtained for both wheelpaths 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 16 through 18 for the right, center, and left lanes, respectively.

Table 9. 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

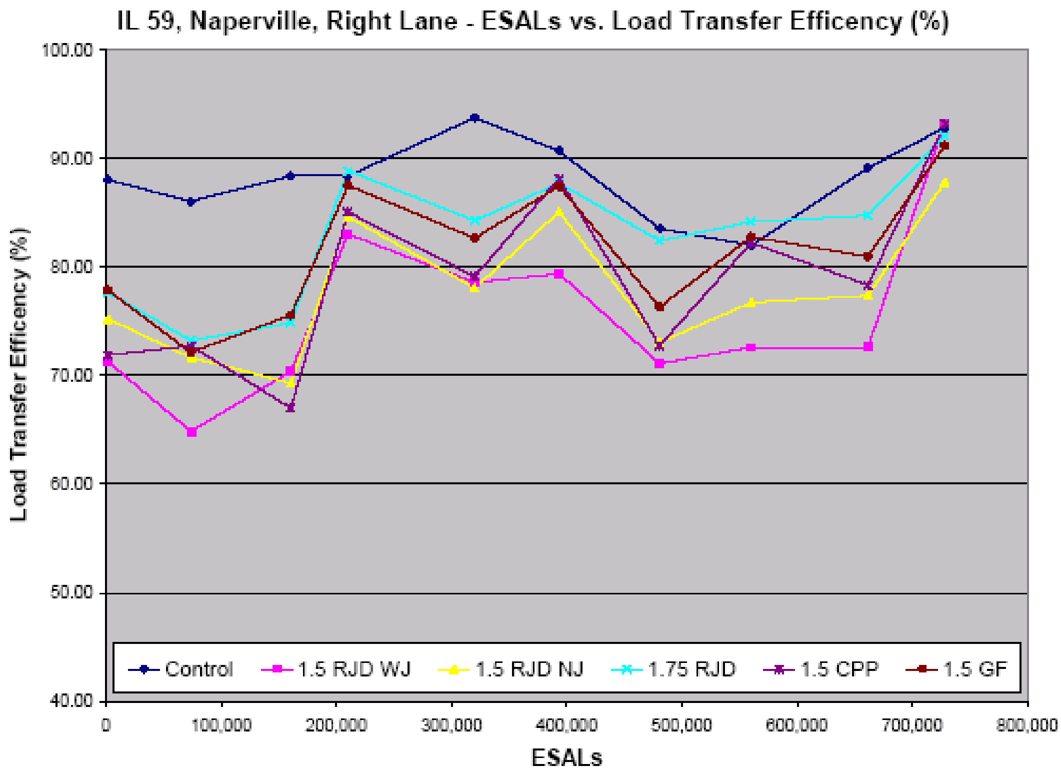

Figure 16. Load transfer efficiency vs. ESALs for the right lane (Gawedzinski 2004).

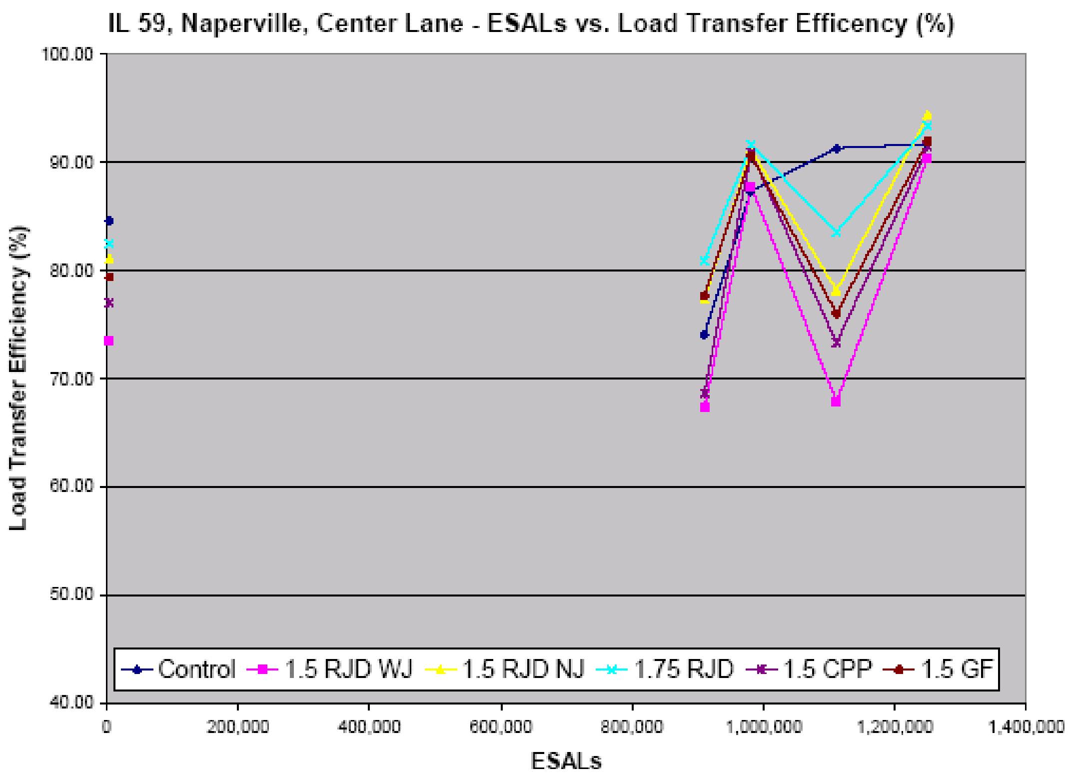

Figure 17. Load transfer efficiency vs. ESALs for the center lane (Gawedzinski 2004).

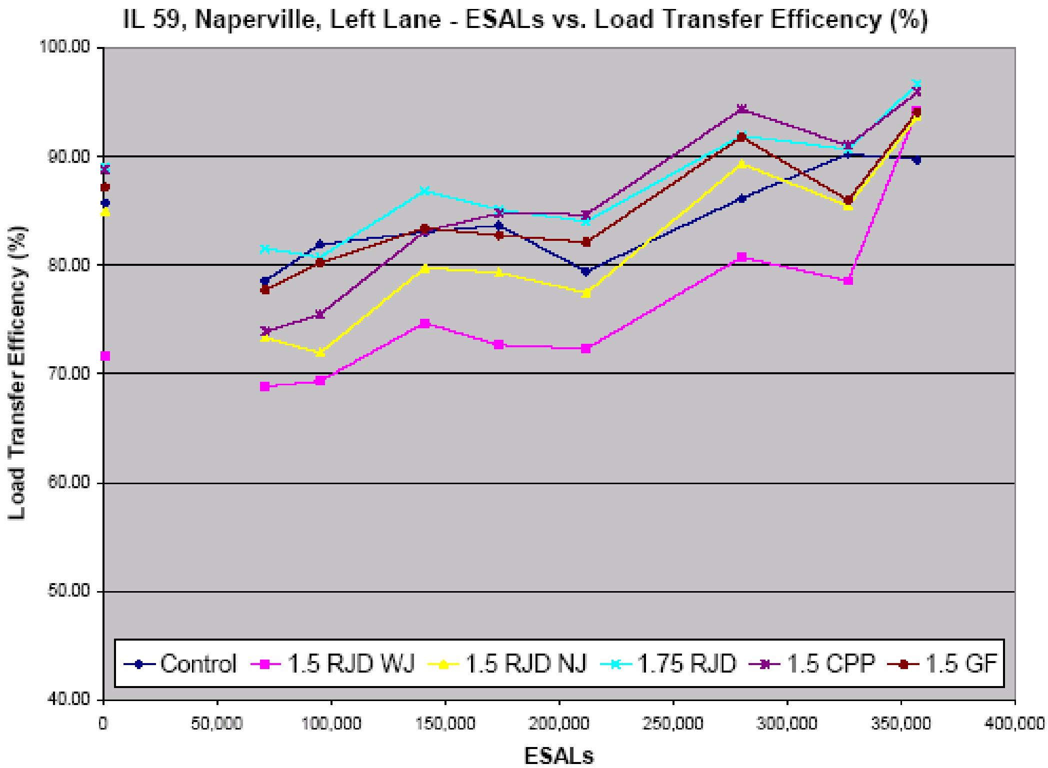

Figure 18. Load transfer efficiency vs. ESALs for the left lane (Gawedzinski 2004).

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 a 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 3 to 5 mils. The LTE graphs show behavior consistent with a decrease in joint deflection. Figure 19 shows the same type of behavior displayed at the Williamsville test site (IL 1). Plots of average values show no relationship between LTE or joint deflection and average pavement temperature. The control bars (38.1-mm [1.5-in.] diameter 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. Nevertheless, LTE values on the order of 70 to 80 percent after only a few years of service may suggest that the FRP bars are not suitable for long-term performance.

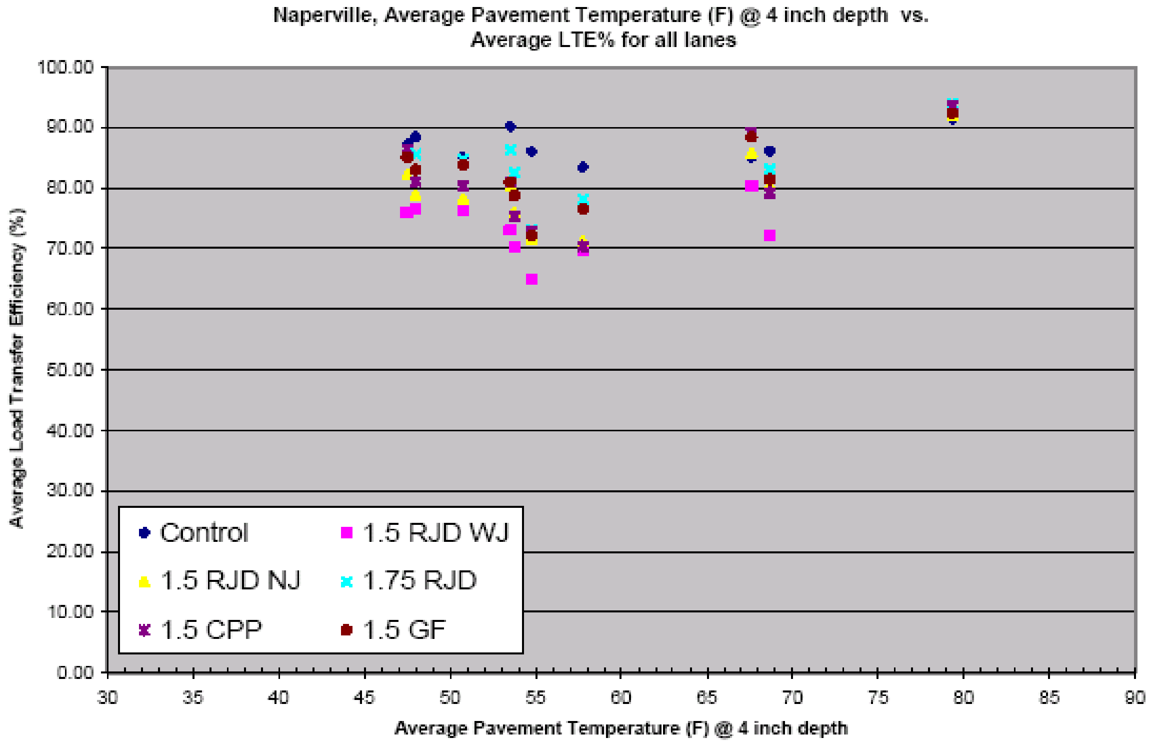

Figure 19. Average load transfer efficiency vs. pavement temperature for all lanes (Gawedzinski 2004).

Points of Contact

David Lippert
(217) 782-8582

Mark Gawedzinski
(217) 782-2799
GawedzinskiMJ@dot.il.gov

Illinois Department of Transportation
Bureau of Materials and Physical Research
126 E Ash Street
Springfield, IL 62704

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Chapter 7. ILLINOIS 3 (US 67, Jacksonville)

Introduction

The Illinois Department of Transportation's (IDOT's) second TE-30 project, and their third evaluating alternative dowel bar materials, is located on the two westbound lanes of US Route 67, west of Jacksonville (see Figure 20). This project was constructed in 1999.

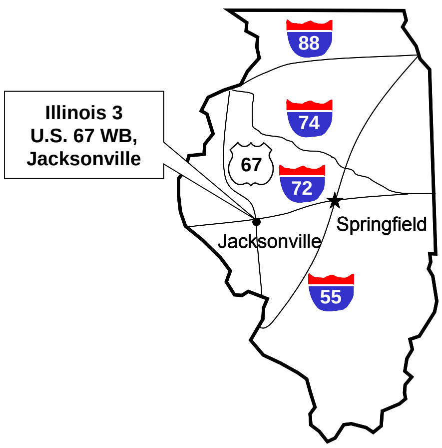

Figure 20. Location of IL 3 project.

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 for 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 21. This figure summarizes the main features included in each of the sections. The experimental design matrix for this project is shown in Table 10.

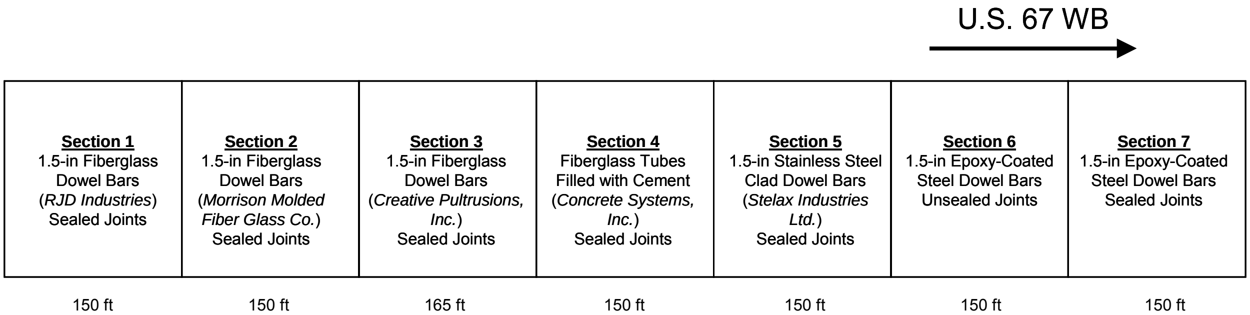

Figure 21. Layout of IL 3 project.

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. 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 22 and 23 (Gawedzinski 2000). Figure 22 shows the average load transfer for the seven experimental sections in both the driving and passing lanes, whereas Figure 23 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 (typically between 80 and 90 percent) 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.

Table 10. 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)

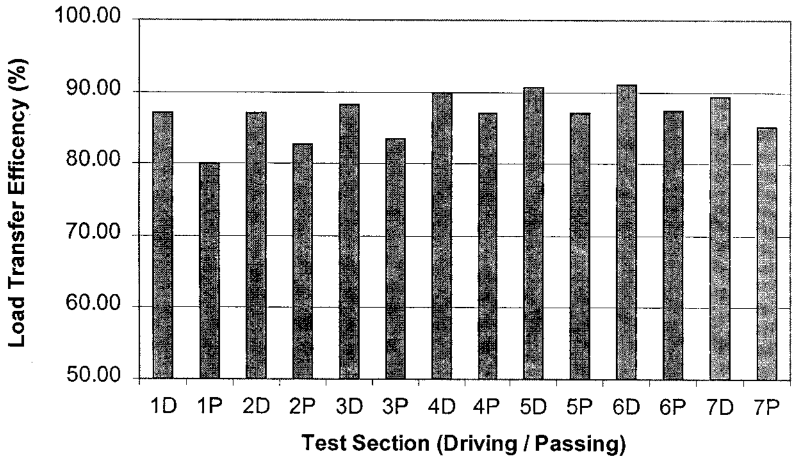

Figure 22. Load transfer efficiency on IL 3 (Gawedzinski 2000).

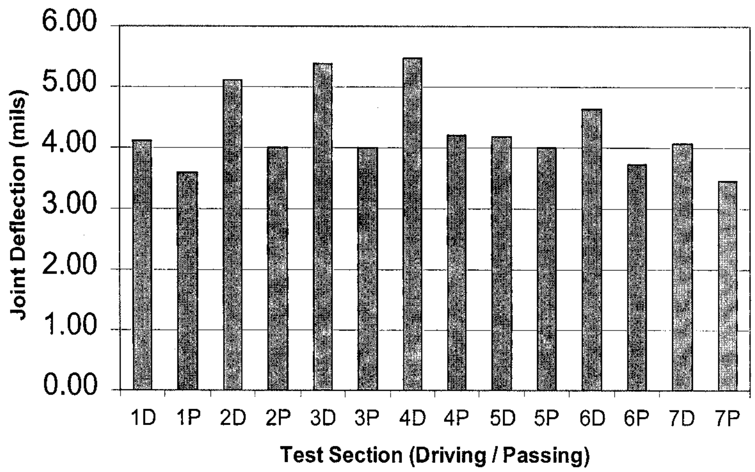

Figure 23. Maximum joint deflections on IL 3 (Gawedzinski 2000).

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, and Findings

FWD tests are conducted semi-annually along with periodic visual observations of joint performance. Traffic data are collected using an ADR 3000, manufactured by Peek Traffic. The data are periodically polled and converted to ESALs using standard IDOT conversion factors. A summary of the cumulative ESALs is provided in Table 11.

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 11. 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 wheelpaths or had been pushed deeper in the joint and was debonded from both sides of the pavement joint. Small rocks were also compressed into the joint seal material at the joint surface. As with the other Illinois sites, no obvious signs of joint distress were apparent during the visual observations.

Similar behavior as observed at the older two sites (IL 1 and IL 2) is shown in the following figures. The control set (38.1-mm [1.5-in.] diameter 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. The pavement at Jacksonville (IL 3) was constructed on a cement aggregate mixture subbase (CAM 2 with 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 that 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.

Deflection data are presented in Figures 24 through 26. It is observed that the FRP bars are approaching (or exceeding) the critical LTE value of 70 percent, calling into question the capability of these load transfer mechanisms to provide long-term performance.

Points of Contact

David Lippert
(217) 782-8582

Mark Gawedzinski
(217) 782-2799
GawedzinskiMJ@dot.il.gov

Illinois Department of Transportation
Bureau of Materials and Physical Research
126 E Ash Street
Springfield, IL 62704

References

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

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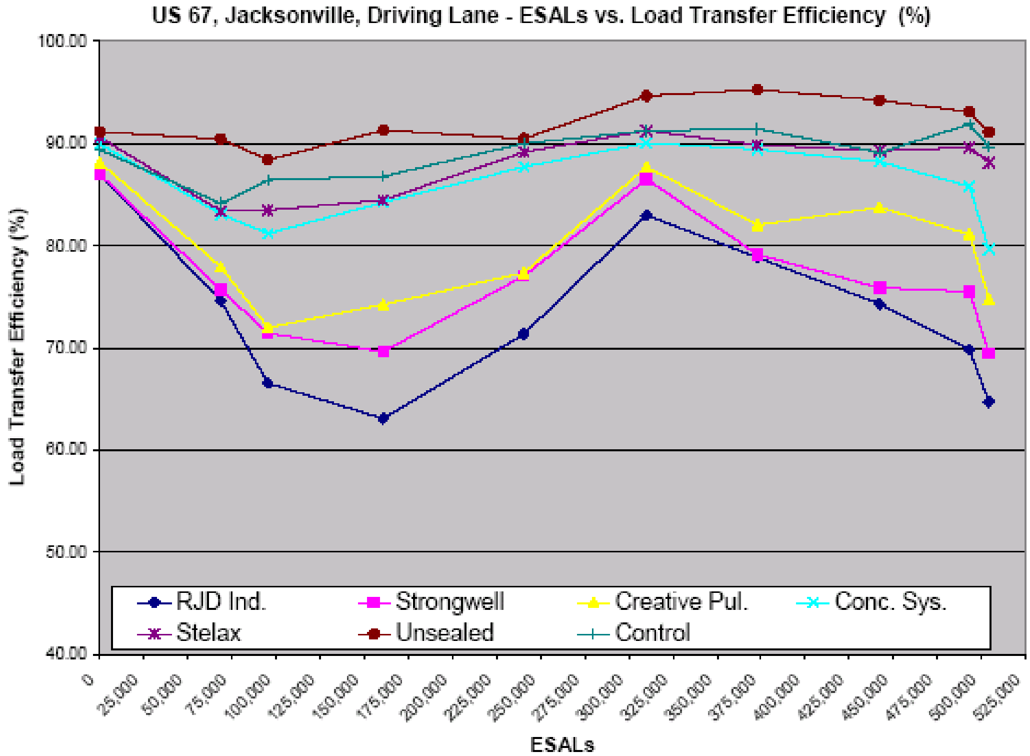

Figure 24. Driving lane load transfer efficiency vs. ESALs (Gawedzinski 2004).

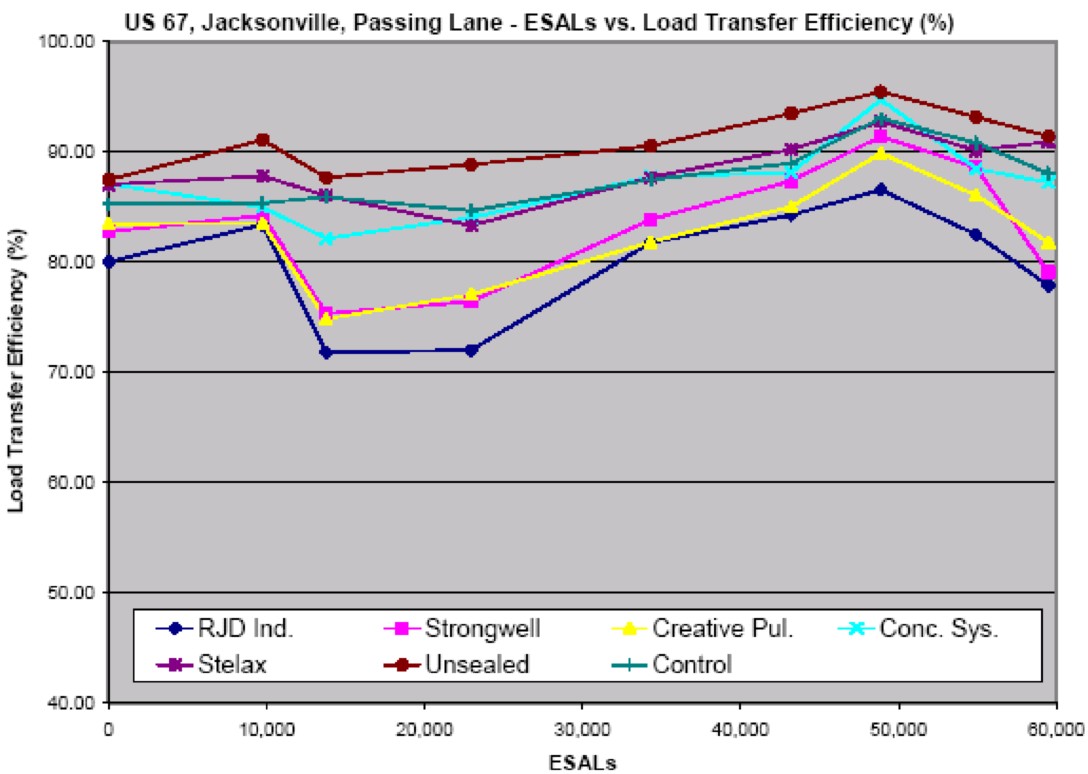

Figure 25. Passing lane load transfer efficiency vs. ESALs (Gawedzinski 2004).

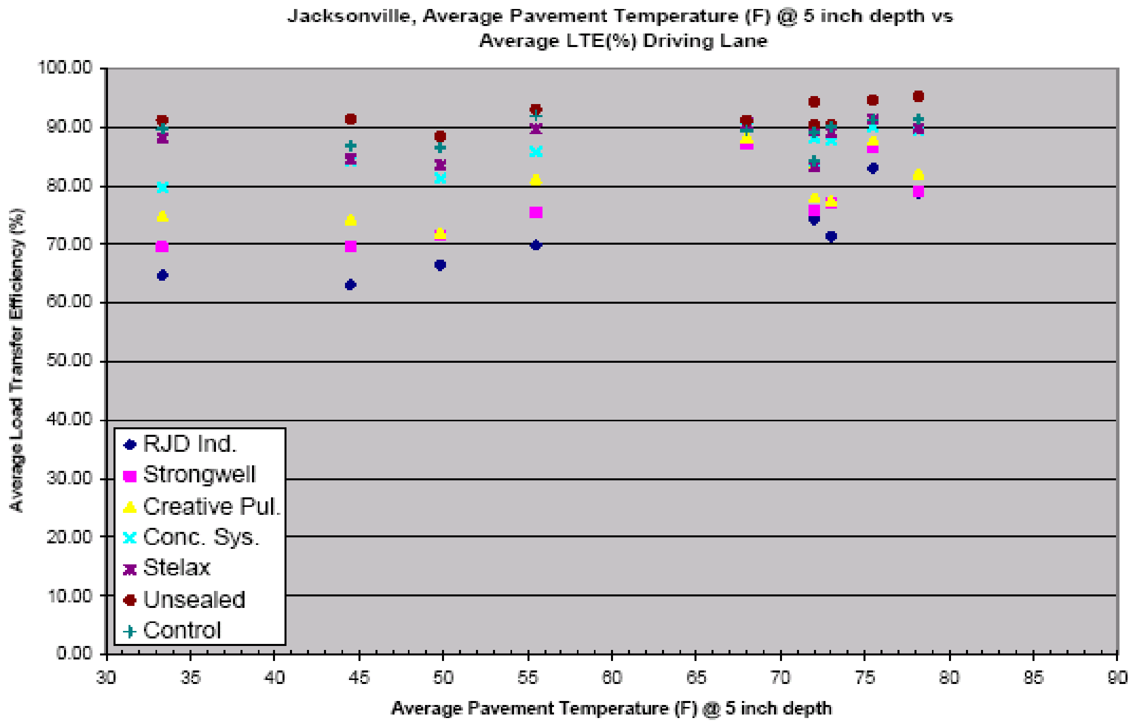

Figure 26. Average load transfer efficiency vs. average pavement temperature (Gawedzinski 2004).

Chapter 8. ILLINOIS 4 (Route 2, Dixon)

Introduction

A fourth project evaluating alternative dowel bars was constructed by the Illinois Department of Transportation (IDOT) in April 2000. The experimental project is located in the driving lane of the northbound direction of Illinois Route 2 in Dixon (see Figure 27) where it replaced an existing concrete pavement (Gawedzinski 2000).

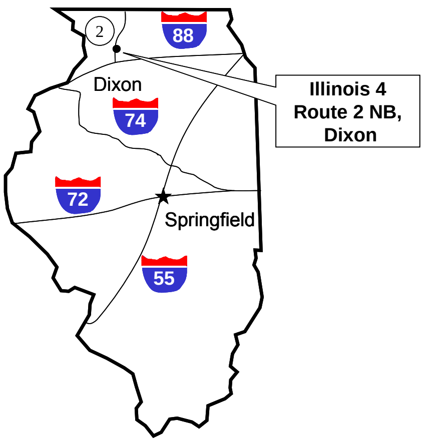

Figure 27. Location of IL 4 project.

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. More detailed construction information is provided by Gawedzinski (2004).

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 are 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, and Findings

Data has been collected on a semi-annual basis for the past 3 years. The cumulative ESALs are provided in Table 12. Results of deflection testing are illustrated in Figures 28 and 29.

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

Current Observations (Gawedzinski 2004)

At the time of construction, all of the test joints were unsealed. Visual observations 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 materials 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 for relatively new pavements.

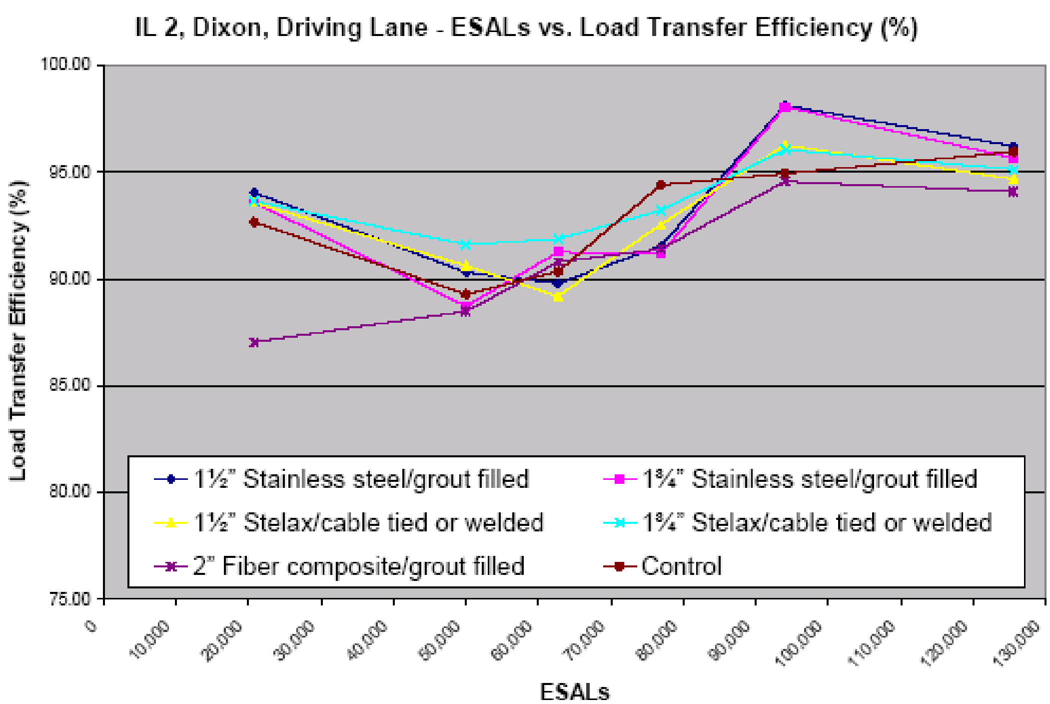

Figure 28. Driving lane load transfer efficiency vs. ESALs (Gawedzinski 2004).

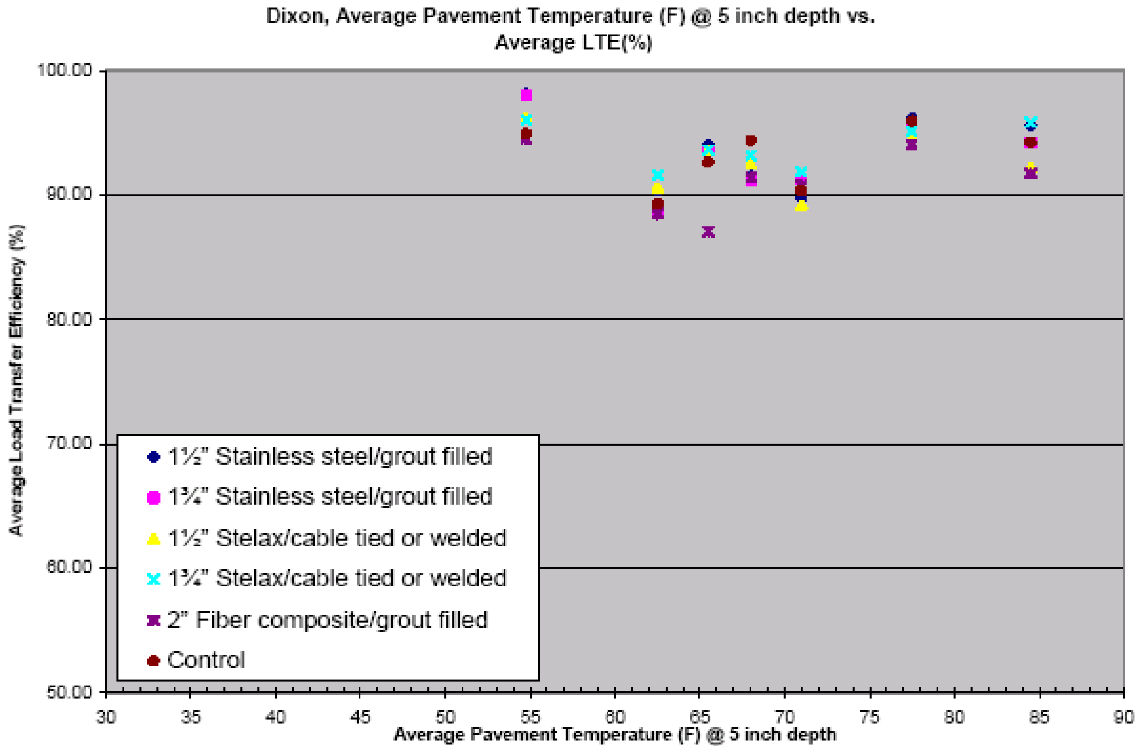

Figure 29. Average load transfer efficiency vs. average pavement temperature (Gawedzinski 2004).

Points of Contact

David Lippert
(217) 782-8582

Mark Gawedzinski
(217) 782-2799
GawedzinskiMJ@dot.il.gov

Illinois Department of Transportation
Bureau of Materials and Physical Research
126 E Ash Street
Springfield, IL 62704

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Chapter 9. INDIANA 1 (I-65 at SR-60, Clark County)

Introduction

Indiana is interested in building a jointed concrete pavement with an extended performance life. Research has suggested that jointed concrete pavements built with "low slab curvature" also have extended pavement life. This project, located on I-65 in Clark County (see Figure 30), includes various construction and materials properties that will be evaluated to assess concrete slab response to environment and the resulting stress/strain relationships.

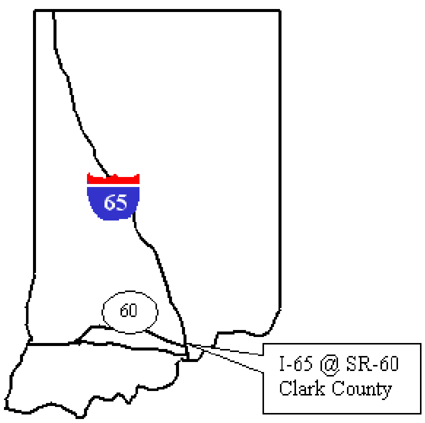

Figure 30. Location of IN 1 project.

Study Objectives

This project will investigate the relationship between specific construction and materials factors and long-term pavement performance by use of an instrumented pavement. These experimental sections will have 38 state-of-the-art vibrating wire strain gauges to monitor the strain in the pavement slabs, 5 tilt meters to monitor the curling direction and tendency, and 4 time domain reflectometers to determine the moisture content in the pavement base and subgrade. A total of 186 data acquisition channels and an automated data acquisition unit will be employed to monitor the behavior of the pavement slabs. In addition, there will be 75 iButton™ temperature sensors (sensors in the form of a button that has a processor and battery life of 3 years to process data inside the button) that will monitor the pavement temperature profile inch-by-inch, to determine the onset of temperature curling in concrete pavement. A GroundHog traffic monitoring system will be also in place to monitor the traffic and vehicle classifications.

A weather station to monitor air temperature, relative humidity, wind speed, wind direction, precipitation, rate of sub-surface temperature and solar radiation will provide data to predict the behavior of the pavement slabs, especially the correlation between climatic conditions and the pavement behavior response.

The results of this experimental project, when fully implemented, will minimize or eliminate the needs of early pavement rehabilitation and frequent pavement maintenance.

Project Design and Layout

Three one-directional lanes featuring a 355.6-mm (14-in.) thick slab will be constructed over a 2.49-km (1.55-mi) segment of I-65 during the 2004 construction season. Data analysis from this project will be directed toward evaluating the following:

- Low built-in curl (controlling temperature gradients during construction).
- Low sensitivity to moisture warping (low permeability).
- Low sensitivity to temperature gradients after construction (low thermal coefficients of concrete).
- Fracture properties, tensile strength, elastic modulus, entrained air, permeability, and thermal expansion from field concrete.
- Materials properties that impact the above properties include low w/c, aggregate soundness, and coarse aggregate fracture properties.

The estimated time of completion for this project is 36 months. The first 12 months will concentrate on a state-of-the practice review of modern instrumentation to measure important parameters that influence the performance of concrete pavement.

Static and dynamic sensors will be embedded in the pavement structure at specific locations to measure the appropriate parameters. The dynamic sensors are used to measure pavement response to traffic while the static sensors are used to measure the pavement response to changes of environmental conditions such as variations of temperature, moisture, and curling during construction, after construction, and in-service pavement. In addition, the traffic and weather conditions will be monitored by using a GroundHog and an RTWin weather station, respectively. Table 13 summarizes the instrumentation currently being considered for installation for this project (Nantung 2004).

The second 12-month period will consist of the data analysis described. This period will include laboratory testing in conjunction with the joint project with Purdue University to complete a finite element analysis, damage model, slab contact model, layered base model, and thermal gradient model. Data analysis from this project will be analyzed using the new finite element models to determine the damage analysis of two different dimensions of PCC slabs.

The final 12 months will include additional analysis and verification of all data analysis to be included in a final report.

State Monitoring Activities

Field monitoring will be limited to the first 12 months after construction.

Preliminary Results/Findings

Selection of the state-of-the-art monitoring equipment has been completed. Currently Indiana DOT is performing laboratory checks on all equipment prior to implementation during construction. A summary of the proposed instrumentation for this project is shown in Table 13 (Nantung 2004).

Points of Contact

Lee Gallivan
FHWA-Indiana Division
575 N Pennsylvania St., Room 254
Indianapolis, IN 46204
(317) 226-7493

Tommy E. Nantung
(765) 463-1521 x248
TNANTUNG@indot.state.in.us

Scott Newbolds
(765) 463-1521 x 244

INDOT Research Division
1205 Montgomery
P.O. Box 2279
West Lafayette, IN 47906

Reference

Nantung, T. E. 2004. *High Performance Concrete Pavement*. Indiana Public Works News, Volume 5, Number 1. Anchor Media Co.

Table 13. Summary of Proposed Instrumentation (Nantung 2004)

SENSOR TYPE	MEASUREMENT TYPE	PURPOSE
Vibrating Wire Tiltmeter	Tilt angle	Measure the curling at all four corners of the slab.
Time Domain Reflectometer	Moisture	Measure the moisture content of the soil under the slab.
IButton™	Temperature	Measure the temperature profile in the pavement.
GroundHog™	Traffic	Measure vehicle volume, vehicle speed and length, road surface temperature, roadway wet/dry condition, and amount of salt used for anti-icing.
RTWin™ Weather Station	Weather	Measure air temperature, relative humidity, wind speed, wind direction, and precipitation.

Chapter 10. IOWA 1 (a and b) (Highway 5, Carlisle, and US 30, Carroll)

Introduction

The Iowa Department of Transportation's (DOT's) first TE-30 project consists of an evaluation of the effect of concrete mixing time on several critical plastic and hardened concrete properties. This investigation was conducted using several different mix designs for pavements constructed at two different sites: Iowa Highway 5 near Carlisle and US 30 near Carroll (Cable and McDaniel 1998a). The locations of these sites are shown in Figure 31.

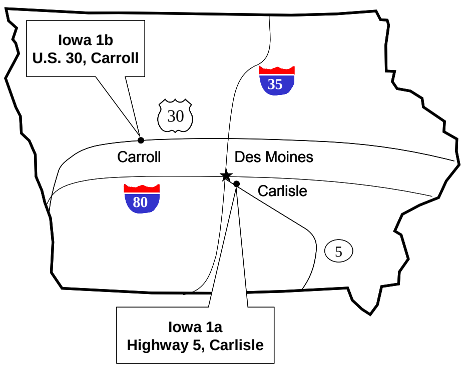

Figure 31. Location of IA 1 projects.

Study Objectives

The issue of mixing time has become a concern to the Iowa DOT as several plant manufacturers have claimed consistent and sufficient mixing in as short as 30 seconds (Cable and McDaniel 1998a). Therefore, the primary goal of this research project is to investigate the effect of concrete mixing time on the resultant air content, air distribution, consolidation, and workability of concrete used for pavement construction (Cable 1998). Secondary objectives include the evaluation of a contractor-designed concrete mix and the evaluation of an alternative mixer.

Project Design and Layout

The investigation of mixing time on concrete properties was conducted at both the Carlisle and Carroll test sites. Nominal mixing times of 30, 45, 60, and 90 seconds were selected for evaluation, and two different mix designs were used: a standard mix developed by the Iowa DOT and a contractor-developed mix (Cable and McDaniel 1998a). No information on the exact composition of the two mixes is available, but the contractor mix reportedly is a "Shilstone" mix containing a uniformly graded aggregate.

Two different mixers were also employed in the study. A standard 7.65 m^3 (10 yd^3) drum mixer was used at the Carlisle project, whereas a modified drum mixer employing rotation of blades within the drum was used at the Carroll project (Cable and McDaniel 1998a). The same contractor was employed for the construction of each paving project. The experimental design matrix for this study is shown in Table 14.

Table 14. Experimental Design Matrix for IA 1

Mixing Time (seconds)	CARLISLE TEST SITE (STANDARD DRUM MIXER)		CARROLL TEST SITE (MODIFIED DRUM MIXER)	
	Iowa DOT Mix	Contractor Mix	Iowa DOT Mix	Contractor Mix
30			X	
45	X	X	X	
60	X	X		
90	X	X		

State Monitoring Activities

To achieve the objectives of this project, the Iowa DOT and the Iowa State University jointly participated in the testing and monitoring of the concrete mixing and paving activities. The paving was performed in the summer of 1996.

The testing methods in ASTM C 94 were used in this study to determine the significance of the mixing time on the consistency of the concrete mix delivered and placed on grade (Cable and McDaniel 1998a). ASTM C 94 is designed to check the consistency of the material at the beginning and near the end of the truck discharge. Using this standard, measurements of slump, unit weight, air content, retained coarse aggregate, and compressive strength were obtained and used to compare the consistency of the mix at different points of delivery (Cable and McDaniel 1998a).

At both the Carlisle and Carroll sites, the tests listed in Table 15 were conducted for each combination of mixing time and mix design included in the investigation. For the slump, unit weight, plastic air content, and wash tests, samples were obtained from three different locations for the same load of material: the center of the haul truck, the side of the haul truck, and on grade in front of the paver (Cable and McDaniel 1998a). Compressive testing of cylinders and cores was performed on concrete retrieved from the same batch as for the slump, unit weight, air content, and wash tests (Cable and McDaniel 1998a). Air void distribution testing was conducted on hardened concrete cores also taken from the same batch (Cable and McDaniel 1998a).

The haul trucks to be tested were selected at random at approximately ½- to 1-hour increments (Cable and McDaniel 1998a). Trucks were selected for testing only when the paver and plant were in continuous operation to ensure representative samples. Sufficient concrete was obtained from each truck to provide for tests of slump, unit weight, air content, and for the preparation of cylinders for compressive testing (Cable and McDaniel 1998a). Upon completion of the unit weight test, the material in the unit weight bucket was washed through a sieve to remove all fines and cement, and then the retained coarse aggregate was weighed and compared to the unit weight and the expected weight of coarse aggregate in the unit weight bucket (Cable and McDaniel 1998a).

Table 15. Tests Conducted at IA 1 Test Sites

TEST	TEST METHODOLOGY	TESTING LOCATIONS/SPECIMENS
Slump	Conducted in accordance with ASTM C143	Center of truck Side of truck On grade in front of paver
PCC unit weight	Conducted in accordance with ASTM C138	Center of truck Side of truck On grade in front of paver
Air content (plastic concrete)	Conducted in accordance with ASTM C231	Center of truck Side of truck On grade in front of paver
Wash test	Conducted in accordance with ASTM C94	Center of truck Side of truck On grade in front of paver
Compressive strength (cylinders)	Conducted in accordance with ASTM C42	Cylinders cast from each specific batch
Compressive strength (cores)	Conducted in accordance with ASTM C42	Cores obtained from known batch locations
Air void distribution (cores)	Measured using low-vacuum electron microscope and computer imaging analysis	Cores obtained behind the paver

The approximate location of a sampled batch of concrete in the pavement was recorded during the paving operation for later coring operations (Cable and McDaniel 1998a). Core sampling was conducted at the noted locations after the concrete had reached a strength of 3.4 MPa (500 lbf/in^2).

The instruments used for the air void analysis of hardened concrete cores were a Hitachi 2460 N low-vacuum scanning electron microscope, a Tetra back-scattered electron detector, Deben stage automation, and an Oxford Instrument ISIS x-ray analysis system (Cable and McDaniel 1998a). Samples were prepared from the cores, and special software was used to determine the area and size of the air voids in each image (Cable and McDaniel 1998a).

No further monitoring or reporting is anticipated. This project has now been completed.

Results/Findings

Extensive statistical analyses were conducted on the data collected from each test site. Comparisons of key concrete properties (slump, unit weight, air content, and compressive strength) were made between testing location (side, center, or on grade) and between the various mixing times (30, 45, 60, and 90 seconds). Based on these analyses, the following general conclusions were drawn (Cable 1998; Cable and McDaniel 1998a):

- Dump-truck-type hauling units do not significantly change the quality of the material being delivered to the paver and should continue to be allowed in addition to agitator-type hauling vehicles.
- Mixing times of 60 seconds or greater do have a positive influence on the physical characteristics of the concrete product and should be retained as the minimum mixing time for all mixer types.
- Mixing times did not significantly affect the hardened air content or distribution for the Iowa DOT mix designs, but the data showed conflicting results for the contractor-designed mix. This may be the result of a different matrix of coarse and fine aggregates in the contractor mix. Therefore, it is recommended that contractor mix designs should be thoroughly laboratory tested prior to use in the field to determine the impact of admixtures and differences in aggregate/cement matrix on the physical performance characteristics of the mix.
- Mixing times of less than 60 seconds should be allowed only when steps have been taken to change the mixing process to ensure coating of all aggregates prior to mixer discharge into the hauling unit.
- Visual examination of the mix at the Carlisle site (30- and 45-second mixing times) indicated visible sand seams (uncoated sand particles) in the discharged material. The concrete produced under this set of mixing conditions was also noted to be very difficult to place and finish.

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Points of Contact

Jim Cable
Iowa State University
Department of Civil and
Construction Engineering
378 Town Engineering Building
Ames, IA 50011
(515) 294-2862

Mark Dunn
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50011
(515) 239-1111

References

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Chapter 11. IOWA 2 (US 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 US 65 Bypass near Des Moines (Cable and McDaniel 1998b). Figure 32 shows the location of this project.

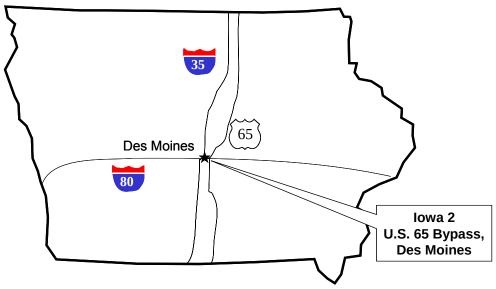


Figure 32. Location of IA 2 project.

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 US 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 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 16, and the layout of the test sections is shown in Figure 33. The dowel bar spacing configurations used on this project are illustrated in Figure 34.

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

Table 16. 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)	

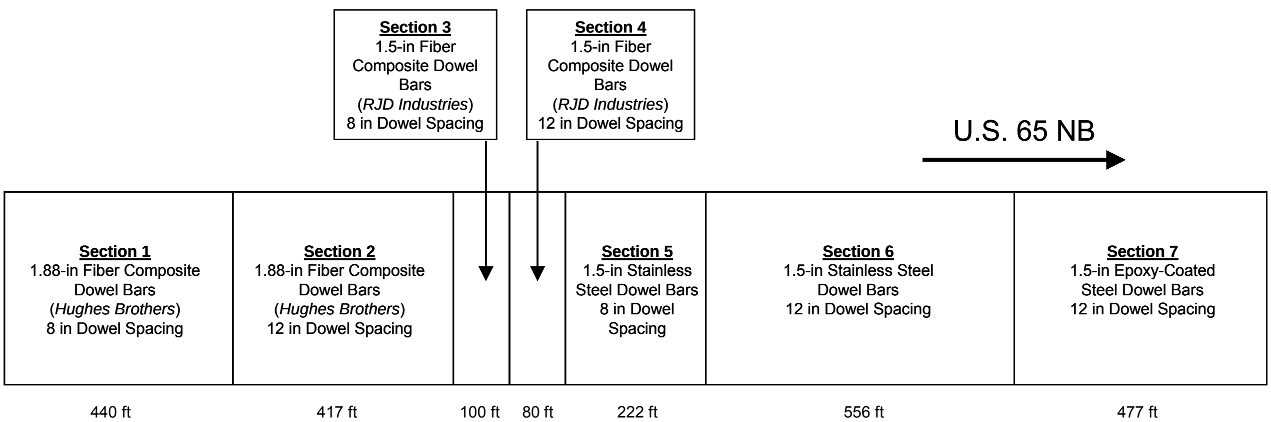


Figure 33. Layout of IA 2 project.

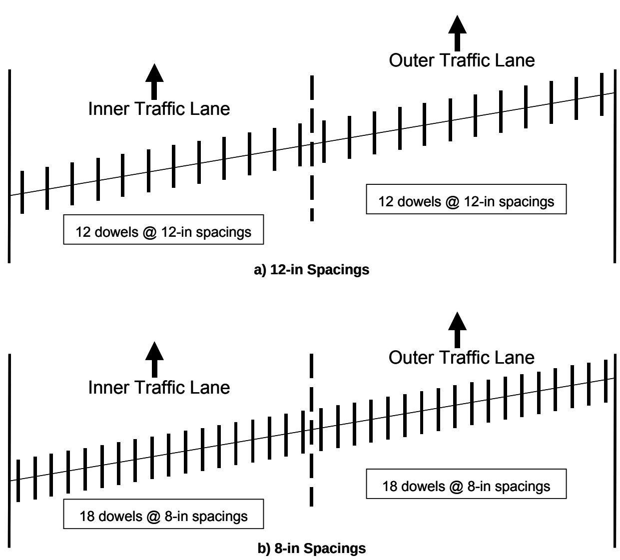


Figure 34. Illustration of dowel bar spacing configurations on IA 2.

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 masonry 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.

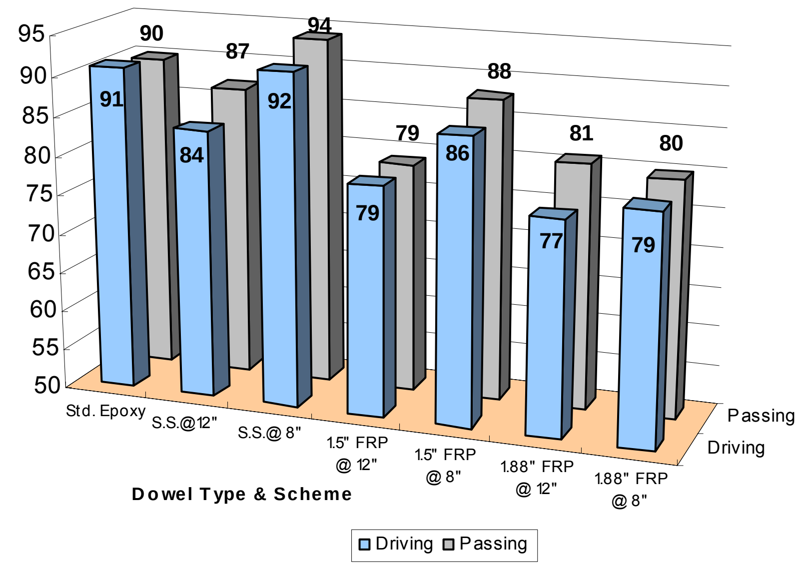
- Nails were attached to the bottom of the fiber composite tie bars to facilitate their location using both cover meters and GPR.

- 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

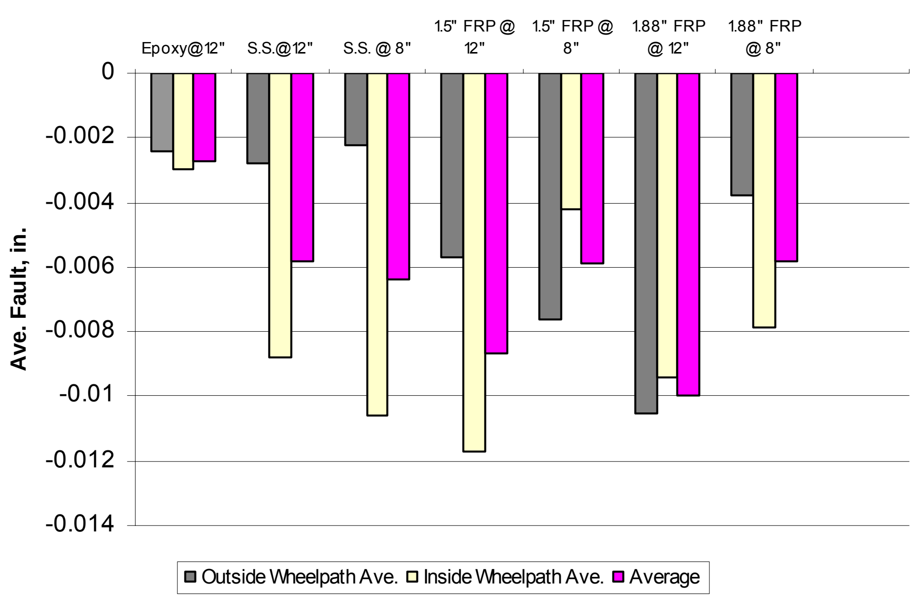
Project test sections were tested twice a year, beginning in the fall of 1997, with the final tests in the spring of 2002 (testing could not be performed in the fall of 2000). The results of the FWD testing were interpreted through calculating LTE.

The results of the load transfer analysis are illustrated in Figure 35 (Cable and Porter 2003). In Figure 35, the dowel bars are labeled according to their material and spacing: standard epoxy (std. epoxy), stainless steel (S.S.), and fiber composite (FRP). Figure 36 displays the overall average faulting over the period of research (Cable and Porter 2003). Figure 37 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 5-year monitoring period (Cable and Porter 2003).



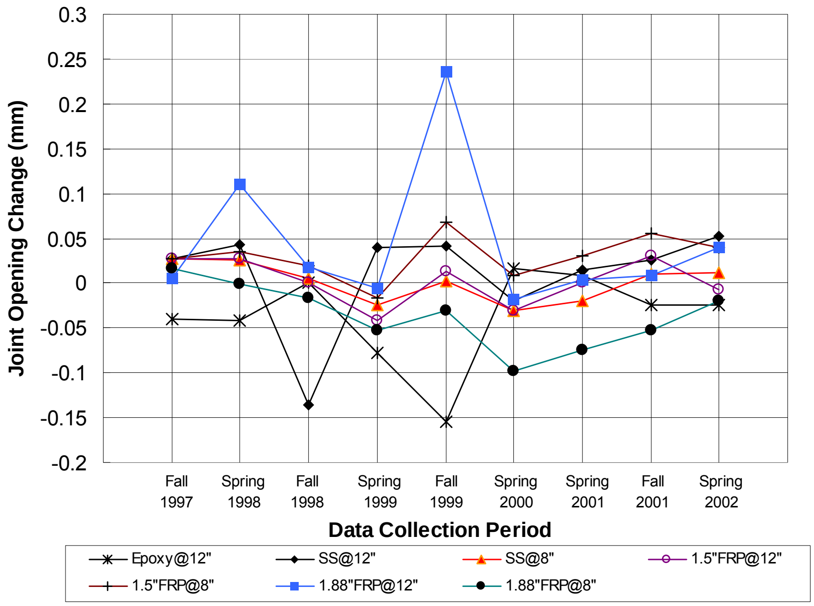
Note: FRP = fiber composite; S.S. = stainless steel; Std. Epoxy = standard epoxy

Figure 35. Average load transfer efficiency for IA 2 project.



Note: FRP = fiber composite; S.S. = stainless steel

Figure 36. Average faulting on IA 2 project.



Note: FRP = fiber composite; S.S. = stainless steel; Std. Epoxy = standard epoxy

Figure 37. Joint opening trends on IA 2.

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 5-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 5 years.
- FRP dowels of the sizes tested in this research should be spaced no greater than 203 mm (8 in.) apart to gain load transfer performance at the same level as epoxy-coated steel dowels at 305-mm (12-in.) spacing.
- No deterioration due to road deicers was found on any of the dowel materials retrieved in the 2002 coring operation.

Points of Contact

Jim Cable
Iowa State University
Department of Civil and Construction Engineering
378 Town Engineering Building
Ames, IA 50011
(515) 294-2862

Mark Dunn
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50011
(515) 239-1447

References

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.

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.

Chapter 12. IOWA 3 (US 151, Lynn/Jones Counties)

Introduction

This project is located on US 151 in Linn/Jones Counties (see Figure 38) and is studying the use of fly ash for soil stabilization of subgrades beneath concrete pavements. The scope of the project includes the use of two different qualities of fly ash and two different qualities of select fill.

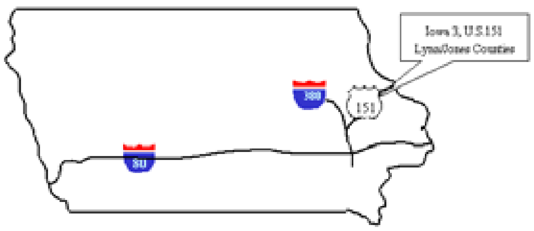


Figure 38. Location of IA 3 project.

Study Objectives

The objective of this research is to evaluate and monitor the performance of pavements constructed on treated subgrades with respect to control sections.

Project Design and Layout

The subgrade and pavement for this project are of conventional design and thickness, with the only difference in the test sections being the presence of 15 percent fly ash used to treat the subgrade. The quantity of fly ash to be used is based upon research performed by Dr. Ken Bergeson at Iowa State University (ISU). Two types of fly ash are being evaluated: (1) Stoker ash, which is a Class C fly ash that has a carbon content too high to be used in concrete mixtures, and (2) conventional Class C fly ash (this ash has been used in many stabilization roles and is included as a control). Two types of existing subgrade soils will be evaluated: (1) An approved select glacial clay loam; and (2) a higher plasticity select glacial clay loam. Control sections with untreated soil of both types will also be included. Some chemical evaluation of the fly ashes has already been performed at ISU. Additional tests from each fly ash source, as delivered, will be performed by ISU to monitor chemical variability.

The procurement, placement, and mixing of subgrade components will be performed by the contractor for the project under change order to an existing contract. Each test regime will be approximately 1.6 km (1 mi) long. The fly ash will be incorporated into the subgrade using a pulverizer and will be placed over the full shoulder-to-shoulder road width. The subgrade will be stabilized the full width of the pavement and to a depth of 304.8 mm (12 in.). The base material will be Iowa DOT standard untreated granular drainable base.

State Monitoring

The Iowa DOT will perform subgrade modulus tests before, during, and after placement of the subgrade using a dynamic cone penetrometer. Additional bearing tests will be performed after the subgrade has been placed. Moisture testing of the soil will be performed on samples taken at the borrow site, on grade, and from the final mixes. Samples of soil will be collected on grade and tested for gradation and soil properties. Concrete slump, strength, and air content will be reported for each test section. After paving is complete, deflection, smoothness, and ride testing will be performed annually for 5 years. Curling and warping will be identified from the smoothness data. Deflection differences between slabs exhibiting curling and warping and those without will be investigated. The impact of subgrade treatments on curling and warping, if any, will be noted.

Preliminary Results/Findings

A construction report, including photographs and video documentation, will be submitted by the Iowa DOT to FHWA. A progress report will be submitted after deflection testing is finished, summarizing the performance and noting any problems. The final report prepared by the Iowa DOT will be due 5 years after completion of the final pavement.

In January 2004, Iowa DOT indicated that a draft construction report had been completed, but it had not yet been made available.

Points of Contact

Mark Dunn
Research Engineer, Materials Office
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
(515) 239-1447

References

None.

Chapter 13. IOWA 4 (IA 330, Jasper, Story, and Marshall Counties)

Introduction

The Iowa DOT research project Dowel Bar Optimization: Phase I and II (Porter, Guinn, and Lundy 2001) and information from other dowel bar research studies (Porter and Guinn 2002) provided laboratory research results on spacing and bearing stresses for installing conventional round dowels and elliptical dowels. Field evaluation and calibration of the results are important to the application of the results to the pavement design process. To achieve this goal, the Iowa State University, in conjunction with the Iowa Department of Transportation (Iowa DOT), investigated the relative performance of elliptical and conventional round dowels in field conditions. Test sections were constructed in 2002 on Iowa 330 in Jasper, Story, and Marshall counties. Figure 39 shows the location of this project.

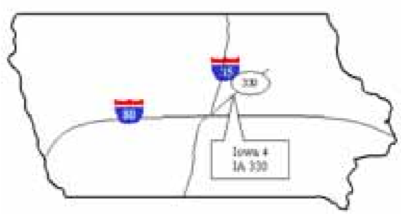


Figure 39. Location of IA 4 project.

Study Objectives

This project is evaluating the performance of elliptical dowels and the constructibility of such devices in an actual construction project. Specifically, the field research is designed to answer the following questions (Cable, Edgar, and Williams 2003):

- What is the relative performance of medium- and large-sized elliptical steel dowels as compared to that of conventional steel dowels?
- What is the impact of dowel spacing on the relative performance of the elliptical and round dowels in field conditions?
- What is the impact on performance of the various dowel shapes when placed in cut or fill sections of the roadway?
- What constructibility problems, if any, are associated with the installation of dowel shapes other than round?

Project Design and Layout

Two types of elliptical steel dowels (medium and heavy) in addition to conventional 38-mm (1.5-in.) round steel dowels were installed and monitored in this project. The specifications of the dowel bars are as follows (Cable, Edgar, and Williams 2003):

- Heavy elliptical (major axis is 50.013 mm [1.969 in.]; minor axis is 33.985 mm [1.338 in.]; area is 1344.513 mm^2 [2.084 in^2]).
- Medium elliptical (major axis is 42.012 mm [1.654 in.]; minor axis is 38.321 mm [1.508 in.]; area is 950.321 mm^2 [1.473 in^2]).
- Standard round (diameter is 38 mm [1.5 in.] and area is 1139.998 mm^2 [1.767 in^2]).

Each type of dowel bar was placed at three different spacings across the transverse joints: 305 mm (12 in.), 380 mm (15 in.), and 460 mm (18 in.). Three replicate sections of each dowel size and spacing were placed in cut, fill, and transition roadway sections.

Twelve test sections were constructed where dowel bars were placed only in the wheelpaths. A set of four bars at 305-mm (12-in.) spacings was installed in each wheelpath. Six sections were constructed using elliptical medium dowels and the other six sections using conventional 38-mm (1.5-in.) diameter round dowels.

Each test section consisted of 20 transverse joints placed at 6-m (20-ft) intervals and separated from the next test section by a minimum of five joints containing 38-mm (1.5-in.) round steel dowels.

To measure the actual strain responses within the bars, six baskets in the outside lane were fitted with strain gauges. The basket configurations chosen for instrumentation were:

- Medium elliptical dowels at 305-mm (12-in.) spacing.
- Medium elliptical dowels at 460-mm (18-in.) spacing.
- Heavy elliptical dowels at 305-mm (12-in.) spacing.
- Heavy elliptical dowels at 460-mm (18-in.) spacing.
- Heavy elliptical dowels at 380-mm (15-in.) spacing.
- Standard 38-mm (1.5-in.) dowels at 305-mm (12-in.) spacing.

State Monitoring Activities

The research team is monitoring the performance of these test sections for 5 years. FWD tests, visual distress surveys, joint faulting and joint opening measurements, and longitudinal profile measurements are being conducted twice per year (spring and fall) (Cable, Edgar, and Williams 2003). In addition, strain gauge responses were monitored using loaded trucks 1 to 7 days after paving and will be monitored 1, 3, and 5 years after paving (Cable, Edgar, and Williams 2003).

Preliminary Results/Findings

The following conclusions can be drawn from the construction process (Cable, Edgar, and Williams 2003):

- The use of elliptical dowels had no effect on the handling or installation of standard, full-lane width, dowel baskets.
- The wheelpath-only baskets provide for more options in the pavement placement process, are easy to handle, and require less dowel materials. However, the wheelpath-only baskets require more time to align on the base than conventional baskets of full-lane width.
- Current construction practices make it somewhat difficult to install and protect the strain gauges during the various phases of construction.

A general concern about the use of elliptical dowel bars is their suitability for placement with a dowel bar inserter.

Points of Contact

Mark Dunn
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50011
(515) 239-1447

Max L. Porter
Iowa State University
Center for PCC Pavement Technology
2901 S Loop Drive, Suite 3100
Ames, IA 50010-8632
(515) 294-8103

References

Cable, J. K., L. Edgar, and J. Williams. 2003. *Field Evaluation of Elliptical Steel Dowel Performance—Construction Report.* Iowa State University, Ames.

Porter, M. L., and R. J. Guinn. 2002. *Assessment of Dowel Bar Research*. Project HR-1080 Final Report. Iowa Department of Transportation, Ames.

Porter, M. L., R. Guinn, and A. Lundy. 2001. *Dowel Bar Optimization: Phases I and II*. Final Report. American Highway Technology and Center for Portland Cement Concrete pavement Technology, Ames.

Chapter 14. IOWA 5 (Iowa 330, Melbourne)

Introduction

In 2002, the Iowa DOT, in conjunction with Iowa State University, constructed concrete pavement test sections containing elliptical fiber reinforced polymer (FRP) dowel bars. The test sections are located on Iowa 330, just west of Melbourne, Iowa (see Figure 40). The primary advantage of FRP dowels over conventional steel dowels is that they are not susceptible to corrosion. In addition, the benefits of an elliptical shape over circular are the reduction of bearing contact stress between the concrete and the bar. This project provides the opportunity to evaluate the performance of elliptical FRP dowels compared to both round and elliptical steel dowels installed on the same highway project.

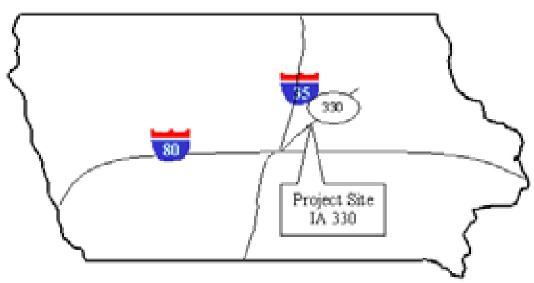


Figure 40. Location of IA 5 project.

Study Objectives

The goal of this project is the evaluation of the elliptical FRP dowels and elliptical dowel bar baskets to provide for transfer of load across the concrete pavement joints (Cable, Porter, and Guinn 2003).

Project Design and Layout

The elliptical FRP dowel bars used in this project are 457-mm (18-in.) long. The major and minor axes have dimensions of 57.15 and 49.28 mm (2.25 and 1.94 in.), respectively (Cable, Porter, and Guinn 2003). Dowel bars were placed in the northbound lanes at 30 joint locations, each 6 m (20 ft) apart. Three different dowel bar spacings (254, 305, and 381 mm [10, 12, and 15 in.]) were used. Figure 41 illustrates the layout of the dowel bars (Cable, Porter, and Guinn 2003).

The use of FRP material caused an inability to use the conventional method of welding steel dowel bars to the baskets. Therefore, the dowels were attached to the baskets using plastic ties and epoxy (Cable, Porter, and Guinn 2003). Special care was taken to make sure that, when dried, the epoxy was strong enough to hold the bars in position during the concrete placement, yet brittle enough to crack and allow the bars to move in the longitudinal direction after the concrete had set (Cable, Porter, and Guinn 2003).

To determine the stresses on the dowel bars at different spacings, one dowel bar from each of the 254-, 305-, and 381-mm (10-, 12-, and 15-in.) spacings was fitted with eight strain gauges. The strain gauge positions, as shown in Figure 42 (Cable, Porter, and Guinn 2003), were chosen to be the same as the strain gauges placed on the coinciding elliptical steel dowel bar project to facilitate a comparison between the two types of bars.

State Monitoring Activities

The research team is monitoring the performance of these test sections for a period of 5 years. Visual distress surveys, joint opening and joint faulting measurements, FWD tests, and profile measurements are being conducted twice a year (spring and fall). Load testing using a loaded Iowa DOT tandem axle truck was conducted in fall 2002 and winter 2003 and will be conducted in the future only as long as the wiring to the test bars continues to function.

Preliminary Results/Findings

The following conclusions were drawn from this project during the construction process:

- The elliptical FRP dowels can be placed on metal baskets and successfully placed in concrete pavements.
- Testing of the FRP dowel locations indicates no specific difference with the steel dowels on an adjacent project.

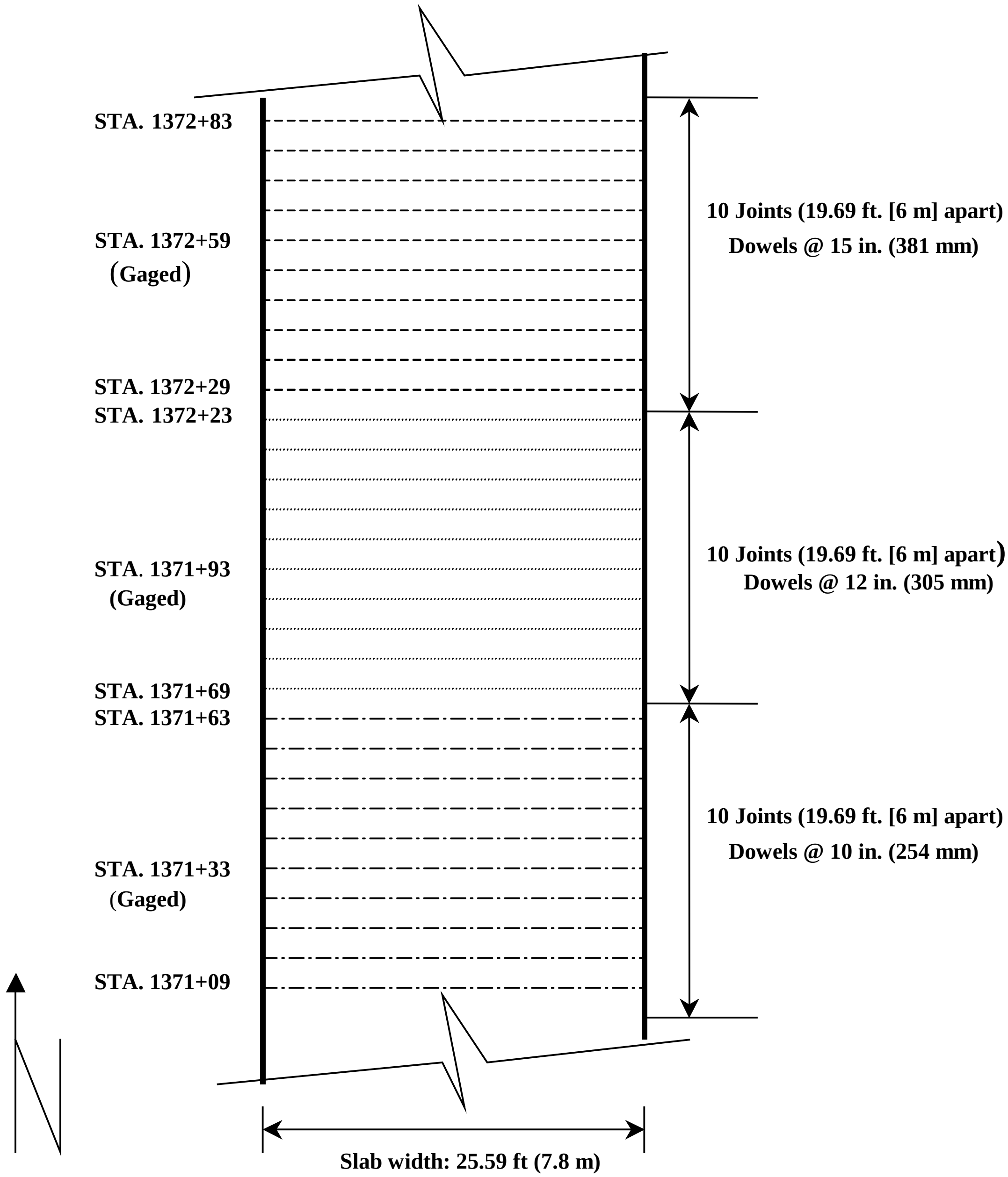


Figure 41. Layout of dowel bars on IA 5 project.

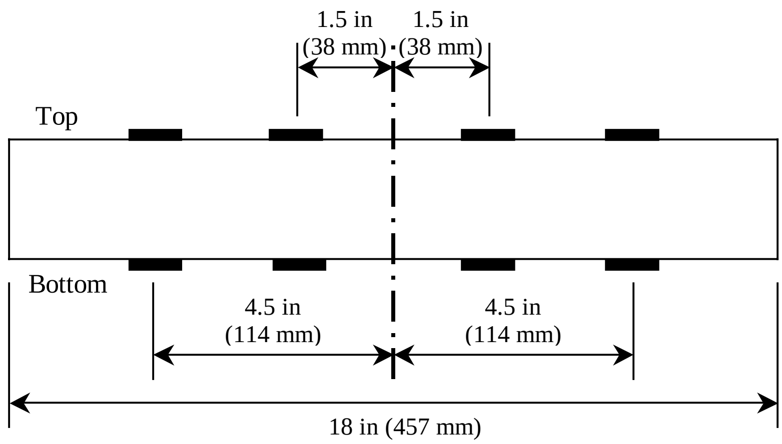


Figure 42. Location of strain gauges on elliptical FRP dowel bar.

Points of Contact

Mark Dunn
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50011
(515) 239-1447

Max L. Porter
Iowa State University
Center for PCC Pavement Technology
2901 S Loop Drive, Suite 3100
Ames, IA 50010-8632
(515) 294-8103

Reference

Cable, J. K., M. L. Porter, and R. J. Guinn. 2003. *Field Evaluation of Elliptical Fiber Reinforced Polymer Dowel Performance—Construction Report.* Iowa State University, Ames.

Chapter 15. IOWA 6 (Various Locations)

Introduction

Iowa State University, in cooperation with Iowa DOT and Iowa Fly Ash Affiliates (IFAA), started full-scale field tests in 2002 to develop and implement practical guidelines for soil stabilization with a wide range of fly ashes. This research will not only benefit the paving industry by emphasizing the importance of subgrade uniformity, but it will develop knowledge of successful design and application of soil stabilization with fly ash materials.

Study Objectives

The research has four primary objectives (White 2002):

1. Evaluate and document the influence of subgrade uniformity on pavement performance from studies of past and test sections selected in this study.

2. Determine from field test sections and laboratory analysis how various raw fly ashes, hydrated fly ashes (HFA), and conditioned fly ashes (CFA) in combination with the wide range of Iowa soil types will bring about uniform properties for subgrade support.

3. Recommend design and construction procedures and develop a Ash Stabilization Guide.

4. Create a technology transfer seminar to disseminate the knowledge.

Project Design and Layout

Several potential fly ash stabilization projects including sections of US highways 63 and 330 have been identified for this study. Field testing will be conducted to determine engineering properties of subgrade soils, such as classification, stiffness, strength, density, and moisture content before and after stabilization (White 2002). Laboratory testing will be conducted on various ash/soil mixtures to evaluate the effects of ash on compressive strength, volumetric stability, and soil index properties of subgrade soils (White 2002).

State Monitoring Activities

At the time of this report, the planned monitoring activities are yet to be determined.

Preliminary Results/Findings

No preliminary results or findings are available.

Points of Contact

Mark Dunn
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50011
(515) 239-1447

David J. White
Iowa State University
Center for PCC Pavement Technology
2901 S Loop Drive, Suite 3100
Ames, IA 50010-8632
(515) 294-8103

Reference

White, D. W. 2002. *Field Trials of Guidelines for Soil Stabilizers of Non-Uniform Subgrade Soils—Proposal*. Iowa State University, Ames.

Chapter 16. Iowa 7 and MD 2 (Implementation of TEMP)

Introduction

The most common uses of the Total Environmental Management for Paving (TEMP) system is to predict the critical time to open concrete pavements to traffic and to determine the appropriate time for joint sawing activities (Transtec 2002). Concrete strength development and pavement opening times are computed using maturity concepts in which the opening time can be predicted based on the past (known) and future (predicted) concrete temperatures. Maturity methods have been used effectively for a number of years. The key advantages of the TEMP system over the state-of-the-practice are automation and prediction. At a minimum, a TEMP System is comprised of concrete temperature gauges as well as a computer running the TEMP System software. An optional portable weather station can be included to further enhance the predictive ability of the software.

In 2003, a cooperative project was initiated to implement the TEMP system in the field. Three candidate implementation sites, two in Iowa and one in Maryland, have been identified for this project. The results from this project will help an agency or contractor better utilize concrete maturity data and improve the efficiency and quality of concrete paving.

Study Objectives

The objectives of this project include proof testing in the field as well as associated implementation activities of the TEMP system. These objectives can be categorized as follows (Turner and Ruiz 2003):

- Demonstrate an automated maturity data collection system that automatically collects and relays maturity data in the field.
- Demonstrate an automated maturity data management and reporting system that automatically catalogs maturity data in a fashion suitable for easy reporting and interpretation by the practitioner.
- Demonstrate a system that performs real-time predictions of future concrete strengths and time of opening to traffic.

Project Design and Layout

Up to three implementation sites are required for this project, which have been tentatively identified as (Turner and Ruiz 2003):

1. *US 151 in Jones County, Iowa*. This project is a four-lane relocation and new construction on a new alignment. The total length of the project is 9.93 km (6.17 mi) from North of Monticello, Iowa, to Cascade, Iowa. The pavement cross section is a 240-mm (9.5-in.) JPCP on a 260-mm (10.2-in.) granular base. The inside and outside lanes will be 3.6 m (11.8 ft) and 4.2 m (13.8 ft) wide, respectively. Perpendicular transverse joints will be doweled and spaced at 6-m (20-ft) intervals. The pavement will have granular shoulders.

2. *County roads, including 320th Street in Washington County, Iowa*. This project is a combination of four individual sections in Washington County. Two sections (6.50 and 2.75 km [4.04 and 1.71 mi] long) are 6.7-m (22-ft) wide JPCP on 102-mm (4-in.) of rubblized concrete. The new concrete layer varies from 254 mm (10 in.) at the edges to 178 mm (7 in.) at the centerline. The pavement will have skewed and doweled transverse joints at a spacing of 4.6 m (15 ft) and will have granular shoulders. A third, 4.36-km (2.71-mi) long section follows the same concrete surface thicknesses but will be constructed on an existing granular surfaced road. A fourth section will be approximately 12.9 km (8 mi) long and will follow the same design for the concrete surface layer, but will be constructed as whitetopping (on existing asphalt) or on granular material.

3. *Frederick, Maryland*. Details on this project are still being collected.

Temperature monitoring sensors (iButtons™) will be installed into fresh concrete during paving. IButtons™ will be installed at the mid-depth of the pavement and 0.3 to 0.9 m (1 to 3 ft) from the outside edge and transverse joint with intervals typically between 30.5 and 305 m (100 and 1000 ft) (Turner and Ruiz 2003).

State Monitoring Activities

Concrete temperature and strength as well as weather conditions (e.g., temperature, wind speed, and relative humidity) will be monitored using the TEMP System for 3 to 4 days following placement (Turner and Ruiz 2003).

Preliminary Results/Findings

No preliminary results or findings are available at this time.

Point of Contact

Mark Dunn
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50011
(515) 239-1447

References

The Transtec Group. 2002. *Technical Work Plan for Implementation of a Total Environmental Management for Paving (TEMP) System.* The Transtec Group, Inc., Austin, TX.

Turner, D. J., and J. M. Ruiz. 2003. *Final Work Plan for Implementation of a Total Environmental Management for Paving (TEMP) System*. The Transtec Group, Inc., Austin, TX.

Chapter 17. KANSAS 1 (Highway K-96, Haven)

Introduction

In 1997, the Kansas Department of Transportation (KDOT) constructed an experimental project under the TE-30 program. This project incorporates a wide variety of experimental features, from alternative dowel bars to alternative sawing practices to different mix designs (Wojakowski 1998). The project is located on Highway K-96 near Haven, which is located between Wichita and Hutchinson (see Figure 43).

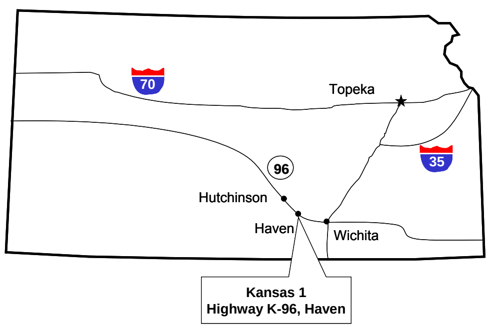


Figure 43. Location of KS 1 project.

Study Objectives

Looking for ways to improve PCC pavement performance, KDOT implemented this project to specifically assess the effects of different design features, mix designs, and construction practices on the constructibility and performance of PCC pavements. Benefits expected to be derived from the project include assessments of the following (Wojakowski 1998):

- Feasibility of including recycled waste materials in a two-lift PCC pavement.
- Feasibility of using a harder aggregate of unknown reactivity mitigated with an appropriate pozzolan in a two-lift PCC pavement.
- Performance of new load transfer devices.
- Life extensions associated with premium materials and concrete mixes.
- Cost-benefit data associated with the various designs.

Project Design and Layout

Two phases were defined in the work plan for this project. Phase I included evaluating the many different construction materials and mix designs to be used in the study, preparing the plans and specifications, doing a construction evaluation, and evaluating and monitoring the project (Wojakowski 1998). Phase II consisted of the actual construction of the test sections.

Constructed in the early fall of 1997, the test sections are located only in the two eastbound lanes of the four-lane divided highway (Wojakowski 1998). Most test sections are 1-km (0.6-mi) long. The soils in the area are typically silty and sandy loams, and the 20-year projected traffic volume is 8,000 vehicles per day, which includes 11 percent trucks (Wojakowski 1998).

The *nominal* pavement structural design for the test sections is a 254-mm (10-in.) JPCP over a cement-treated base (CTB). Transverse joints are spaced at 4.6-m (15-ft) intervals (except for one section), contain 32-mm (1.25-in.) diameter epoxy-coated steel dowels (except for two sections), and are sealed with neoprene compression seals (except for part of one section) (Wojakowski 1998). Longitudinal joints contain tie bars at 914-mm (36-in.) spacings and are sealed with a hot-poured, low-modulus sealant (Wojakowski 1998).

Thirteen experimental pavement sections were constructed on this project, incorporating a wide range of design and construction variables. A description of the features of each experimental section is provided below (Wojakowski 1998):

- *Section 1—Control section.* This pavement section reflects the nominal pavement structural design, containing 32-mm (1.25-in.) diameter dowel bars and placed in a single lift. The concrete mixture contains 337 kg/m^3 (564 lb/yd^3) of Type II cement, a water-cement ratio (w/c) of 0.47, and 6.5 percent air. Three aggregates—35 percent of a coarse limestone (19 mm [0.75 in.] top size), 15 percent of a pea gravel, and 50 percent of a coarse sand—were used and blended to produce a grading approximating a “Shilstone” haystack gradation curve. Consolidation of the concrete was specified to be 98 percent of the vibrated unit weight when measured by a nuclear density meter.

- *Section 2—Single saw cut.* The first 31 transverse joints of this section were created using a narrow (6 mm [0.25 in.]) single saw cut and left unsealed, whereas the remaining 79 joints were widened and sealed with a hot-poured material. Other than the transverse joint sealant, other aspects of this pavement are the same as the nominal pavement structural design.

- *Section 3—Nontraditional dowel type.* With the same nominal structural design as the control, this section incorporates 51-mm (2-in.) diameter FiberCon™ fiberglass dowels, manufactured by Concrete Systems, Inc. These dowels are a composite fiberglass tube filled with a high-strength cement grout. One-half of the length of the bar is machined to a smooth surface of 48 mm (1.9 in.) diameter to allow slippage of the bar as the joint opens and closes. The bars are placed on center-to-center spacings of 305 mm (12 in.).

- *Section 4—X-Flex™ load transfer device.* This short section, consisting only of five joints, incorporates a unique load transfer device called the X-Flex™, developed at Kansas State University. As shown in Figure 44, the configuration of the X-Flex™ is such that the “X” part of the device goes across the transverse joint with the far ends curving to make a loop in a continuous design; wheel loads are transferred through tension in the “X” part of the device rather than by shear. The device is made from 13 mm (0.5 in.) epoxy-coated steel cast bars and spaced at 305-mm (12-in.) centers across the joint. Other design aspects of this section are the same as the nominal pavement design.

- *Section 4a, 5, and 6—Alternate saws.* Three sections were constructed that used different lightweight saws for the establishment of the transverse joints. Section 4a used a Soff-Cut™ saw (to depths of 38 and 64 mm [1.5 and 2.5 in.]), section 5 used a Target saw (to depths of 25, 44, and 64 mm [1, 1.75, and 2.5 in.]), and section 6 used a Magnum saw (to depths of 25, 44, and 64 mm [1, 1.75, and 2.5 in.]). The rest of the design features of this section are the same as the nominal structural design.

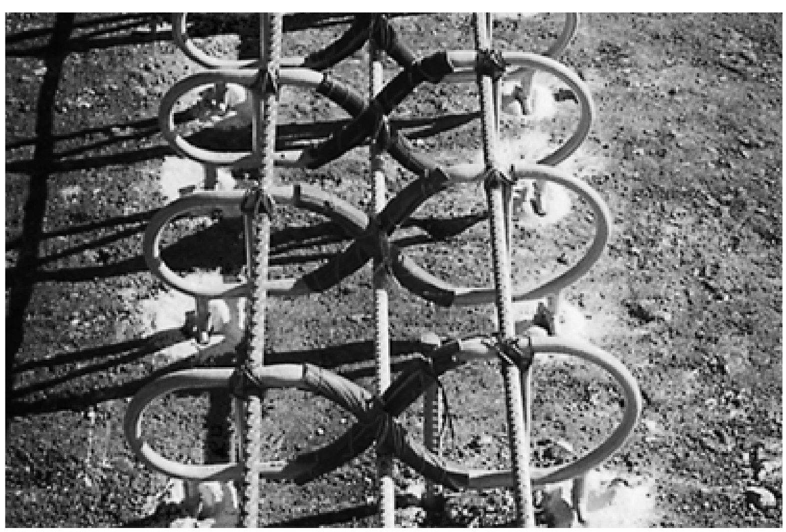


Figure 44. Photo of X-Flex™ load transfer devices.

- *Section 6a—Polyolefin fiber PCC.* This short (152-m [500-ft]) section incorporates polyolefin fibers in the PCC mix design. The transverse joint spacing was extended to 18.3 m (60 ft), and the longitudinal lane-lane joint was eliminated. The fibers are 1.57 mm (0.062 in.) in diameter and were added at the rate of 15 kg/m^3 (25 lb/yd^3). The w/c of the mix was increased from 0.45 to 0.49 to provide additional workability.

- *Section 7—Longitudinal tining*. Instead of conventional transverse tining, this section incorporates longitudinal tining impressed on the surface of the fresh concrete surface. The primary purpose of longitudinal tining is to reduce noise levels produced by passing vehicles. All other design aspects of this section are the same as the nominal structural pavement design.

- *Section 8—Special curing compound*. A special high solids curing compound, conforming to ASTM C 1315, was applied to this section. The purpose is to compare the effectiveness of the special curing compound to the conventional curing compound in terms of surface integrity and compressive strength. The curing compound was applied at the rate of 0.036 L/m^2 (0.03 gal/yd^2), which is about half the rate recommended for a rough surface. All other design elements for this section are the same as the nominal pavement design.

- *Section 9—Two-lift construction with recycled asphalt pavement (RAP).* This section used a two-lift construction process in which the bottom 178 mm (7 in.) of pavement used 15 percent recycled asphalt pavement in place of the intermediate-sized well gravel. Laboratory testing indicated that this mixture could produce 28-day strengths of 27.6 MPa (4,000 lbf/in^2). The top lift was placed 76-mm (3-in.) thick using the standard PCC mixture. All other design aspects for this section are the same as the nominal pavement design.

- *Section 10—Lower w/c Concrete.* This section employed a high-range water reducer to lower the w/c of the mixture by 0.05 from the standard w/c of 0.47. The rest of the pavement design is the same as the nominal structural pavement design.

- *Section 11—Two-Lift Construction with Igneous Rock*. This section used a two-lift construction process in which the top 76 mm (3 in.) incorporates rhyolite, a hard igneous rock. The objective of this design is to evaluate the resistance of the pavement to polishing, to which the limestone aggregates commonly used by KDOT are susceptible. Because the rhyolite is potentially susceptible to alkali-silica reactivity (ASR), a calcined natural clay pozzolan called DuraPoz™ was used to replace 20 percent of the cement (by weight) in the 76-mm (3-in.) surface layer. The bottom 178 mm (7 in.) of this pavement used a soft, high-absorption durable limestone. A 25-mm (1-in.) maximum size that passed the State's freeze-thaw durability test (ASTM C 666, Procedure B) was used.

- *Section 12—Two-Lift Construction with Lower w/c Concrete*. This two-lift construction section used the same lower w/c PCC mixture used in section 10 but only in the top 76 mm (3 in.) layer. The bottom 178 mm (7 in.) layer used the same 25-mm (1-in.) soft high absorptive limestone used in section 11. The remaining design aspects of this section are the same as the nominal pavement design.

- *Section 13—Random Transverse Tining and Special Curing Compound*. This section was constructed in the spring of 1998 and used a special tining rake that imparted variably spaced transverse impressions on the surface of the fresh concrete to reduce tire-surface noise. Additionally, the remainder of the high-solids special curing compound used on section 8 was applied at the rate of 0.18 L/m^2 (0.04 gal/yd^2). All other design elements of the section are the same as the nominal pavement design.

Table 17 provides the simplified experimental design matrix for the sections included in this project. The layout of the pavement test sections is illustrated in Figure 45.

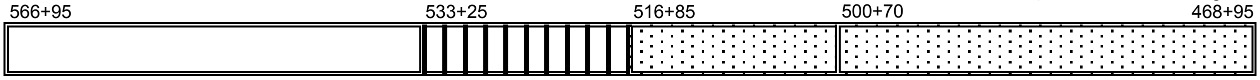
Table 17. Simplified Experimental Design Matrix for KS 1

			CONVENTIONAL PCC MIX WITH SINGLE-LIFT CONSTRUCTION						TWO-LIFT CON-STRUCTION (RECYCLED ASPHALT ON BOTTOM)	TWO-LIFT CONSTRUCTION (IGNEOUS ROCK ON TOP)	TWO-LIFT CONSTRUCTION (LOW W/C ON TOP)	PCC MIX WITH POLYOLEFIN FIBERS
			Conventional Sawcutting Equipment			Lightweight Sawcutting Equipment	Conventional Sawcutting Equipment		Conventional Sawcutting Equipment			Conventional Sawcutting Equipment
			Steel Dowels	FiberCon Dowels	X-Flex Device	Steel Dowels	Steel Dowels		Steel Dowels			Steel Dowels
			Compression Seals			Compression Seals	Hot-Pour Sealant	No Sealant	Compression Seals			Compression Seals
Conventional w/c	**Conventional Curing Compound**	**Conventional Transverse Tining**	Section 1	Section 3	Section 4	Section 4a Section 5 Section 6	Section 2	Section 2 (31 jts)	Section 9	Section 11	Section 12	Section 6a
		Longitudinal Tining	Section 7									
		Random Transverse Tining										
	High Solids Curing Compound	**Conventional Transverse Tining**	Section 8									
		Longitudinal Tining										
		Random Transverse Tining	Section 13									
Lower w/c	**Conventional Curing Compound**	**Conventional Transverse Tining**	Section 10									

Kansas High Performance Rigid Pavement Test Sections

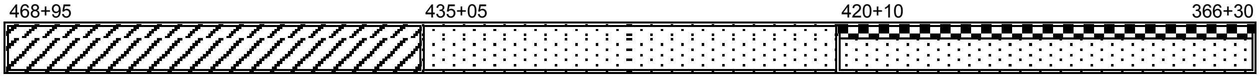
96-78 K 4457-01, Reno County

			Soft Cut Target Magnum
566+95	533+25	516+85	500+70 468+95



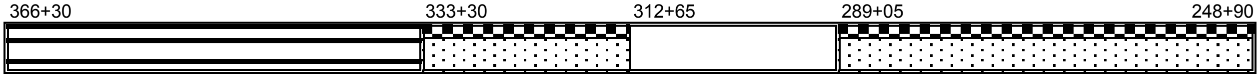
1. Control Section	**2. Single Saw Cut**	**3. Non-Traditional Dowel Type**	**4, 5, and 6. X-Flex, Alternate Saws, and 3M Fibers**
1 km (3,280 lf)	1/2 km (1,640 lf)	1/2 km (1,640 lf)	1 km (3,280 lf) ***(5 X-Flex joints)***
Typical construction	***Single saw cut, no joint sealant*** ***0.14 km open, 0.36 km ss***	***FiberCon dowels***	***Initial control cuts utilizing:*** ***Soff-Cut, Magnum, and Target Early-cut Saws*** ***3M polyolefin fibers (500 ft section only)***
1 lift placement	1 lift construction	1 lift construction	1 lift construction
"Shilstone" design	"Shilstone" design	"Shilstone" design	"Shilstone" design
564 lb cement/cy	564 lb cement/cy	564 lb cement/cy	564 lb cement/cy
6.5% air	6.5% air	6.5% air	6.5% air
98% consolidation	98% consolidation	98% consolidation	98% consolidation
Typical epoxy coated steel dowels (1.25" dia.)	Typical dowels	Typical dowels	Typical dowels, outside of X-flex
0.47 w/c used through section	0.47 w/c ratio 1st day 0.45 w/c ratio 2nd day	0.45 w/c ratio all day	0.47 w/c ratio for fibers 0.45 w/c ratio for all other

468+95	435+05	420+10 366+30



7. Special Pavement Marking/Longitudinal Tining	**8. ASTM C1315 Cure**	**9. Two-Lift, Recycled Asphalt on Bottom**
1 km (3,280 lf)	1 km (3,280 lf)	1 km (3,280 lf)
Longitudinal tining ***Proprietary pavement marking product***	***Special high-solids curing compound***	***15% RAP usage in bottom 7" layer*** ***Typical concrete mix in top 3"***
1 lift construction	1 lift construction	2 lift construction
"Shilstone" design	"Shilstone" design	
564 lb cement/cy	564 lb cement/cy	564 lb cement/cy
6.5% air	6.5% air	6.5% air in each lift
98% consolidation	98% consolidation	98% consolidation
Typical dowels	Typical dowels	Typical dowels
0.45 w/c ratio through section		Single consolidation 3 oz/100 lb Masterpave N in RAP mix 0.45 w/c ratio in RAP mix 0.45 w/c ratio in normal mix

366+30	333+30	312+65	289+05 248+90



10. Lower w/c Ratio	**11. Two-Lift, Igneous Rock on Top**	**12. Two-Lift, Low w/c on Top**
1 km (3,280 lf)	0.63 km (2,065 lf)	1 km (3,280 lf)
High range water reducer to lower w/c 0.05 from standard mix	***High absorption limestone in bottom 7"*** ***Rhyolite aggregate in top 3"*** ***Dura-Poz to counter ASR of Rhyolite***	***High absorption limestone in bottom 7"*** ***High range water reducer in top 3" to lower w/c 0.05 from standard mix***
1 lift construction	2 lift construction	2 lift construction
"Shilstone" design	"Shilstone" design	"Shilstone" design
564 lb cement/cy	564 lb cement/cy	546 lb cement/cy
6.5% air	6.5% air in each lift	6.5% air in each lift
98% consolidation	98% consolidation	98% consolidation
Typical dowels	Typical dowels	Typical dowels
	Single consolidation	Single consolidation

13. Random Pattern Transverse Tining and High Solids Cure   Sta. 966+44 to 982+15 and 1006+00 to 1026+40
1.1 km (3611f)   All else typical construction

Figure 45. Layout of KS 1 project (Wojakowski 1998).

State Monitoring Activities

Construction Monitoring

During the construction of the test sections, several incidents of note were observed by the researchers. These are summarized below (Wojakowski 1998):

- *Section 4*. Because they were placed only about 38 mm (1.5 in.) below the surface, several of the X-Flex™ load transfer devices were struck and dislodged by the paver during construction. Consequently, some of these devices had to be replaced by dowels and the transverse joint hand finished.

- *Section 5*. Sawing with the Soff-Cut™ lightweight saw began about 3 hours and 20 minutes after the concrete was placed. For the 40 joints cut to a depth of 38 mm (1 in.), 6 cracks occurred beneath the saw cut after 1 day, 18 cracks occurred beneath the saw cut after 2 days, 23 cracks after 3 days, 31 cracks after 4 days, 35 cracks after 5 days, and 39 cracks after 7 days. No cracks were observed within the panels for this section or for any of the sections that employed the lightweight saws.*Section 6*. Intermediate cracks occurred almost immediately after this fiber-reinforced PCC pavement was placed. During the first night, intermediate cracks formed on 5 of the 8 panels that were paved, and by the 5th day after placement, 10 cracks had appeared in the 8 panels. The panels containing two cracks were noted to be the first two placed on the day that had a high temperature of 35 ^{o}C (95 ^{o}F) and a low of 21 ^{o}C (69 ^{o}F).

- *Section 8*. The special curing compound was applied with the expectation that improved curing would lead to a more durable pavement. Unfortunately, the environmental conditions at the time of application were quite mild, and the actual coverage rate (0.035 L/m^2 [0.03 gal/yd^2]) was one-half the recommended value.

- *Section 9*. Limited funding prevented the use of a separate paver for the lower lift. Therefore, a belt placer/spreader was used for the placement of the lower lift, the mix design of which had to be stiffened to accommodate its placement. An interval of about 30 minutes was needed between the placement of the two lifts. Small depressions were noted when the top lift was dropped on the bottom lift and some intermixing of the two lifts occurred, but for the most part the process was workable and adequately controlled. This same construction procedure was essentially followed for all two-lift sections.

- *Section 10*. In spite of the lower w/c used in the mix design for this section, it was found to be workable and easy to finish. This workability is attributed to the lubrication provided by the high-range water reducer used to reduce the w/c.

- *Section 13*. The weather at the time that this section was constructed in 1998 mandated the use of cold-weather concreting procedures. The special curing compound was applied at the specified rate of 0.18 L/m^2 (0.04 gal/yd^2).

Early Mix Property Data

During the construction of the different sections, several key mix design properties were measured. These measurements are summarized in Table 18. Generally speaking, most of the results appear reasonable for the various mix designs.

Performance Monitoring

KDOT has been monitoring the performance of these test sections since 1998. The following information is collected: distress data (cracking, faulting, spalling), joint load transfer efficiency, noise, and surface friction (Wojakowski 1998).

Preliminary Results/Findings

Some preliminary results are available from KDOT regarding the costs and performance of these sections. These results are described in the following sections.

Cost Data

Construction costs for these sections are summarized in Table 19 and illustrated in Figure 46. Some notes regarding these costs are documented by Wojakowski (1998):

- *Section 1.* This figure includes the costs of both the mainline pavement and the shoulders.
- *Section 3.* For the small quantities of this project, the costs of the Fiber-Con™ dowels were $28.87/m ($8.80/ft) as compared to $8.00/m ($2.44/ft) for conventional epoxy-coated steel dowels. With a larger project and volume pricing, the cost of these dowels could be expected to be around $16.40/m ($5.00/ft).
- *Sections 4a, 5, and 6.* The lightweight saws provided minimal savings to the overall construction costs.
- *Sections 9, 11, and 12.* These sections included a two-lift construction process that added considerable cost. The two-lift construction costs included a second batch plant, extra hauling of material, a concrete belt placer/ spreader, and extra labor for hauling.

Table 18. Summary of Early Mix Properties of KS 1 Test Sections (Wojakowski 1998)

SECTION	AVERAGE SLUMP, IN.	AVERAGE AIR CONTENT, %	AVERAGE UNIT WEIGHT, LB/FT^3	AVERAGE FLEXURAL STRENGTH, LBF/IN^2	CORE COMPRESSIVE STRENGTH, LBF/IN^2 (28-DAY)
1—Control	0.6	6.0	142.0	525 (7 days)	4,583
2—Single sawcut	1.0	7.1	142.2	N/A	N/A
3—Nontraditional dowel	1.0	5.3	142.6	N/A	N/A
4—X-Flex™	N/A	N/A	N/A	N/A	N/A
4a—Lightweight Soff-cut saw	N/A	N/A	N/A	N/A	4,530
5—Lightweight Target saw	1.0	7.0	141.0	N/A	N/A
6—Lightweight Magnum saw	N/A	N/A	N/A	N/A	N/A
6a—Polyolefin fibers	0.0	5.2	141.0	539 (7 days)	4,598
7—Longitudinal tining	0.8	6.9	141.2	N/A	4,665
8—Special curing compound	2.1	7.4	140.0	N/A	4,760
9—Two-lift with RAP	0.8 (top lift)	6.8	142.0	N/A	3,843
	0.8 (bottom lift)	5.8	142.2	517 (5 days)	
10—Lower w/c	1.1	5.8	144.7	550 (4 days)	5,040
11—Two-lift with igneous rock	2.0 (top lift)	4.3	142.2	583 (8 days)	4,780
	0.8 (bottom lift)	5.9	139.43	475 (4 days)	
12—Two-lift with lower w/c	1.0 (top lift)	5.8	143.8	583 (6 days)	N/A
	1.2 (bottom lift)	5.3	137.82	583 (6 days)	
13—Random tining	N/A	N/A	N/A	N/A	N/A

N/A = not available

Flexural strengths measured under third-point loading.

Table 19. Summary of Construction Cost Data for KS 1 (Wojakowski 1998)

SECTION	EXPERIMENTAL FEATURE	COST DIFFERENCE FROM CONTROL, $/YD^2	TOTAL UNIT COST, $/YD^2
1	Control	—	25.80
2	Unsealed joints	–0.67	25.13
3	FiberCon™ dowels	+5.72	31.52
4	X-Flex™ device	N/A	N/A
4a	Soff-Cut lightweight saw	0	25.80
5	Target lightweight saw	0	25.80
6	Magnum lightweight saw	0	25.80
6a	Polyolefin fibers	+15.63	41.43
7	Longitudinal tining	0	25.80
8	Special curing compound	+0.83	26.63
9	Two-lift construction with RAP	+25.12	50.92
10	Lower w/c	+0.03	25.83
11	Two-lift construction with igneous rock	+26.05	51.85
12	Two-lift construction with lower w/c	+25.09	50.89
13	Random transverse tining	0	25.80

N/A = Not available (experimental device not commercially available)

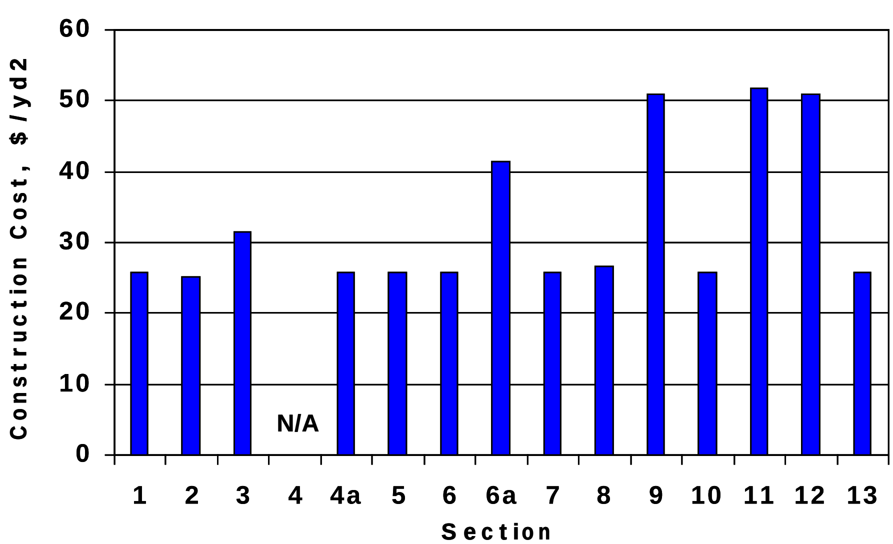


Figure 46. Relative construction costs by section for KS 1.

Early Performance Data

KDOT is monitoring the performance of these test sections, and has produced two annual reports documenting their early performance (KDOT 1998; KDOT 1999). Visual distress surveys are conducted in which cracking, joint faulting, and joint spalling are recorded. The results from these surveys are illustrated in Table 20. An examination of this table shows that although a few cracks have occurred in a few of the sections, overall these sections are in good condition. Faulting levels are all less than 1 mm (0.04 in.), far below critical faulting thresholds of 2.5 to 3.0 mm (0.10 to 0.12 in.). Joint spalling is observed only on two sections.

The initial profile index values for the test sections are shown in Table 20 and plotted in Figure 47. These values are based on a zero-blanking band. Followup roughness measurements are not available.

Table 20. Summary of Early Performance Data for KS 1 Project (KDOT 1998; KDOT 1999)

SECTION	EXPERIMENTAL FEATURE	1998 INITIAL PROFILE INDEX, MM/KM	AVERAGE JOINT FAULTING, MM		AVERAGE SPALLING, MM/JOINT		OTHER NOTED DISTRESSES
			1998	1999	1998	1999	
1	Control	240	0.02	0.02	4.90	15.24	
2	Unsealed joints		0.00	0.21			1 corner crack
	Sealed joints		0.07	0.10			2 corner cracks
3	FiberCon™ dowels	268	0.08	0.25			1.0-m transverse crack 8 corner cracks
4	X-Flex™ device	259	0.00	0.00			
4a	Soff-Cut lightweight saw		0.00	0.07			
5	Target lightweight saw		0.00	0.07			
6	Magnum lightweight saw						
6a	Polyolefin fibers		0.06	0.16			16 mid-panel cracks*; 1 longitudinal crack
7	Longitudinal tining	189					
8	Special curing compound	129	0.02	0.0	10.6	16.9	
9	Two-lift construction with RAP	178	0.05	0.13			
10	Lower w/c	178	0.13	0.10			
11	Two-lift construction with igneous rock	166	0.02	0.02			
12	Two-lift construction with lower w/c	186	0.13	0.17			3.7-m trans. crack
13	Random transverse tining	319		0.03			

***** These cracks in section 6a were retrofitted with dowel bars in 1999.

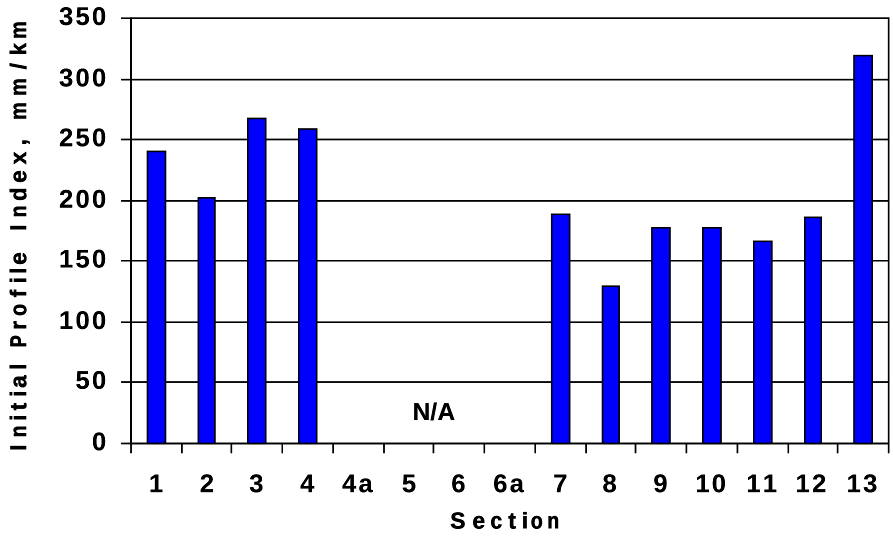


Figure 47. Comparison of Initial Profile Index of KS 1 test sections (Zero-Blanking Band).

A noise study was conducted by KDOT in October 1998 to assess the effect of the various tining methods (sections 1, 7, and 13) (KDOT 1998; KDOT 1999). Noise sampling was recorded from a passenger car and a medium-duty dump truck at three locations adjacent to the roadway. The study concluded that there were no significant differences in either the exterior or interior noise levels generated by vehicles traveling over these test sections (KDOT 1998; KDOT 1999).

Interim Results/Findings

The interim performance of these test sections is documented in the 2002 Annual Report (KDOT 2002) and the 2003 Annual Report (KDOT 2003) and summarized in Tables 21 and 22. FWD testing was performed in January 2002 and again in January 2004, and the results are shown in Figure 48. Based upon the FWD results, some of the load transfer devices are exhibiting marginal efficiency, particularly for a pavement less than 5 years old.

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Table 21. Summary of 2002 Performance Data for KS 1 Project (KDOT 2002)

SECTION	EXPERIMENTAL FEATURE	AVERAGE JOINT FAULTING, MM	AVERAGE SPALLING MM/JOINT	OTHER NOTED DISTRESSES
		2002	2002	
1	Control	0.02	65.2	
2	Unsealed joints	0.02	27.9	1 corner crack
	Sealed joints	0.07	38.9	6 corner cracks
3	FiberCon™ dowels	0.16	32.2	7 corner cracks; 1 1.2-m longitudinal crack
4	X-Flex™ device	0.00	162.6	
4a	Soff-Cut lightweight saw	0.02	143.1	
5	Target lightweight saw	0.07	10.9	
6	Magnum lightweight saw	0.00	50.8	
6a	Polyolefin fibers	0.23	53.3	3 new longitudinal cracks
7	Longitudinal tining	0.07	9.3	
8	Special curing compound	0.07	27.9	
9	Two-lift construction with RAP	0.05	17.8	16 transverse cracks averaging 1.3 m each
10	Lower w/c	0.15	10.2	
11	Two-lift construction with igneous rock	0.07	27.1	395 transverse cracks averaging 0.4 m each; 16 longitudinal cracks averaging 0.4 mm each
12	Two-lift construction with lower w/c	0.12	12.7	5 longitudinal cracks from 4.6 to 9.14 m each; 1 3.7 m transverse crack
13	Random transverse tining	0.05	22.9	2 corner cracks; 1 0.3 m longitudinal crack

Table 22. Summary of 2003 Performance Data for KS 1 Project (KDOT 2003)

SECTION	EXPERIMENTAL FEATURE	AVERAGE JOINT FAULTING, MM 2003	AVERAGE SPALLING MM/JOINT 2003	OTHER NOTED DISTRESSES
1	Control	0.25	41.5	
2	Unsealed joints	0.08	59.0	1 corner crack
	Sealed joints	0.07	50.0	6 corner cracks
3	FiberCon™ dowels	0.22	38.1	10 corner cracks; 1 1.2-m longitudinal crack
4	X-Flex™ device	0.40	106.7	
4a	Soff-Cut lightweight saw	0.07	85.4	
5	Target lightweight saw	0.04	21.8	
6	Magnum lightweight saw	0.00	44.5	
6a	Polyolefin fibers	0.19	25.4	
7	Longitudinal tining	0.07	9.3	
8	Special curing compound	0.02	42.3	
9	Two-lift construction with RAP	0.00	11.0	16 transverse cracks averaging 1.3 m each; light map cracking on 21 of 30 panels
10	Lower w/c	0.12	23.7	
11	Two-lift construction with igneous rock	0.00	21.2	17 longitudinal cracks averaging 2.6 m each; many smaller cracks not recorded
12	Two-lift construction with lower w/c	0.08	24.6	5 longitudinal cracks from 4.6 to 9.14 m each; 1 3.7-m transverse crack; 1 0.3-m transverse crack
13	Random transverse tining	0.07	27.9	2 corner cracks; 1 3.6-m longitudinal crack

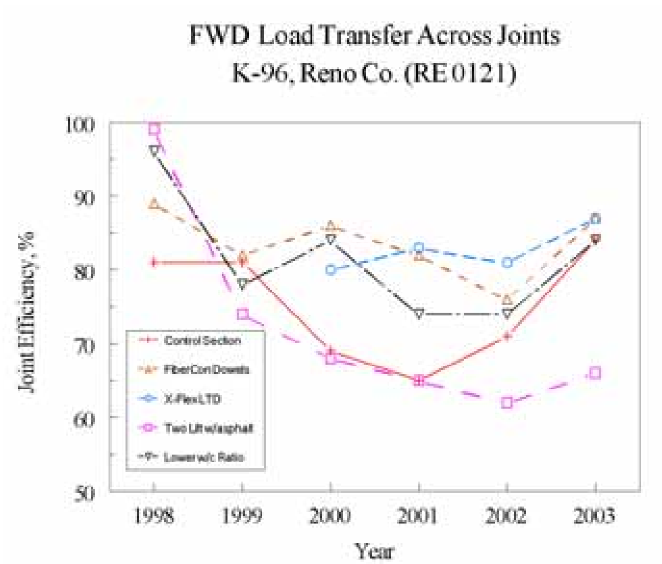

Figure 48. Joint load transfer efficiency on KS 1.

Chapter 18. KANSAS 2 (Hutchinson)

Introduction

The Kansas Department of Transportation, a national leader on pavement smoothness, has undertaken a project to build a PCC pavement that is "super smooth" and maintains that smoothness for an extended time. This project consists of five special sections, is located in Hutchinson, Kansas, and has a construction start date of April 2001. The project is multifaceted. The initial smoothness will be monitored and controlled with new equipment innovations, while the post-construction smoothness will be evaluated for different construction conditions (weather), mixture properties (w/c ratio), and joint spacings.

Study Objectives

To build a super-smooth pavement that maintains most of its original smoothness through increased initial smoothness levels as well as through design and mixture improvements.

Project Design and Layout

The pavement design consists of a 220-mm (8.7-in.) slab with 4.6-m (15-ft) joint spacing, a dense cement treated base, 32-mm (1.25-in.) diameter dowels spaced at 305 mm (12 in.), and a tied concrete shoulder. The transverse and longitudinal joints are sealed with a preformed compression seal and a low-modulus hot-poured material, respectively.

A definite correlation has been shown between monitoring of the smoothness behind the paver and the head of concrete coming into it. A special strike-off bar is being developed to maintain a uniform head of concrete to help alleviate one significant source of roughness. Concrete mixtures have been improved by newer specifications in Kansas, but there still appears to be room for making the mixture more workable without being susceptible to segregation. This will be done by increasing the coarse aggregate content to the maximum possible, while maintaining a non-segregating grading without gaps. Consistency of the mixture and the paving operation along with ambient temperatures must also be monitored for any effect on smoothness. The smoothness of the paving base should also be checked.

During the post construction period, the temperature gradient in the slab will be measured for morning and afternoon placements. Daily temperature ranges (and solar radiation) can also be monitored for their effect on smoothness. Additionally, joint spacing can be varied somewhat, from a standard spacing of 4.6-m (15 ft) to as short as 3.6 m (12 ft) and as much as 5.2 m (17 ft) for each of the three mixtures proposed for the project.

State Monitoring Activities

No information is currently available regarding the State monitoring efforts.

Preliminary Results/Findings

Preliminary results are not available at this time.

Point of Contact

Lon Ingram
Chief of Materials and Research
Kansas Department of Transportation
Bureau of Materials and Research
2300 Van Buren
Topeka, KS 66611-1195
(785) 296-2231

References

None.

Chapter 19. MARYLAND 1 (US 50, Salisbury Bypass)

Introduction

Maryland's TE-30 project was constructed in 2001. It is located on the Salisbury Bypass on US 50, in the southeastern part of the State (see Figure 49). The project is being undertaken jointly by the Maryland State Highway Administration (MSHA), the University of Maryland, and the FHWA.

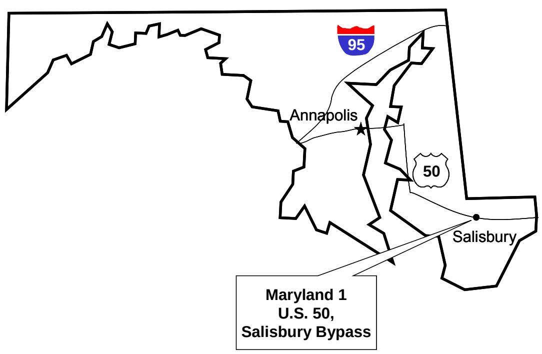

Figure 49. Location of MD 1 project.

Study Objectives

This project is designed to research the benefits of using fiber-reinforced concrete (FRC) and low-shrinkage concrete in Maryland paving operations (Goulias and Schwartz 1999). The researchers believe that the use of these materials may lead to increases in flexural fatigue resistance and reductions in crack development and slab warping effects (Goulias and Schwartz 1999). The objectives of the study will be completed in three phases:

- *Phase I*—Laboratory examination of the design and performance of fiber-reinforced concrete and low-shrinkage concrete for Maryland conditions.
- *Phase II*—Actual construction of the test sections.
- *Phase III*—Short- and long-term monitoring of the test sections, and quantification of benefits for possible incorporation into PCC pavement design.

Project Design and Layout

Laboratory tests were conducted to evaluate the properties of fiber-reinforced and low-shrinkage concrete mixtures. Maryland's standard concrete mixture for highway pavements, MD Mix 7, was used as the control mixture. The low shrinkage mixtures were developed by using the MD #375 aggregate in place of the MD #57 aggregate, or by modifying the w/c ratio. The fiber-reinforced mixtures used the mix design of the control mixture by adding different fiber contents (0.1, 0.2, 0.3, and 0.4 percent). Seven mixtures were evaluated and their characteristics are summarized in Table 23 (Goulias and Schwartz 2003). The characteristics of the fiber used in this study are shown in Table 24 (Goulias and Schwartz 2003). The mixture properties evaluated in the lab included compressive strength, unrestrained shrinkage of hardened concrete, restrained shrinkage of plastic concrete, flexural strength and toughness, fatigue endurance, elastic modulus, temperature, slump, and air content.

Three different concrete pavement test sections were constructed by MSHA in 2001 (Goulias and Schwartz 2003):

- Control section using MD Mix 7.
- Fiber-reinforced concrete section containing 0.1 percent polypropylene fiber.
- Low-shrinkage concrete section using MD #375 aggregate.

The pavement sections were extensively instrumented to provide feedback on the structural responses of the different designs.

Table 23. Mixture Properties of Concrete Used on MD 1

COMPONENT	MIX A	MIX B	MIX C	MIX D	MIX E	MIX F	MIX G
Aggregate type	#57	#57	#57	#57	#57	#375 LS	#57 LS
w/c ratio	0.44	0.44	0.44	0.44	0.44	0.44	0.40
Air content* (%)	6.6%	4.6%	6.6%	7.0%	5.8%	5.0%	6.0%
Slump (in/sec)	1.5	1.5/17	1.125/15	1/21	0.625/29	1	1.5
Fiber content	0.0%	0.1%	0.2%	0.3%	0.4%	0.0%	0.0%
Air entrainment	1.7 oz/ 100 lbs.	1.9 oz/ 100 lbs	1.9 oz/ 100 lbs	1.9 oz/ 100 lbs	1.9 oz/ 100 lbs	2.0 oz/ 100 lbs	2.0 oz/ 100 lbs
Water reducer	(M)5 oz/ 100 lbs	(M)5 oz/ 100 lbs	(M)5 oz/ 100 lbs	(M)5.5 oz/ 100 lbs	(M)6 oz/ 100 lbs	(M)5.5 oz/ 100 lbs	(H)2.7 oz/ 100 lbs

Note: (M) = middle range water reducing admixture; (H) = high range water reducing admixture; LS = low shrinkage;

* Target air content 6.5%

Table 24. Fiber Characteristics for MD 2

FIBER	LENGTH MM (IN.)	DIAMETER MM (IN.)	ASPECT RATIO (L/D)	YIELD STRENGTH MPA (KSI)	ELASTIC MODULUS MPA (KSI)	SPECIFIC GRAVITY
Polypropylene (Fibrillated)	19 (0.75)	N/A	N/A	550–750 (80–110)	3450 (500)	0.91

State Monitoring Activities

A variety of information has been collected on the three experimental sections (Goulias and Schwartz 2003). Two MSHA trucks (with 18 kip and 32 kip axles) were used for field load testing. Condition surveys/profile measurements were conducted using a dipstick device. Nondestructive testing was also conducted using an ultrasound device.

Interim Results/Findings

Analyses were carried out utilizing both the lab results and field instrumentation data to evaluate the response and behavior of the control, fiber reinforced and low shrinkage concrete pavement sections and mixtures. The following conclusions have been reached (Goulias and Schwartz 2003):

- The lab results indicated that fibers reduce the workability of concrete. However, the use of admixtures permits acceptable levels of workability.
- The flexural strength of concrete for fiber contents > 0.1 percent was higher than that of the control concrete mixture. The toughness of concrete increased with increasing fiber content.
- Shrinkage testing indicated that there were small differences in unrestrained shrinkage for the control and the two low shrinkage mixtures. However, fiber-reinforced concrete mixtures exhibited higher levels of shrinkage.
- The fatigue analysis indicated that the addition of polypropylene fibers resulted in higher fatigue strengths. The fatigue strength of FRC increased with decreasing fiber content until 0.3 percent. The endurance limit expressed as a percentage of the

modulus of rupture of the mixture showed an increase with decreasing fiber content until 0.3 percent. Overall, the best fatigue performance was obtained for mixes containing 0.1 percent fiber.

- The field data indicated that overall the sections with the 0.1 percent fiber-reinforced concrete mixture had lower deflections than the control mix and the low shrinkage mixture. Analytical evaluations indicated that the best estimates of the modulus of subgrade reaction (k) and elastic modulus of concrete (Ec) are k of 350 lbf/in^2/in. and 5,000,000 lbf/in^2, respectively.

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Points of Contact

Dimitrios Goulias
University of Maryland
Department of Civil and
Environmental Engineering
0147A Engineering Lab Building
College Park, MD 20742
(301) 405-2624

Charles Schwartz
University of Maryland
Department of Civil and
Environmental Engineering
College Park, MD 20742-0001
(301) 405-1962

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Chapter 20. MICHIGAN 1 (I-75, Detroit)

Introduction

The Michigan 1 project is located on I-75 (Chrysler Freeway) in downtown Detroit (see Figure 50). Built in 1993, this project came about as a result of an FHWA-sponsored tour of European concrete pavement design and construction practices in 1992. During that tour and a followup tour, one major observation of European concrete design and construction practices was the emphasis placed on the quality of the design, materials, and construction of the pavement, with less concern for higher costs or longer construction periods (FHWA 1992; Larson, Vanikar, and Forster 1993). The tours generated substantial interest in constructing a "European-type" concrete pavement to evaluate its constructibility and performance compared to conventional U.S. designs. This interest led to the selection of the 1.6-km (1-mi) test section on I-75 in Detroit (see Figure 50).

The MI 1 project is not a TE-30 project per se, but gave rise to the TE-30 program. After construction of the project in 1993, interest in constructing similar "high-performance" concrete pavements remained high, and in 1994 FHWA and industry agreed to pursue this effort and launched the TE-30 program.

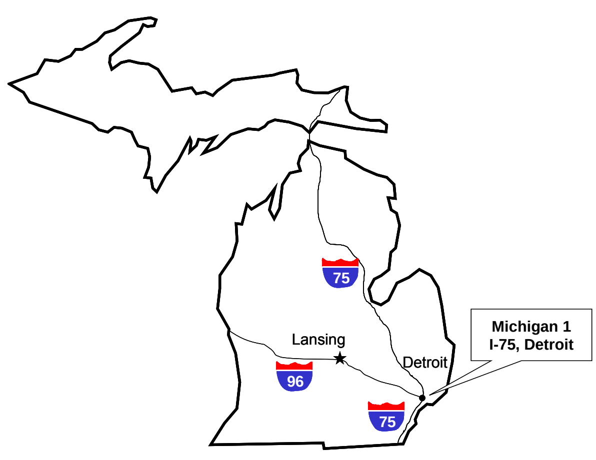

Figure 50. Location of MI 1 project.

Study Objectives

The objective of this project is to determine whether innovative features of typical rigid pavement designs used in European countries can be applied cost effectively to conventional design and construction methods used for rigid pavements in the United States (Smiley 1995). The European pavement was constructed for the purpose of comparing the European design with conventional designs to demonstrate the applicability of certain European concepts to U.S. highway construction (Weinfurter, Smiley, and Till 1994).

Project Design and Layout

This project is located in the northbound lanes of I-75 and consists of two sections: a "control" section representing the Michigan Department of Transportation's (MDOT's) then-current standard concrete pavement design, and the European concrete pavement section incorporating several innovative design features (Weinfurter, Smiley, and Till 1994). The layout of these sections is shown in Figure 51, which also lists some of the key design features of each section.

The existing roadbed for the project lies within an approximate 7.6-m (25-ft) cut section (Weinfurter, Smiley, and Till 1994). The subgrade is predominately a silty clay material, which was required to be compacted to 95 percent of its maximum unit weight in accordance with Michigan's One-Point T-99 (Proctor) Test (Weinfurter, Smiley, and Till 1994).

These sections on I-75 contain three to four traffic lanes in each direction. In 1993, this portion of I-75 carried about 111,000 vehicles per day, including 11 percent heavy trucks (Smiley 1995).

Construction of the sections began in July 1993 and was opened to traffic in November of that same year (Smiley 1995). During the 1994 construction season, southbound I-75 traffic was detoured onto northbound I-75 while it was reconstructed. The entire I-75 reconstruction project was completed in October 1994.

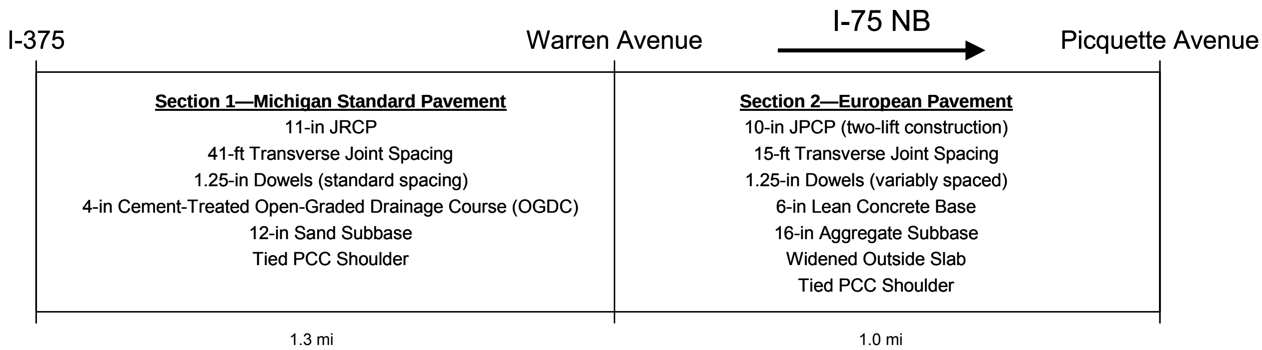

Figure 51. Layout of MI 1 project.

Section 1—Michigan Standard Pavement

The Michigan standard section is 2.1-km (1.3-mi) long and is located south of the European section. The cross section for the Michigan standard section is shown in Figure 52 (Smiley 1995). This is a 279-mm (11-in.) JRCP design with transverse joints spaced at 12.5-m (41-ft) intervals (Weinfurter, Smiley, and Till 1994).

The PCC mix used a higher quality coarse aggregate than standard so that the performance could be compared to the European section. A 100-mm (4-in.) permeable cement-treated base is located beneath the slab. The aggregates for the open-graded base were obtained by crushing the existing I-75 pavement and were stabilized with 6 percent cement (Weinfurter, Smiley, and Till 1994). A geotextile separator is located beneath the open-graded base and above the underlying 305-mm (12-in.) sand subbase (Weinfurter, Smiley, and Till 1994). Longitudinal collector drains (152 mm [6 in.] diameter) are located beneath both the inside and outside reinforced concrete shoulders. All traffic lanes in this section are 3.7 m (12 ft) wide.

Section 2—European Pavement

The European pavement section is 1.6 km (1 mi) long and is located north of the Michigan standard section. It consists of a 254-mm (10-in.) JPCP that was placed in two lifts (see Figure 53 for the pavement cross section). The layers were placed “wet-on-wet” to ensure bonding between the top 64-mm (2.5-in.) concrete layer and the bottom 190-mm (7.5-in.) concrete layer. The same sources for cement and aggregate were used in the top and bottom layers, except that the course aggregate for the top layer was specified to be a 100 percent crushed basalt rock to provide resistance to polishing (Weinfurter, Smiley, and Till 1994). Conventional paving equipment was used for the placement of the two layers.

The two-layer JPCP was constructed directly on a 152-mm (6-in.) lean concrete base (LCB) without the use of a bonding agent. Plane-of-weakness joints were sawed in the LCB at transverse and longitudinal locations to match those of the overlying JPCP slab and thereby prevent reflection cracking (Weinfurter, Smiley, and Till 1994). The joints were sawed to a depth of 61 mm (2.4 in.). A comparison of the specified concrete properties of the LCB, the top layer PCC slab, the bottom layer PCC slab, and the Michigan standard pavement is provided in Table 25 (Weinfurter, Smiley, and Till 1994).

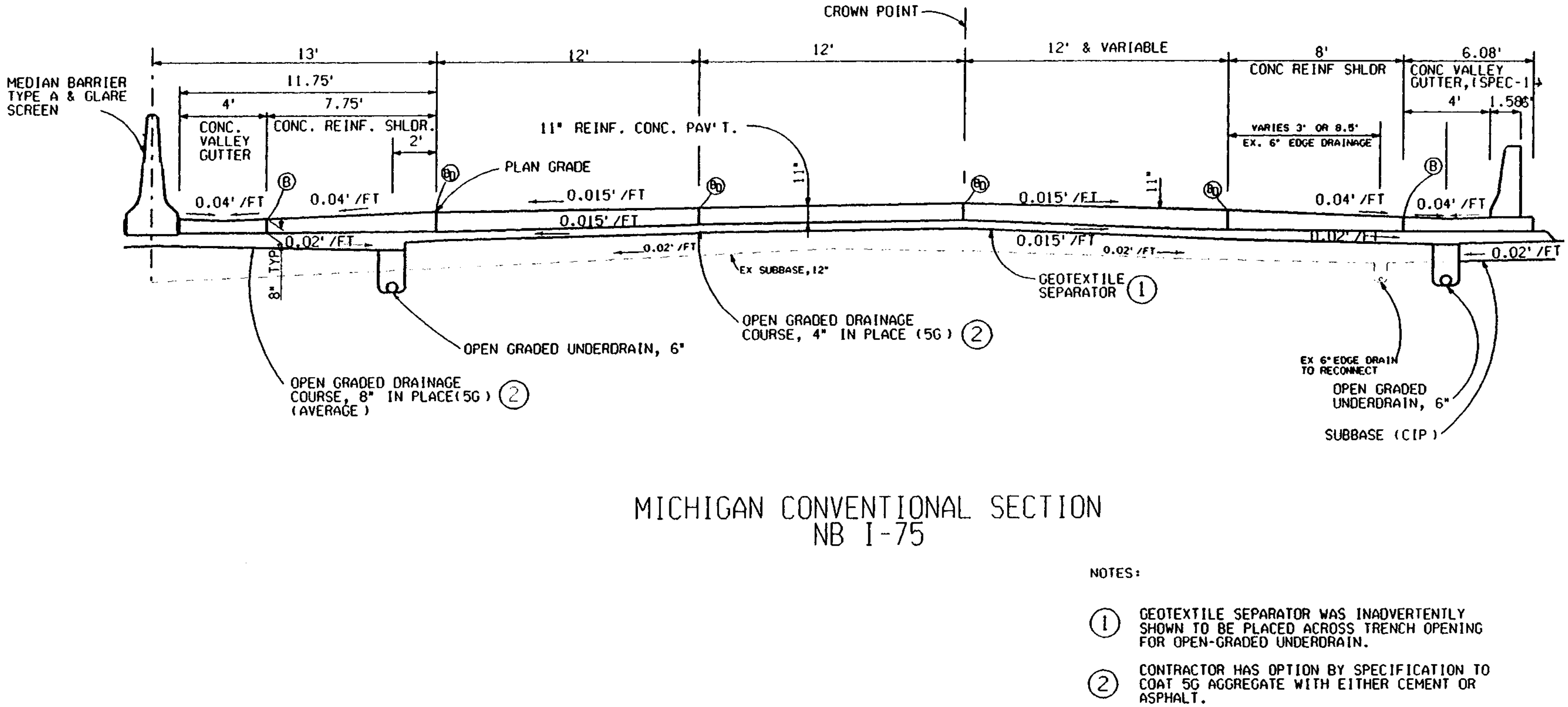

Figure 52. Cross section for Michigan standard pavement (Smiley 1995).

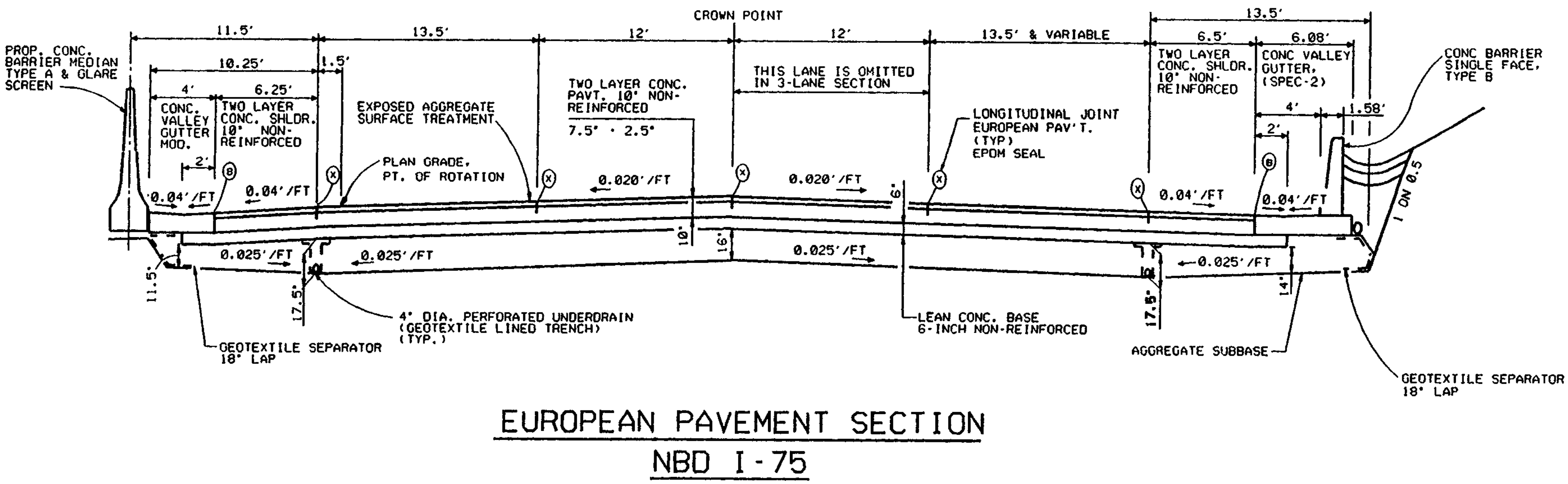

Figure 53. Cross section for European pavement (Smiley 1995).

Table 25. Comparison of Specified Concrete Properties on MI 1 Project (Weinfurter, Smiley, and Till 1994)

PORTLAND CEMENT CONCRETE PROPERTY	EUROPEAN PAVEMENT			MICHIGAN STANDARD PAVEMENT
	TOP LAYER	BOTTOM LAYER	LCB	
28-day compressive strength, lbf/in^2	5,500	5,000	2,500	3,500
28-day flexural strength, lbf/in^2	—	—	—	650
Maximum w/c (by weight)	0.40	0.42	0.70	0.50
Minimum cement content, lb/yd^3	752	588	420	550
Maximum slump, in.	3	3	3	3
Air content, %	6.5 ± 1.5	6.5 ± 1.5	6.5 ± 1.5	6.5 ± 1.5

LCB = lean concrete base

An exposed aggregate surface was specified for the top layer of concrete. This exposed aggregate surface provides surface texture and is expected to reduce noise levels. The exposed aggregate surface was produced through a patented process developed by Robuco, Ltd. of Belgium, consisting of the following steps (Weinfurter, Smiley, and Till 1994):

- Evenly spraying the surface with a set retarder within 30 minutes of the finishing operation.
- Covering the concrete surface with plastic waterproof sheeting (for a period of approximately 20 hours).
- Removing the sheeting and brushing the surface with a brushing machine.
- Placing a curing compound on the newly exposed aggregate surface.

Joint sawing operations were made through the protective sheeting prior to the brushing operation.

A 406-mm (16-in.) thick, nonfrost-susceptible aggregate subbase was placed directly on the subgrade, and longitudinal edge drains were installed beneath both the inside and outside PCC shoulders (Weinfurter, Smiley, and Till 1994). The outer lane consisted of a 4.1-m (13.5-ft) wide outer slab to reduce critical edge loading encroachments.

Transverse contraction joints were spaced at 4.6-m (15-ft) intervals and were designed to match those joints in the underlying LCB. Polyethylene-coated dowel bars, 32 mm (1.25 in.) in diameter and 508 mm (20 in.) long, were placed on chairs at the mid-depth of the composite slab and at the variable spacings shown in Figure 54 (Weinfurter, Smiley, and Till 1994).

The longitudinal and transverse joints were sealed with an ethylene propylene diene terpolymer (EPDM) seal (Weinfurter, Smiley, and Till 1994). Similar to conventional neoprene compression seals, these seals are placed without a lubricant/adhesive and require only a clean (but not dry) joint prior to installation (Weinfurter, Smiley, and Till 1994).

State Monitoring Activities

MDOT has been monitoring the performance of both sections since 1993. Performance data collected include surface distress, ride quality, surface friction, and tire noise levels (Weinfurter, Smiley, and Till 1994). Seasonal pavement deflection measurements are also taken periodically to identify any structural inadequacies that may develop in either pavement section (Weinfurter, Smiley, and Till 1994). Although limited performance monitoring continues, no formal reports have been prepared since 2000.

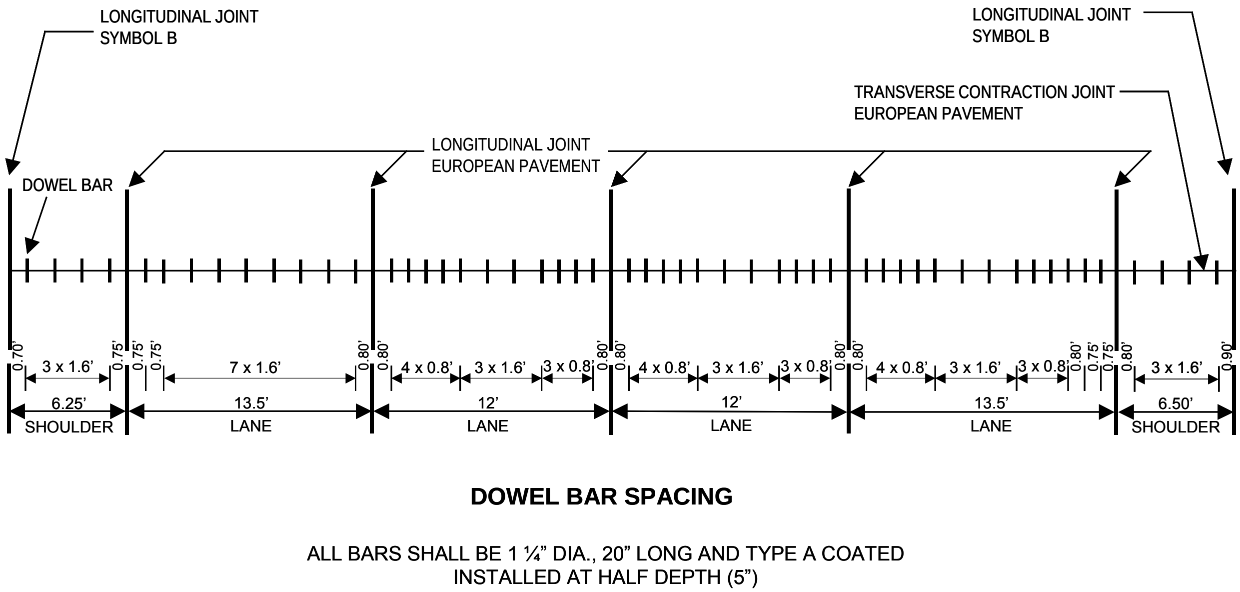

Figure 54. Variable dowel spacings used on European pavement section (Weinfurter, Smiley, and Till 1994).

Results/Findings

Initial Construction Findings

The construction of the European pavement section was accomplished without any major difficulties. Slower production rates were noted, much of which is attributed to an unfamiliarity with two-layer construction and exposed aggregate surfaces (Weinfurter, Smiley, and Till 1994). Among some of the specific recommendations for similar future projects include (Weinfurter, Smiley, and Till 1994):

- The saw cut depth for longitudinal joints in a two-layer pavement is recommended to be between 40 and 45 percent of the total pavement thickness. The saw cut depth for transverse joints is recommended to be between 25 to 30 percent of the total pavement thickness.
- The variable spacing of dowel bars in a basket assembly should be arranged such that the spacing between bars actually represents a standard "uniform" spacing but with selected bars missing. This will save on the costs associated with the fabrication of special dowel bar baskets.
- The top layer of the two-layer pavement should be designed no less than 70 mm (2.75 in.) thick in order to reduce the chance for poor consolidation and a thin surface layer to occur.
- The concrete mixture for the top layer should be revised to eliminate sand particles larger than 1 mm (0.04 in.). These coarser particles prevent the coarse aggregate in the mixture from "locking" together, which is needed in order to reduce noise.
- The environmental ramifications of the dust and slurry generated from the surface brushing operations must be clarified in the design stage. There was excessive dust generated during the brushing operation but fortunately the location was not near a residential area. Disposal of the slurry must meet all local regulations.
- Repair methods need to be developed for exposed aggregate surfaces when the texture depth is determined to be out of the specified range.

One-Year Performance Findings

In the first year after the construction of the pavement sections, distress surveys, skid testing, and noise studies were conducted. Results of those monitoring activities are given below (Smiley 1995):

- The first performance evaluation of the project was conducted in October 1994 just prior to the I-75 freeway being opened to normal traffic operations (the southbound lanes had been re-routed on the northbound lanes). Observations from that initial survey include:

  – On the European pavement section, only one transverse crack was identified. A core retrieved over the crack indicated significant honeycombing. It was later determined in conversations with the construction project staff that the crack was likely a cold joint between old concrete and a botched attempt to patch fresh concrete that had been damaged by contractor paving equipment.

  – Occasional surface popouts were noted throughout the European pavement section. These diameter of these popouts was normally between 25 and 50 mm (1 and 2 in.).

  – The EPDM joint seals on the European pavement section appeared to be in very good condition, although there was occasional evidence of "camelback humping" on some of the transverse joints.

  – The exposed aggregate surface of the European pavement section appeared to have lost macro-texture in the two inner lanes, where traffic was during most of 1994 when the southbound lanes were being reconstructed.

  – On the Michigan standard pavement section, approximately 50 percent of the 12.5-m (41-ft) long panels contained transverse cracks, typical of JRCP designs. The cracks were tight and typically irregular in direction.

- Surface friction was measured on both sections in accordance with ASTM E-274. Tests were conducted in November 1993 and again in April 1994. Over that time, the overall average friction number for the European pavement section increased from 38 to 42, while the overall average friction number for the Michigan standard pavement section increased from 46 to 53.

- A traffic noise study was conducted in June 1994 on both the European pavement section and the Michigan standard pavement section. Both interior and exterior noise levels were recorded. The results from the study indicate that the exposed aggregate surface did not produce the expected reduction in noise levels that are perceptible to persons residing adjacent to the project or when traveling by car. One possible reason for this is that the exposed aggregate surface had too much macrotexture from the excessive spacing of the coarse aggregate particles.

Five-Year Performance Findings

A 5-year analysis of the performance of the two test sections was just recently completed (Buch, Lyles, and Becker 2000). This analysis included an evaluation of traffic, pavement distress, roughness, surface friction, and deflection data obtained on the sections from 1993 to 1998. An economic analysis of each section was also conducted. Summaries of these analyses are provided in the following sections.

Traffic

Pavement performance is typically assessed in terms of how well the pavement stands up to traffic loading, which is generally expressed in terms of 80-kN (18-kip) equivalent single-axle load (ESAL) applications. However, because of variable commercial traffic levels and questionable vehicle classification data, the 5-year evaluation used total traffic volume as the basis for performance comparisons (Buch, Lyles, and Becker 2000). The cumulative total traffic volume (traffic in all lanes in one direction) for these sections is shown in Figure 55 (Buch, Lyles, and Becker 2000).

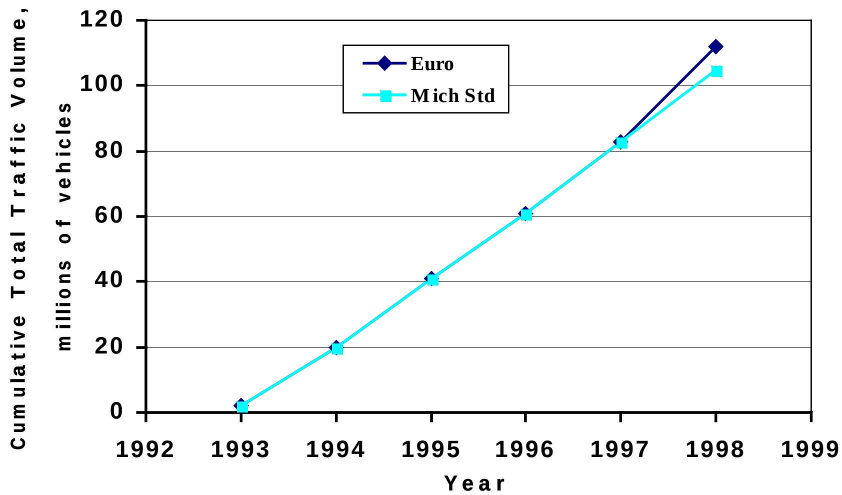

Figure 55. Cumulative total traffic volumes for MI 1 test sections (Buch, Lyles, and Becker 2000).

Pavement Distress

Pavement distress surveys are conducted regularly on Michigan's highways as part of MDOT's pavement management activities. The condition of a pavement is reported in terms of a distress index (DI), which is computed based on the type, extent, and severity of distress. Data from 1995 and 1997 showed a DI of 0 for the European pavement section, indicating a distress-free pavement well below the rehabilitation trigger value of 50 (Buch, Lyles, and Becker 2000). The 1995 and 1997 DI values for the Michigan standard pavement section are 1 for both years, also suggestive of a pavement in very good condition (Buch, Lyles, and Becker 2000).

Roughness

MDOT has monitored the roughness of these pavement sections using an inertial profiler. International Roughness Index (IRI) values computed from the measured profiles are shown in Figure 56 as a function of total traffic (Buch, Lyles, and Becker 2000). The European pavement is noted to be slightly rougher than the Michigan standard pavement, but overall the smoothness levels have remained fairly constant. It should be noted that IRI values less than 1.3 m/km (80 in./mi) are considered to be smooth (Buch, Lyles, and Becker 2000).

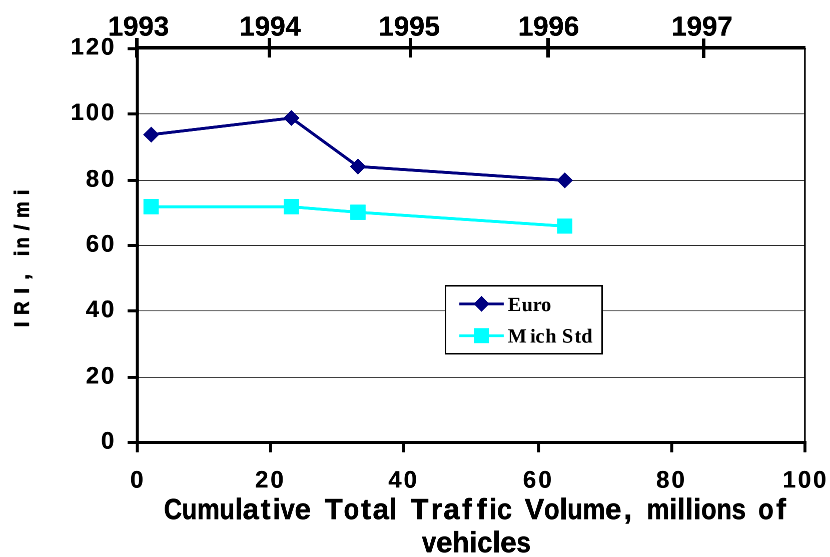

Figure 56. Computed IRI values for MI 1 test sections (Buch, Lyles, and Becker 2000).

Deflection Analysis

Deflection testing on the test sections has been performed twice: once in November 1993 and once in April 1995 (Buch, Lyles, and Becker 2000). The 1993 measurements were taken during daylight hours prior to the pavement being opened to traffic; the 1995 measurements were taken at night because of lane closure restrictions (Buch, Lyles, and Becker 2000). An FWD using a 4000-kg (9000-lb) load was used to conduct the testing.

Table 26 summarizes the results of the FWD testing. It is observed that the magnitude of the maximum mid-slab deflections are less for the European pavement than for the Michigan standard pavement, which is not surprising given the strong base and thick subbase located beneath the European pavement slab. However, it is surprising that the load transfer efficiencies for both sections are as low as they are for such new pavements, and that the most recent LTEs for the European pavement are less than the Michigan standard pavement. One possible reason for this is the wet weather conditions that had preceded the April 1995 testing, which may have contributed to warping of the slabs (Buch, Lyles, and Becker 2000).

Surface Friction

Friction numbers measured for these test sections are shown in Figure 57 (Buch, Lyles, and Becker 2000). The friction numbers for the Michigan standard pavement section are higher than those of the European pavement section, which is somewhat unexpected because of the exposed aggregate surface. Both sections show an initial increase in friction number, which is most likely due to the wearing off of the curing compound (Buch, Lyles, and Becker 2000).

Economic Analysis of Pavement Sections

It was expected that the European pavement section would cost more to construct than the Michigan standard pavement, but the result would be a longer-lasting concrete pavement. A cost analysis showed that the European pavement cost about 234 percent more to construct than the Michigan standard pavement (Buch, Lyles, and Becker 2000). However, it should be noted that the European pavement was a demonstration project that was constructed as part of an "open-house" conference, so the costs are not representative of a conventional paving project.

Table 26. Summary of Deflection Testing Results for MI 1 Sections (Buch, Lyles, and Becker 2000)

		EUROPEAN PAVEMENT		MICHIGAN STANDARD PAVEMENT	
TEST PROPERTY	TEST LOCATION	NOVEMBER 1993	APRIL 1995	NOVEMBER 1993	APRIL 1995
Average maximum mid-slab deflection	Outside lane	1.30 mils	1.41 mils	1.99 mils	2.05 mils
	Lane left of outside lane	1.37 mils	1.32 mils	2.13 mils	2.07 mils
	Inside lane	1.27 mils	1.33 mils	2.28 mils	2.07 mils
Average transverse joint load transfer efficiency	Outside lane	77%	59%	68%	70%
	Lane left of outside lane	79%	62%	72%	70%

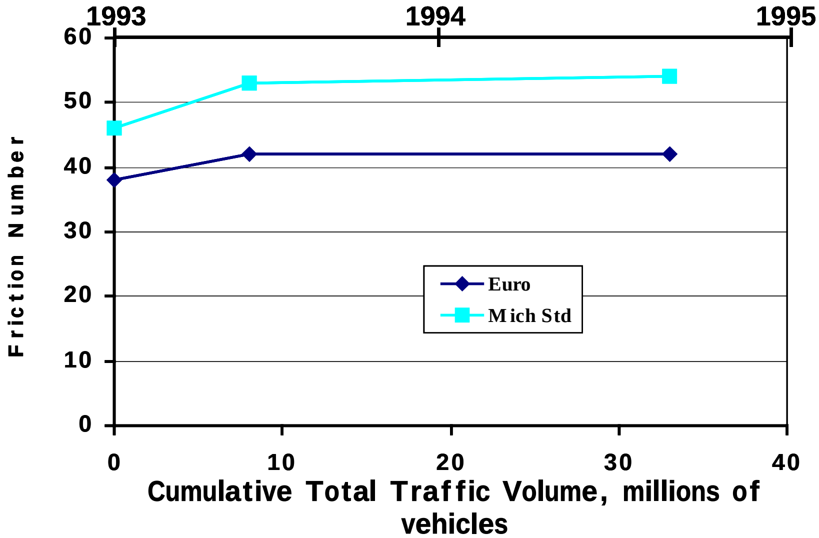

Figure 57. Computed friction numbers for MI 1 test sections (Buch, Lyles, and Becker 2000).

An economic analysis was conducted to compare the life-cycle costs (LCC) of the European and Michigan standard pavements. This required several assumptions regarding future performance, future maintenance cycles, and future rehabilitation schedules. Based on the analysis, it was determined that the European pavement is not competitive with the Michigan standard pavement. However, the calculations are theoretical in the sense that the projected time to maintenance and rehabilitation activities are based on MDOT estimates (Buch, Lyles, and Becker 2000). In addition, the construction costs of the European pavement may not be representative since it was a demonstration project. Nevertheless, the extrapolated data suggest that in order for the European pavement to be competitive, it can cost no more than approximately 17 percent more than the Michigan standard pavement (Buch, Lyles, and Becker 2000).

Point of Contact

Tom Hines
Michigan Department of Transportation
8885 Ricks Road
P.O. Box 30049
Lansing, MI 48909
(517) 322-5711

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Chapter 21. MINNESOTA 1 (I-35W, Richfield)

Introduction

The Minnesota Department of Transportation (Mn/DOT) is reconstructing a portion of I-35W in the Minneapolis-St. Paul area (see Figure 58) using high-performance concrete pavement concepts. Located between I-494 and Highway 62 in Richfield (a suburb of Minneapolis), this project is being partially funded under the TE-30 program and is being designed for a 60-year service life (Rettner 1999).

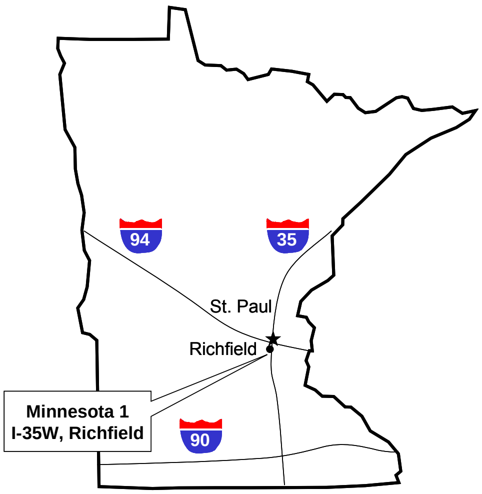

Figure 58. Location of MN 1 project.

Study Objectives

The benefits associated with a pavement that lasts for 60 years include reduced maintenance and user costs (Rettner 1999). However, such a pavement is also expected to have a higher initial cost. This project has the following objectives (Rettner 1999):

- Evaluate the cost/benefit of designing a pavement to last 60+ years with zero maintenance in a very high traffic volume environment.
- Evaluate the performance benefits of using a high performance concrete in an urban paving environment. The specific mix will be designed to have high durability using a number of design features, including:
  - Low permeability.
  - High air content.
  - Well-graded aggregate.
  - High-quality aggregate.
  - Ground granulated blast furnace slag (GGBFS).
- Evaluate the durability of dowel bars clad in stainless steel compared to solid stainless steel dowel bars and assess their ability to perform satisfactorily for 60+ years.
- Evaluate the long-term performance of epoxy-coated dowel bars compared to stainless steel dowel bars.
- Evaluate the relative performance of 38-mm (1.5-in.) and 44-mm (1.75-in.) diameter dowel bars.

Project Design and Layout

The nominal pavement design for this project is a 340-mm (13.4-in.) thick JPCP with 4.6-m (15-ft) transverse joints. The slab thickness was developed based on a design traffic of 100 million ESAL applications over the 60-year design period. Tied concrete shoulders, 315-mm (12.4-in) thick, will be included with the expectation that they will carry traffic during construction or during future rehabilitation activities (Rettner 1999).

The existing sand base course beneath the original pavement is several feet thick and will be left in place for the new PCC pavement. It will be capped with 305 mm (12 in.) of select granular material, which in turn will be topped with a 100-mm (4-in.) thick, Class 5 dense-graded granular base to provide a paving platform for the new PCC pavement. No open-graded base or edge drains are being included in the design.

At least five different test sections were originally planned to be constructed as part of this project (Rettner 1999):

- Section 1—JPCP containing a high-performance concrete mixture and 38-mm

(1.5-in.) by 457-mm (18-in.) stainless steel dowel bars. This design will be constructed for 35 joints over five lanes (three driving, two shoulders) for a total length of 161 m (525 ft).

- Section 2—JPCP containing a high-performance concrete mixture and 44-mm (1.75-in.) by 457-mm (18-in.) stainless steel clad dowel bars. This design will be constructed for 35 joints over five lanes for a total length of 161 m (525 ft).

- Section 3—JPCP containing a high-performance concrete mixture and 38-mm (1.5-in.) by 457-mm (18-in.) stainless steel clad dowel bars. This design will be constructed over the rest of the mainline pavement project (approximately 2500 m [8,200 ft]).

- Section 4—JPCP containing a high-performance concrete mixture and 38-mm (1.5-in.) by 457-mm (18-in.) stainless steel clad dowel bars. This will be constructed on all ramps except the bottom of the northwest exit ramp to CSAH 53. These ramps will be constructed to the same thickness (340-mm [12.4-in.]) as the mainline pavement.

- Section 5—JPCP containing a high-performance concrete mixture and 38-mm (1.5-in.) by 457-mm (18-in.) epoxy-coated dowel bars. This will be constructed only at the bottom of the northwest exit ramp to CSAH 53. This ramp will be constructed to the same thickness (340-mm [12.4-in.]) as the mainline pavement.

Table 27 displays the experimental design matrix for this project. The section with epoxy-coated dowel bars is located on a ramp because of the desire for the entire length of the mainline pavement to last the 60-year design period without traffic disruptions that may be required for epoxy-coated dowel bars (Rettner 1999). Indications in Minnesota are that epoxy-coated bars may not last beyond 25 years without significant corrosion (Rettner 1999). In addition, a project being constructed immediately north of the current project includes epoxy-coated dowel bars and will also serve as a control section for this variable (Rettner 1999).

The high-performance concrete mix was used throughout the entire project for durability and longevity. The aggregate quality specifications were tightened so that only the most durable aggregate sources were eligible, including a stipulation limiting the amount of limestone to no more than 20 percent (Rettner 1999). In addition, a well-graded coarse aggregate was used for improved workability without the use of excessive amounts of water reducer. Other notable features of the concrete mix design include a higher specified air content (6 to 8 percent entrained) for increased freeze-thaw protection, the inclusion of ground granulated blast furnace slag for reduced permeability, a 28-day rapid chloride permeability of 2500 coulombs (which, according to ASTM C 1202, is a moderate level of permeability), and a water-to-cementitious-material ratio less than 0.40 (Rettner 1999).

Table 27. Experimental Design Matrix for MN 1 Project

	DIAMETER (IN.)	MAINLINE PAVEMENT	RAMP PAVEMENT
Stainless Steel Clad Dowel Bar	1.5	Section 3	Section 4
	1.75	Section 2	
Solid Stainless Steel Dowel Bar	1.5	Section 1	
	1.75		
Epoxy-Coated Steel Dowel Bar	1.5		Section 5
	1.75		

These sections were constructed during the summer of 2000. However, there was considerable difficulty in obtaining the required number of stainless steel clad dowel bars from the manufacturer, and consequently several modifications to the original experimental design were made during construction. Most of the northbound and all of the southbound lanes were constructed with solid stainless steel dowel bars, and only a portion of the northbound lanes received stainless steel clad dowel bars. Furthermore, the outside shoulders were constructed with plastic-coated bars, and it appears as if the exit ramp that was to receive epoxy-coated bars will be also be constructed using plastic-coated bars. Finally, the inside shoulders will not be paved until the 2001 construction season.

State Monitoring Activities

Mn/DOT will monitor the performance of these pavement sections for a minimum of 5 years. Pavement distress surveys and ride quality measurements will be conducted every other year using standard Mn/DOT procedures (Rettner 1999). Load transfer efficiency will be measured on the outside lane using the Mn/DOT FWD on an every other year basis if traffic allows (Rettner 1999).

Preliminary Results/Findings

Some findings on the use of high performance concrete specifications and high performance materials in Minnesota were presented a the 2003 Annual TRB meeting. The following sections summarize some of the findings (Turgeon 2003; Rangaraju 2003).

Cement Paste Specification

The contractor elected to provide a mix containing 229 kg/m^3 (384 lbs/yd^3) cement and 123 kg/m^3 (206 lb/yd^3) GGBFS. This met all mix requirements including the RCP test. Typically contractors elect to use the 25 percent fly ash mixes under the standard specification. Fly ash is approximately half the cost of cement or GGBFS. The introduction of GGBFS to the paving process registered some complaints of a "sticky" mix from the finishers.

The mix was produced concurrently from two batch plants located at the same site, using the same mix design. This further aggravated the variability in the plastic air contents, which ranged from below 5 percent to over 10 percent. Upon completion of the project, price reductions were assessed for failing plastic air contents. Most price reductions related to tests results of around 6.0 percent, but some tests were as low as 4.0 percent.

The hardened air contents, as determined using ASTM C457, were determined for the top, middle, and bottom third of each sample. Results ranged from 6.7 percent to 15.7 percent, with an average value of 10.6 percent. Sixteen of the 26 hardened air contents were above the maximum allowable of 10.0 percent. Only one was below the range minimum of 7.0 percent. The cores were taken at random therefore they do not coincide with any of the plastic air tests taken. Even so, these results do not correspond well with the plastic air contents discussed previously. Further analysis of the linear traverse results showed the prevalence of small voids with diameters less than 100 microns. Few air voids are present in the 150 micron to 250 micron range. It has been theorized that the volumetric air meters used to determine plastic air contents are not efficient when measuring small bubbles, which could account for the test result differences. Small bubbles may be more susceptible to filling with secondary reaction by-products thus undermining the long-term void structure effectiveness. The ratio of entrapped to entrained air was determined to be acceptable; however this determination was not available until a few months after paving was complete.

Aggregate Specification

The coarse aggregate supplied consisted of a low carbonate class C gravel. This material qualified for the full aggregate quality bonus. The contractor added "Safety Grit," No. 8 sieve to 9.5-mm (0.375-in.) sand, to the typical coarse rock and sand mixture to meet the denser gradation requirements.

Pavement Thickness

The pavement was specified and constructed at a thickness of 340 mm (13.5 in.). The ability to finish the surface, avoiding edge slump deformation and meeting the low w/c ratio, were competing considerations that required the contractor's constant attention. The additional thickness added to the complexity of these issues.

Load Transfer

The first paving was scheduled for fall 1999, but project conflicts delayed it until spring 2000. At that time, approximately 75 percent of the required quantity of stainless clad bars was available for installation. To keep the project on schedule, plastic-coated dowel bars of the same dimension were substituted and used in the shoulder. The opposing direction of traffic was paved in summer 2000, but sufficient stainless steel clad bars were not available. Consequently, the majority of this work used solid stainless dowel bars at a significant increase in cost.

Cost

Placement costs are primarily a factor of the complexity of the project and the pavement thickness. Projects that include long uninterrupted stretches of pavement are less costly to place since less time is spent mobilizing equipment and setting up. For thicker pavements, the larger volume of concrete needed to produce an equivalent area of concrete slows production, which leads to higher placement costs. Edge slump is also a more prevalent problem with thicker pavements. The placement bid price for the 340-mm (13.5-in.) pilot HPC project was $7.55/m^2 ($6.31/yd^2). Four projects let the same year with pavement thickness ranging from 254 mm to 267 mm (10.0 to 10.5 in.) had an average placement cost of $6.85/m^2 ($5.72/yd^2). The cost difference seems to be primarily due to thickness.

The two factors that greatly influence the structural concrete unit price are the project quantity and the ability of paving contractors to produce the material from their own mobile batch plants. Small quantity projects or projects without space to set up a portable batch plant require the paving contractor to purchase concrete from a local ready-mix supplier at a higher cost. Due to the confined location, the paving contractor purchased the 22,841 m^3 (30,054 yd^3) of concrete from a ready-mix supplier at a unit price of $97.40/m^3 ($74.02/yd^3). This unit price does not reflect any potential incentive / disincentives. Four similarly sized, non-HPC projects let that same year had the following quantities and unit bid prices for structural concrete: 13,824 m^3 at $78.65/m^3 (18,189 yd^3 at $59.77/yd^3); 19,838 m^3 at $86.37/m^3 (26,103 yd^3 at $65.64/yd^3); 40,806 m^3 at $73.13/m^3 (53,692 yd^3 at $55.58/yd^3); and 7,665 m^3 at $116.18/m^3 (10,085 yd^3 at $88.30/yd^3). The different quantities, aggregate availability, and project complexities make it difficult to make a direct assertion as to the added cost of the HPC specification. However, based upon feedback from the paving contractor, the added cost for the HPC structural concrete is approximately $13.00/m^3 ($10.00/yd^3).

The unit costs of the different types of dowel bars also varied considerably. The epoxy-coated dowel bar cost $5.20; the 38-mm (1.5-in.) stainless steel clad dowel bar $11.60; the 44-mm (1.75-in.) stainless steel clad dowel bar $14.30; and the solid stainless steel dowel bar $19.70. A 1.6-km (1.0-mi) long project consisting of two 3.6-m (12-ft) lanes and 4.6-m (15-ft) joint spacing requires 8,448 dowel bars, yielding a total dowel bar cost of $43,929.60 for the epoxy-coated dowels, $97,996.80 for the 38-mm (1.5-in.) stainless steel clad dowels, $120,806.40 for the 44-mm (1.75-in.) stainless steel clad dowels, and $166,425.60 for the solid stainless steel dowels. The annualized cost for a 60-year design using solid stainless dowels is significantly higher than the annualized cost of a standard 35-year design (ignoring user costs).

Point of Contact

Curt Turgeon
Minnesota Department of Transportation
1400 Gervais Avenue
Maplewood, MN 55109
Ph: (651) 779-5535
Fax: (651) 779-5616
curt.turgeon@dot.state.mn.us

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Chapter 22. MINNESOTA 2 (Mn/Road Low-Volume Road Facility)

Introduction

The Minnesota Road Research Project (Mn/Road) is a major highway research initiative studying the performance of asphalt-, concrete-, and aggregate-surfaced roadways. Opened to traffic in 1994, the purpose of the test road is to gain a better understanding of pavement response and pavement behavior to traffic and environmental loadings, with the expectation that improvements to existing pavement design and evaluation procedures will result.

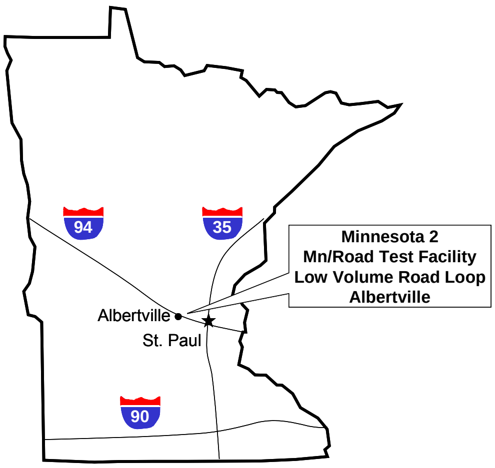

Figure 59. Location of MN 2 project.

The Mn/Road facility is located near Albertville, just northwest of Minneapolis-St. Paul (see Figure 59). It consists of two different road segments running parallel to I-94: one a 5.7-km (3.5-mi) mainline roadway in the westbound direction carrying interstate traffic, and the other a 4.0-km (2.5-mi) low-volume road (LVR) loop exposed to controlled truck weight and traffic loadings. A variety of heavily instrumented test sections, consisting of different thicknesses of concrete, asphalt, and aggregate surfaces as well as other design features, have been constructed within each road segment. The asphalt sections on the LVR loop were designed to last approximately 3 years, and after 5 years several of these sections have required rehabilitation or reconstruction (Mn/DOT 2000). By eliminating one of the deteriorated gravel test sections on the LVR, three new JPCP test sections (numbered 32, 52, and 53) were constructed with partial funding from the TE-30 program (Mn/DOT 2000). Figure 60 shows the layout of the Mn/Road facility and the approximate location of the new test sections.

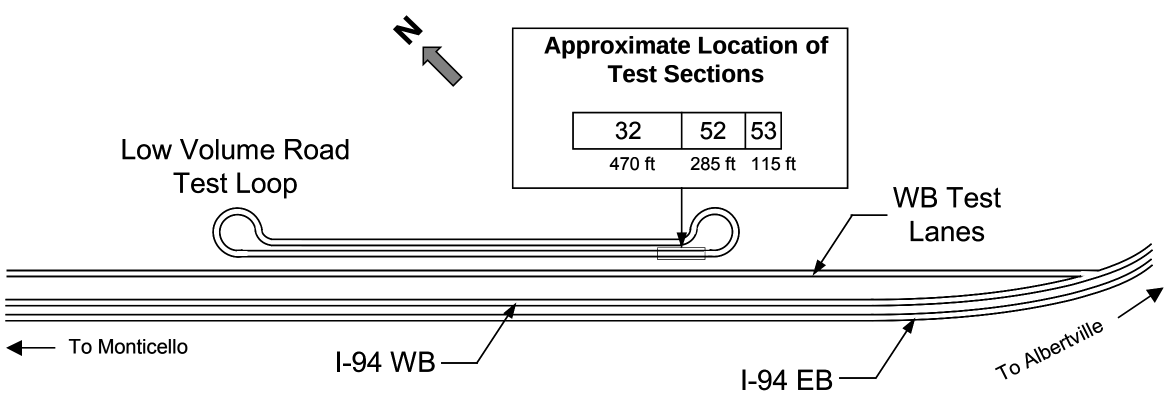

Figure 60. Approximate location of new LVR test sections at Mn/Road facility.

Study Objectives

These three new test sections were constructed to obtain data not previously considered in the original LVR designs, as well as to satisfy the data needs of local agencies (Mn/DOT 2000). Specific objectives include the following (Mn/DOT 2000):

- Characterization of early-age and long-term slab curling and warping.
- Measurement of early-age and long-term internal slab stresses and shrinkage.
- Evaluation of long-term joint load transfer behavior of different dowel bar types.
- Evaluation of the performance of a thin, low-cost JPCP for low-volume applications.
- Evaluation of a concrete mixture containing ground granulated blast furnace slag (GGBFS).
- Validation of sensor readings from other Mn/Road test sections.
- Further investigation of the feasibility of retrofitting sensors in a pavement.

Project Design and Layout

The three test sections are located on the southern tangent of the LVR, near the eastern loop. Design characteristics of each section are provided below (Mn/DOT 2000):

- Section 32—127-mm (5-in.) JPCP with 3.1-m (10-ft) transverse joints. The pavement rests on a 178-mm (7-in.) aggregate base. The joints are sealed with a hot-poured joint sealant material and do not contain dowel bars. The concrete mix contains 35 percent GGBFS.
- Section 52—190-mm (7.5-in.) JPCP with 4.6-m (15-ft) transverse joints. A 127-mm (5-in.) aggregate base is located beneath the pavement slab. The transverse joints are sealed with silicone. The outside lane is 4.0-m (13-ft) wide and the inside lane is 4.3-m (14-ft) wide. Four different load transfer devices are included in the section:
  - 25-mm (1-in.) diameter, 381-mm (15-in.) long, epoxy-coated dowel bars.
  - 32-mm (1.25-in.) diameter, 381-mm (15-in.) long, epoxy-coated dowel bars.
  - 32-mm (1.25-in.) diameter, 457-mm (18-in.) long, fiber-reinforced polymer dowel bars.
  - 38-mm (1.50-in.) diameter, 457-mm (18-in.) long, fiber-reinforced polymer dowel bars.
- Section 53—190-mm (7.5-in.) JPCP with 4.6-m (15-ft) transverse joints. A 127-mm (5-in.) aggregate base is beneath the pavement. The transverse joints are sealed with silicone. The outside lane is 4.0-m (13-ft) wide and the inside lane is 4.3-m (14-ft) wide. None of the joints is doweled.

Table 28 summarizes some of the key design features for the MN 2 test sections. Figure 61 illustrates the layout of test sections 52 and 53.

Instrumentation layout for these test sections began in April 2000, with actual construction commencing in June (Mn/DOT 2000). Table 29 summarizes the type and number of sensors in these test sections (Burnham 2001). Concrete paving was performed on June 30, which was immediately followed by 72 hours of continuous monitoring of the internal shrinkage, strain, and temperatures, as well as the external shape of the slabs (Mn/DOT 2000).

Table 28. Summary of Design Features for MN 2 Test Sections

DESIGN FEATURE	SECTION 32	SECTION 52	SECTION 53
Pavement type	JPCP	JPCP	JPCP
Concrete mix	GGBFS	Conventional	Conventional
Slab thickness, in.	5	7.5	7.5
Base type	7 in., aggregate	5 in., aggregate	5 in., aggregate
Transverse joint spacing, ft	10	15	15
Load transfer	None (aggregate interlock)	1-in. epoxy-coated dowels 1.25-in. epoxy coated dowels 1.25-in. fiber reinforced polymer 1.50-in. fiber reinforced polymer	None (aggregate interlock)
Joint sealant	Hot-pour	Silicone	Silicone
Longitudinal joint tie bars	No. 4 bars @ 30-in. spacings	No. 4 bars @ 30-in. spacings	No. 4 bars @ 30-in. spacings
Lane width, ft	12 (both)	13 (outside) 14 (inside)	13 (outside) 14 (inside)
Section length, ft	470	285	115

JPCP = jointed plain concrete pavement; GGBFS = ground granulated blast furnace slag

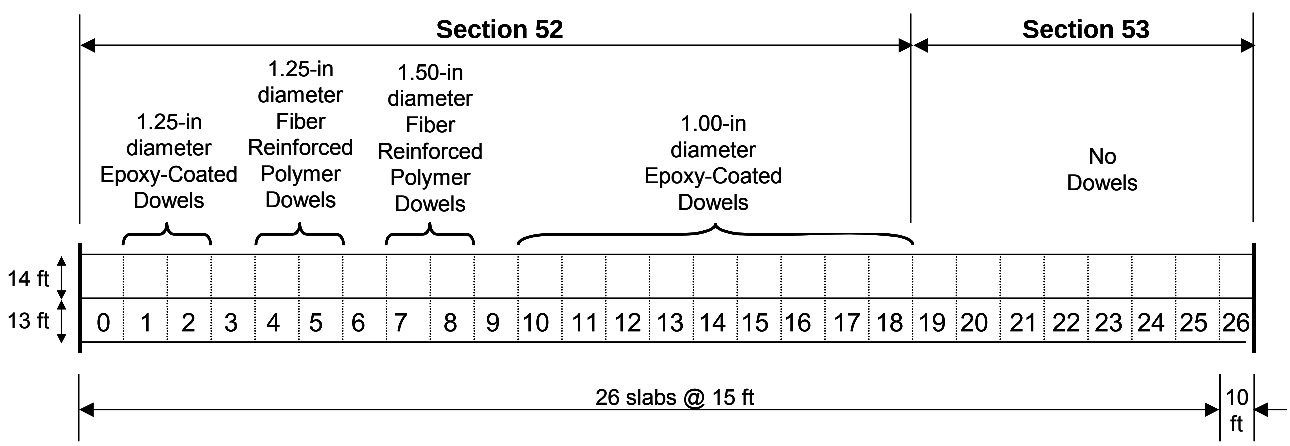

Figure 61. Layout of test sections 52 and 53 for MN 2 project.

Table 29. Sensor Types and Quantities for Test Sections 32, 52, and 53

SENSOR TYPE	MEASUREMENT TYPE	MANUFACTURER	QUANTITIES		
			SECTION 32	SECTION 52	SECTION 53
Concrete embedment sensor	Strain	Tokyo Sokki	37	87	20
Displacement transducer	Joint opening	Tokyo Sokki	4	8	4
Invar reference rod	Elevation	Mn/DOT		4	4
Stainless steel reference rod	Elevation	Mn/DOT	2	5	1
Concrete embedment sensor	Strain	MicroMeas	8		
Psychrometer sensor	Relative humidity	Wescor	4	4	4
Moisture resistance sensor	Relative humidity	ELE International	6	6	6
Dynamic Soil Pressure Sensor	Pressure	Geokon	1	3	3
Static soil pressure sensor	Pressure	Geokon	1	2	2
Thermocouple (T-type)	Temperature	Omega	16	16	16
Time domain reflectometer	Soil moisture	Campbell Scientific	6	4	4
Vibrating wire strain sensor	Strain	Geokon	16	35	34

State Monitoring Activities

As with all pavements at the Mn/Road test facility, these sections are subjected to an intensive data collection effort. In addition to the data obtained from the instrumented slabs within each section, visual distress data, faulting measurements, ride quality, and FWD deflection data are collected (Mn/DOT 2000).

Preliminary Results and Findings

One of the objectives of this study was to evaluate the long-term joint performance of different dowel bar types. Preliminary data on the dowel bars is provided below in Figures 62 and 63. The initial load transfer efficiency was lower for the FRP bars compared to the steel dowels by about 5 percent. After 2 years in service, the load transfer efficiency for the FRP bars dropped from approximately 80 percent to approximately 70 to 75 percent, which are considered critical minimum levels. The performance of the epoxy-coated steel bars appears relatively constant since construction. The differential deflections measured across each joint are still relatively low, as can be seen in Figure 63.

Another objective of the MN 2 project was to evaluate the early-age and long-term slab curling and warping. Early-age profile measurements made using the Dipstick are provided in Figures 64 to 66. The profiles measurements revealed that large positive temperature moments can produce sufficient deformation in the slab such that the slab is unsupported along the whole length of the transverse joint due to upward curvature. Large negative gradients can result in a deformed slab that forces the bottom of the slab into the base layer along the complete length of the transverse joint. The total range of movement of the corner of the slab adjacent to the asphalt shoulder for the temperature conditions present during measurement period was approximately 3,500 microns. The total movement for the corner of the slab adjacent to the longitudinal joint was 2,500 microns. These measurements were for slabs without dowel or tie bars. A more thorough analysis of the early-age performance data can be found in Vandenbossche (2003).

Extensive research has also been performed on the analysis of measured dynamic strains in the thin concrete pavements on the MN2 project. An in-depth discussion of the findings is provided by Burnham (2003; 2004).

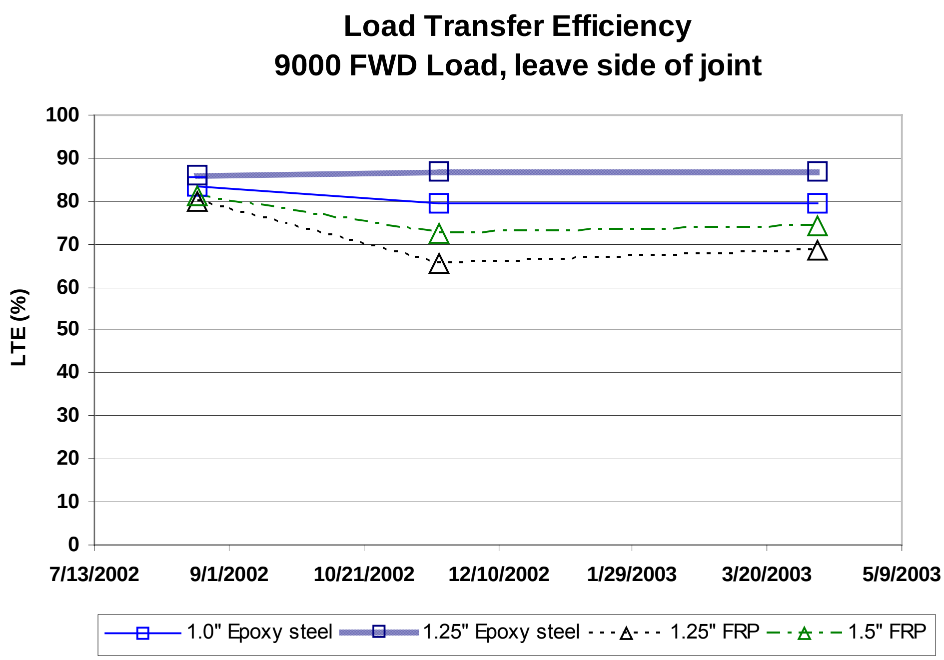

Figure 62. Load transfer efficiency of joints with different types of dowel bars for MN 2 project.

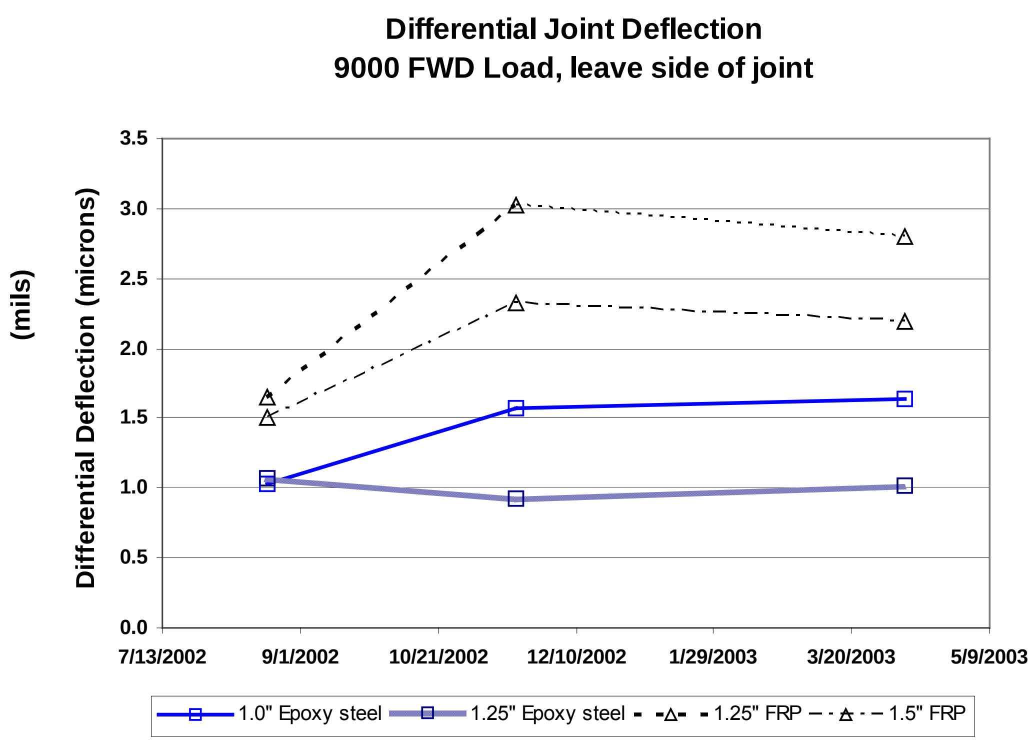

Figure 63. Differential deflections across joints with different types of dowel bars for MN 2 project.

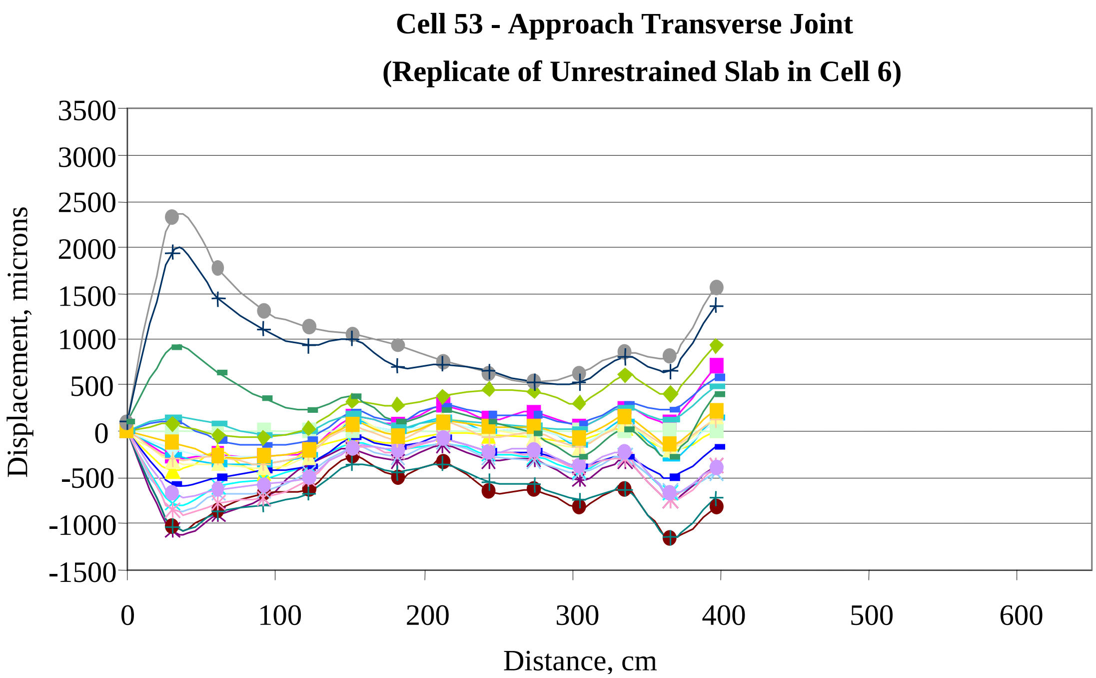

Figure 64. Transverse profile of unrestrained slab in Cell 53 for MN 2 project (Vandenbossche 2003).

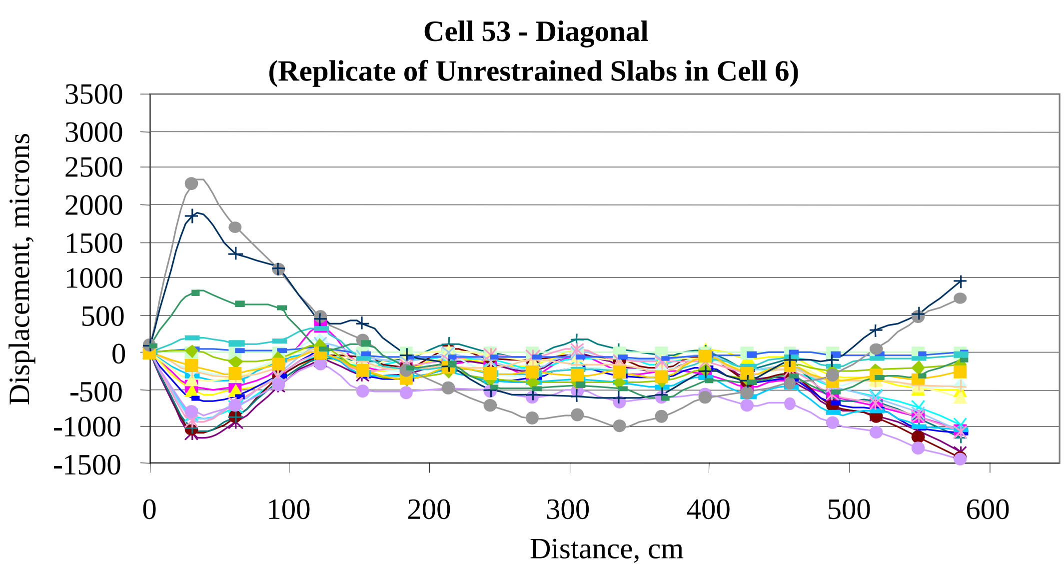

Figure 65. Diagonal profile of unrestrained slab in Cell 53 for MN 2 project (Vandenbossche 2003).

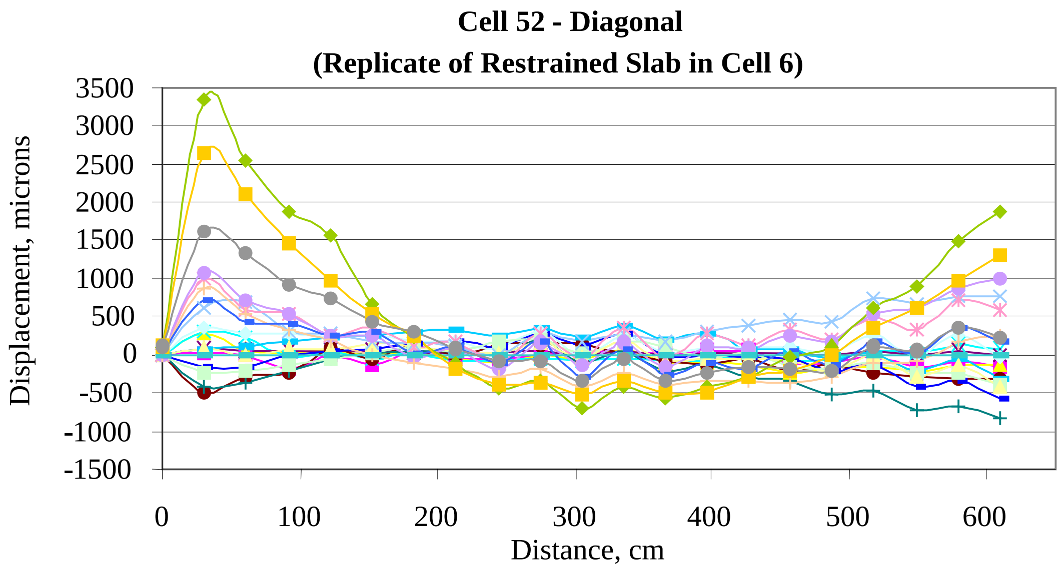

Figure 66. Diagonal profile of restrained slab in Cell 52 for MN 2 project (Vandenbossche 2003).

Point of Contact

Thomas Burnham
1400 Gervais Avenue
Maplewood, MN 55109
(651) 779-5606
Tom.Burnham@dot.state.mn.us

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Chapter 23. MINNESOTA 3 (Mn/Road, Mainline Road Facility, and US 169, Albertville)

Introduction

This study included two projects: three test sections on US 169 in Elk River and six test sections on I-94, at the Minnesota Road Research (Mn/Road) test facility (see Figure 67).

The US 169 site represents a typical application for ultrathin whitetopping. Most of the loads are static and the traffic is constantly starting and stopping. This section of US 169 carries approximately 400,000 ESALS per year. The few cracks that were present in the existing asphalt pavement were in good condition, but the asphalt itself was not, especially along the outer edge. A large amount of rutting (greater than 25 mm [1 in.]) was also present before milling.

The I-94 section was in good condition. Very few cracks were present on the existing asphalt pavement and all were of low severity. The roadway is subjected to heavy traffic loadings (over 1 million ESALs per year). Although this is not a typical application, it provided the opportunity to monitor the performance of the overlay under accelerated loading conditions and to evaluate ultrathin whitetoppings as an overlay alternative for high-volume roads (Vandenbossche and Rettner 1998; Vandenbossche and Rettner 1999).

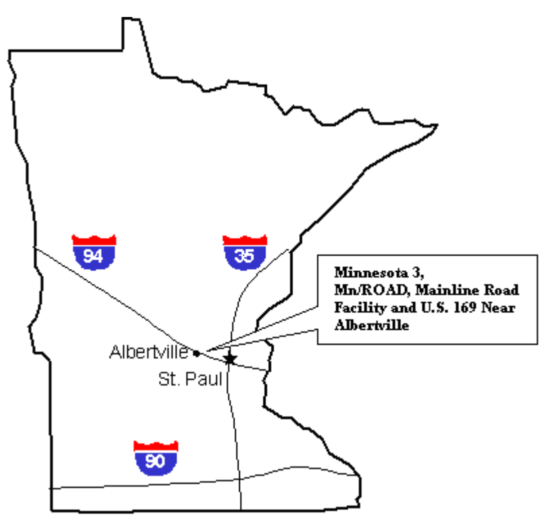

Figure 67. Location of MN 3 project.

Study Objectives

The purpose of the test sections was to further evaluate how whitetopping performs in Minnesota and to determine what design features are desirable to optimize the life of the pavement. These heavily instrumented sections allow the static and dynamic response of the pavement under various applied and environmental loading conditions to be measured. In doing this, a better understanding can be obtained on how to more accurately model whitetopping overlays so that a more efficient design method and performance prediction model can be developed.

Project Design and Layout

The first project was located on the outer southbound lane of US 169 in Elk River at the intersections of Jackson, School, and Main streets. The first 240.2 m (788 ft) north of each intersection was overlaid with 76.2 mm (3 in.) of fiber reinforced concrete and the final 3.66 m (12 ft) was paved 203.2 mm (8 in.) thick. The original asphalt pavement was constructed in 1961 on a sandy subgrade and consisted of a 101.6-mm (4-in.) asphalt surface on 127-mm (5-in.) of a relatively dense-graded aggregate base and 152.4-mm (6-in.) of subbase. In 1991, 50.8 (2 in.) of asphalt was milled and the pavement was overlaid with 38.1 mm (1.5 in.) of asphalt. The average asphalt thickness based on a total of 10 cores was 158.75 (6.25 in.). Temperature and dynamic and static strain sensors were installed at the Jackson Street intersection. A summary of each test section is provided in Table 30.

On the Mn/Road project, a 342.9-mm (13.5-in.) full-depth asphalt pavement was whitetopped with a fiber-reinforced concrete overlay. The original asphalt pavement was constructed in 1993 on a silty-clay subgrade. The existing asphalt pavement was previously a transition zone that separated the 5- and 10-year mainline test cells. The test section was divided up into six separate test cells with various thicknesses and joint patterns. These cells were instrumented with strain, temperature and moisture sensors. A summary of the test cells is provided in Table 31.

The US 169 test sections were paved on September 17, 1997 and the I-94 test sections on October 23, 1997. All concrete was required to have an air content of 6.5 percent ± 1.5 percent and a flexural strength of 2757.9 kPa (400 lbf/in^2) at 3 days so that the overlays could be opened to traffic. The flexural and compressive strengths for the concrete on US 169 are provided in Tables 32 and 33, respectively. The flexural strengths for both the polypropylene and polyolefin mixes were similar. The polypropylene concrete had slightly higher compressive strengths even though the w/c ratio for the polyolefin mix was lower. Increasing the fiber content in the polyolefin mix by 8 times that of the polypropylene mix contributed to the lower strength of the polyolefin concrete. Both mixes met the Minnesota Department of Transportation's (Mn/DOT's) flexural and compressive strength requirements.

In addition to the material testing on the US 169 project, extensive material property testing was also performed on the concrete mixture used to construct the I-94 test sections. These material properties are summarized in Tables 34 through 36.

Additional information on the concrete mixture design and the construction of these test sections can be found in the report by Vandenbossche and Rettner (1998).

Table 30. Summary of US 169 Whitetopping Test Sections for MN 3 Project

TEST CELL DESCRIPTION	SENSOR	NO. OF SENSORS
Jackson Street intersection:		
75 mm – 1.2 m x 1.2-m panels	Dynamic strain	32
(3 in. – 4 ft x 4-ft)	Static strain	4
Polypropylene fibers	Thermocouple	14
Main Street intersection:		
75 mm – 1.2 m x 1.2-m panels	None	–
(3 in. – 4 ft x 4-ft)		
Polypropylene fibers		
School Street intersection:		
75 mm – 1.8 m x 1.8-m panels	None	–
(3 in. – 6 ft x 6-ft)		
Polyolefin fibers		

Table 31. Summary of Mn/Road Whitetopping Test Sections for MN 3 Project

MnROAD TEST CELL DESCRIPTION		SENSOR	NO. OF SENSORS
Cell 93	100 mm – 1.2 m x 1.2-m panels (4 in. – 4 ft x 4-ft) Polypropylene fibers	Dynamic strain Dynamic asphalt foil strain gauge Static strain Thermocouple Moisture	32 2 8 14 12
Cell 94	75 mm – 1.2 m x 1.2-m panels (3 in. – 4 ft x 4-ft) Polypropylene fibers	Dynamic strain	32
Cell 95	75 mm – 1.5 m x 1.8-m panels (3 in. – 5 ft x 6-ft) Polyolefin fibers	Dynamic strain Dynamic asphalt foil strain gauge Static strain Thermocouple Moisture	32 2 8 12 12
Cell 96	150 mm – 1.5 m x 1.8-m panels (6 in. – 5 ft x 6-ft) Polypropylene fibers	Dynamic strain	32
Cell 97	150 mm – 3 m x 3.7-m panels (6 in. – 10 ft x 12-ft) Polypropylene fibers	Dynamic strain Dynamic asphalt foil strain gauge Static strain Thermocouple Moisture	32 2 8 16 12
Cell 97b	150 mm – 3 m x 3.7-m panels (6 in. – 10 ft x 12-ft) Polypropylene fibers Doweled	None	–

Table 32. Flexural Strengths for US 169 on the MN 3 Project

TIME OF STRENGTH TESTING	CONCRETE WITH POLYPROPYLENE FIBERS (JACKSON STREET INTERSECTION) (LBF/IN^2)	CONCRETE WITH POLYOLEFIN FIBERS (MAIN STREET INTERSECTION) (LBF/IN^2)
28-day	590	570

Table 33. Compressive Strengths for US 169 on the MN 3 Project

TIME OF STRENGTH TESTING	CONCRETE WITH POLYPROPYLENE FIBERS (JACKSON STREET INTERSECTION) (LBF/IN^2)	CONCRETE WITH POLYPROPYLENE FIBERS (SCHOOL STREET INTERSECTION) (LBF/IN^2)	CONCRETE WITH POLYOLEFIN FIBERS (MAIN STREET INTERSECTION) (LBF/IN^2)
28-day	4900	4900	4400
28-day	5400	5900	5300

Table 34. Compressive Strengths for I-94 on the MN 3 Project

TIME OF STRENGTH TESTING	CONCRETE WITH POLYPROPYLENE FIBERS (LBF/IN^2)	CONCRETE WITH POLYOLEFIN FIBERS (LBF/IN^2)
1-day	2000	1600
3-day	3900	2900
7-day	4800	4200
14-day	5500	4800
28-day	6100	5300

Table 35. Elastic Moduli for I-94 on the MN 3 Project

TIME OF STRENGTH TESTING	CONCRETE WITH POLYPROPYLENE FIBERS (LBF/IN^2)	CONCRETE WITH POLYOLEFIN FIBERS (LBF/IN^2)
7-day	4.5 x 10-6	4.3 x 10-6
28-day	4.8 x 10-6	4.4 x 10-6

Table 36. Poisson's Ratio for I-94 on the MN 3 Project

TIME OF STRENGTH TESTING	CONCRETE WITH POLYPROPYLENE FIBERS	CONCRETE WITH POLYOLEFIN FIBERS
7-day	0.19	0.19
28-day	0.19	0.19

State Monitoring Activities

Climatic, static strain, and dynamic strain data are collected periodically throughout the year on the US 169 project. On the I-94 project, temperature, moisture, and static strain data was collected every 15 minutes since construction. Dynamic strain data was collected for both static and dynamic truck loads and in conjunction with FWD testing four times per year, once during each season. Distress, ride and faulting data was also collected four times per year.

Preliminary Results/Findings

Distinct cracking patterns developed within each test section. The UTW test sections with a 1.22-m x 1.22-m (4-ft x 4-ft) joint pattern contained corner breaks and transverse cracks. The corner breaks occurred primarily along the inside longitudinal joint and the lane/shoulder (L/S) longitudinal joint while the transverse cracks developed in the panels adjacent to the shoulder. The transverse cracks typically develop approximately 1/3 of the length of the panel away from the transverse joint. The test section with the 1.83-m x 1.83-m (6-ft x 6-ft) joint pattern performed significantly better because the longitudinal joint does not lie in the inside wheelpath. This significantly reduces the edge and corner stresses. Very tight longitudinal cracks developed on the 152.4-mm (6-in.) overlay with a 1.83-m x 1.83-m (6-ft x 6-ft) joint pattern on I-94. Reflective cracking was not observed in any of the test sections, although reflective cracking has been found to occur in UTW placed on thicker HMA pavements, such as on I-94 (Vandenbossche 2003).

The strains measured on I-94 were consistently lower than those on US 169, even when measurements were made at higher HMA temperatures. The reduction in strain is a result of the increase in thickness and quality of the HMA on I-94 and an increase in the bond strength between the two layers. Increases in the temperature of the HMA also produce much smaller increases in strain on I-94 compared to US 169, except for the strains measured at mid-panel. Strains at mid-panel on the I-94 project approach those measured at mid-panel on the US 169 project. It was found that applying a load when the HMA temperature is high produces similar strains in the UTW regardless of the thickness of the HMA layer when a good bond is obtained (Vandenbossche 2003).

The strain measurements emphasize the importance of the support provided by the HMA layer. A reduction in this support occurs when the temperature of the HMA is increased or when the HMA begins to ravel. The results from the strain measurements and the cores pulled from the test section indicate the HMA ravels at a faster rate along the joints where there is greater access for the water to enter the pavement structure. The lane-shoulder joint is the most difficult to keep sealed and therefore the HMA along this joint was found to be more susceptible to stripping/raveling. Consideration should be given to sealing this joint to limit the water coming into contact with the HMA layer (Vandenbossche 2003).

Repairs were made on three of the six Mn/ROAD test sections on June 20, 2001, after over 4.7 million ESALs. The repairs were made to 13 different areas in the ultrathin whitetopping test sections. In the section with a 3-in overlay and 5-ft by 5-ft joint spacing, four panels were repaired (two locations). Eighteen panels were repaired (six locations) in the section with a 3-in overlay and 4-ft by 4-ft joint spacing. Nineteen panels were repaired (five locations) in the section with a 4-in overlay and 4-ft by 4-ft panels. A detailed description on the repair techniques used was provided by Vandenbossche and Fagerness (2002).

Current Project Status, Results and Findings

The test sections on US 169 were in service between September 1997 and September 1999. During that period, the sections accumulated approximately 670,000 equivalent single-axle loads (assuming a 152.4-mm (6-in.) portland cement concrete pavement and a terminal serviceability of 2.5). Additional information on the performance of these test sections can be found in the report by Vandenbossche (2002).

Three of the test sections on I-94 will be reconstructed in the summer of 2004. The new test sections will consist of a 127-mm (5-in.) and a 101.6-mm (4-in.) overlay with 1.52-m x 1.83-m (5-ft x 6-ft) panels. Half of each section will have sealed joints and the joints in the other half of each test section will remain unsealed. The monitoring activities will be similar to that for the original whitetopping test sections. The new test sections will be instrumented with temperature, moisture, and static and dynamic strain gauges.

Point of Contact

Thomas Burnham
1400 Gervais Ave.
Maplewood, MN 55109
(651) 779-5606
Tom.Burnham@dot.state.mn.us

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Chapter 24. MISSISSIPPI 1 (US 72, Corinth)

Introduction

The Mississippi Department of Transportation (MDOT) constructed an experimental project in 2001 under the TE-30 program to investigate the performance of a resin-modified pavement (RMP). RMPs are a new composite paving material consisting of a thin layer (50 mm [2 in.]) of open-graded hot mix asphalt (HMA) whose internal voids (approximately 30 percent) are filled with a latex rubber-modified portland cement grout (MDOT 1999). RMPs provide a tough and durable pavement surface that resists rutting, surface abrasion, and deterioration due to fuel spillage (Anderton 1996). To date, they have been used almost exclusively on military bases, but current research suggests they are suitable for any low-speed traffic application where resistance to heavy loads, tracked vehicle traffic, or fuel spillage is required (Anderton 1996). The cost of an RMP is typically about 50 to 80 percent higher than a comparable HMA pavement, but about 30 to 60 percent less than a comparable PCC pavement design (Anderton 1996).

To evaluate the performance of RMP, the MDOT constructed a series of test sections under the TE-30 program on US 72 in Corinth (see Figure 68) in 2001. These test sections were constructed at two HMA pavement intersections on US 72 that have a history of rutting and high traffic loading. For comparison purposes, an alternative pavement was constructed at each intersection adjacent to the RMP but in the opposite traffic direction. One of the alternative pavements was an ultrathin whitetopping (UTW) design, the other was a polymer-modified HMA pavement.

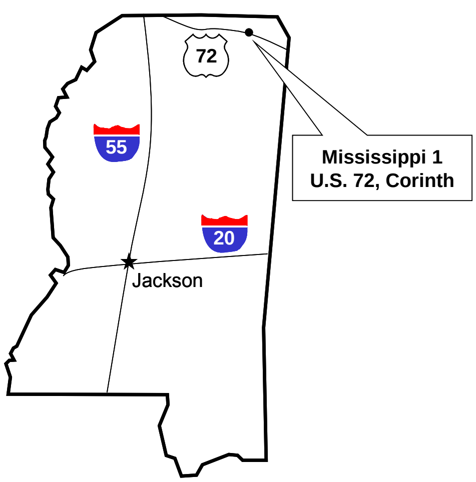

Figure 68. Location of MS 1 project.

Study Objectives

The objective of this project is to construct a demonstration RMP highway project and compare its performance to that of a UTW overlay and a polymer-modified HMA overlay.

Project Design and Layout

The construction of this experimental project took place from April to June of 2001. A total of four test sections were constructed at two different intersections on US 72 in Corinth:

- Intersection of US 72 and Hinton Street, South Parkway, and Liddon Lake Road.
  - RMP in eastbound lanes.
  - UTW overlay in westbound lanes.
- Intersection of US 72 and Cass Street.
  - RMP in eastbound lanes.
  - Polymer-modified HMA overlay in westbound lanes.

Figure 69 shows the layout of the test sections at the intersection of US 72 and Hinton Street, South Parkway, and Liddon Lake Road, while Figure 70 shows the layout of the test sections at the intersection of US 72 and Cass Street.

The existing HMA pavement varied in thickness from 250 to 350 mm (10 to 14 in.), and was exhibiting rutting typically between 25 and 38 mm (1 and 1.5 in.). This portion of US 72 carried approximately 22,600 vehicles per day in 2000 (including about 10 percent heavy trucks). The projected cumulative

KESALs in one direction for 20-year design life are 11,257 and 7,257 for rigid and flexible pavements, respectively.

At both intersection locations, the RMP was constructed 50 mm (2 in.) thick. The existing HMA pavement was first milled to a depth of 50 mm (2 in.), after which a 50-mm (2-in.) lift of open-graded HMA (PG 67-22) was placed. The cement grout containing a resin additive PL-7 was applied to the open-graded HMA the following day.

A 5000 lbf/in^2 air-entrained concrete mix containing fibrillated polypropylene fibers was utilized for the UTW that was placed at the intersection of US 72 and Hinton Street. The existing HMA pavement was first milled to a depth of 75 mm (3 in.). The 75-mm (3-in.) UTW was then placed, with green sawing of the slabs conducted using Soff-Cut saws. The UTW was sawed into 0.9-m (3-ft) square slabs.

The polymer-modified HMA overlay was placed at the US 72 and Cass Street intersection. The existing HMA pavement was first milled to a depth of 100 mm (4 in.). The polymer-modified HMA with a 12.5 mm (0.5 in.) nominal maximum aggregate size (NMAS) and a PG 82-22 asphalt binder was then placed in two 50-mm (2-in.) lifts.

The excess milling of the existing pavement during the construction increased the unit costs of UTM and RMP sections. The unit cost of RMP sections was further increased by the removal of a portion of the sections due to the lack of full-depth grout penetration. The estimated and actual unit costs for these overlay treatments are shown in Figure 71.

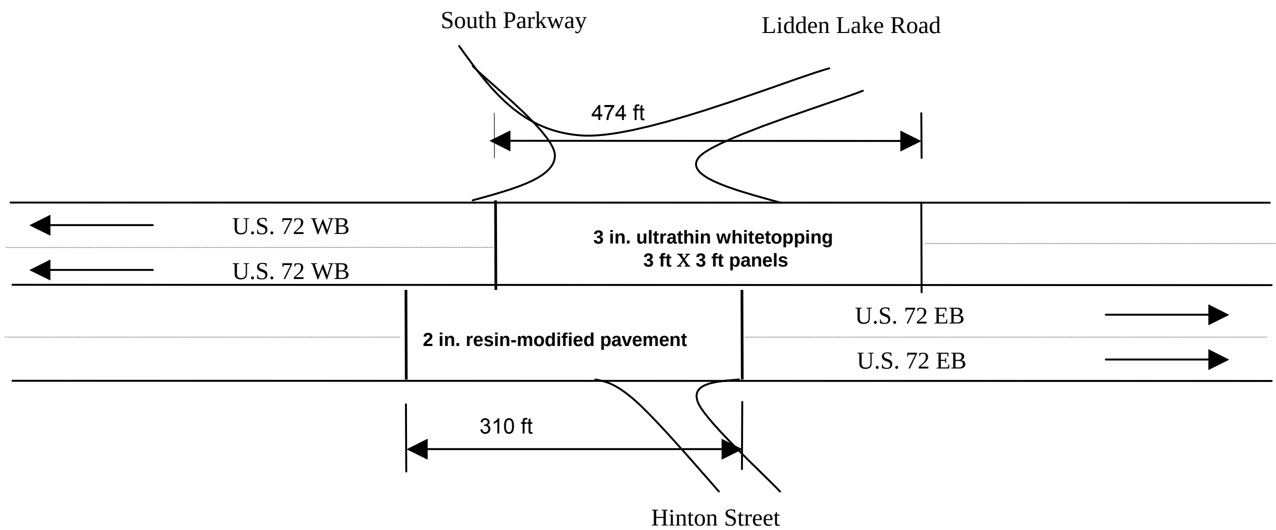

Figure 69. Layout of test sections at the intersection of US 72 and Hinton Street, South Parkway, and Liddon Lake Road.

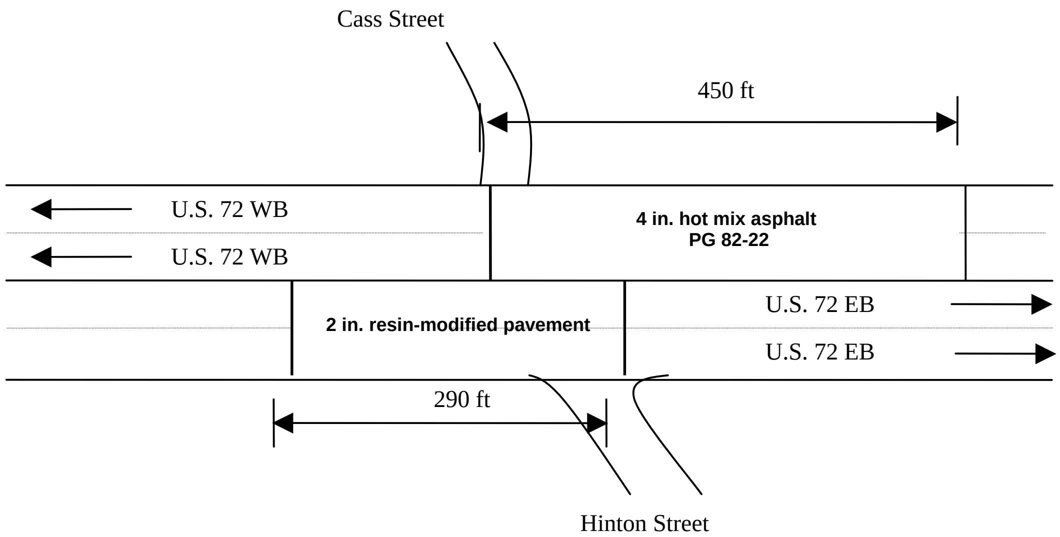

Figure 70. Layout of test sections at the intersection of US 72 and Cass Street.

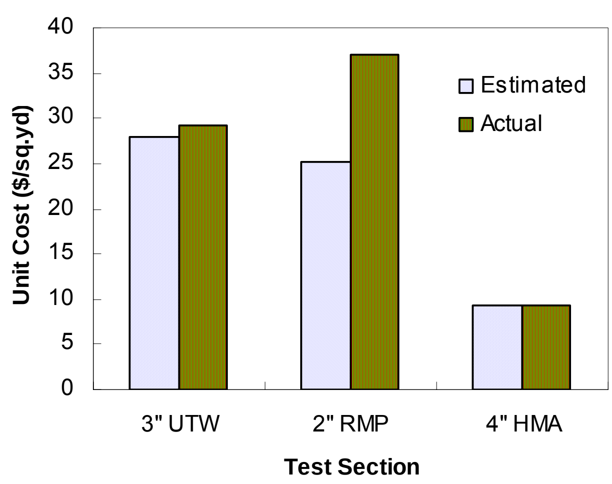

Figure 71. Relative unit costs of the test sections.

State Monitoring Activities

Construction Monitoring

During the construction of test sections, several incidents were observed by the researchers that might affect long-term pavement performance. These are summarized below (Battey 2002):

- After the inside lanes of both RMP sections were constructed, the temperature in Corinth dropped to 32 ^oF during the night. The severe cold slowed down the curing of the grout to such an extent that it appeared to be "powdering up" on the surface. However, the next day as the temperature warmed up, the powdering condition was not evident and the grout appeared to be gaining strength.

- During the construction of the right lane of the HMA section, 10 vehicles drove on the fresh asphalt mat before the breakdown roller could begin compacting the section. The rutting due to the early traffic averaged 2.2 mm (0.09 in.) in the left wheelpath and 1 mm (0.04 in.) in the right wheelpath.

Performance Monitoring

MDOT will monitor the performance of these test sections for a period of 5 years. Performance data collected include Profile Index (PI), International Roughness Index (IRI), and skid number. Rutting data are also being collected for the HMA section.

Preliminary Results/Findings

Some preliminary performance data collected under the monitoring program are summarized in Table 37 (a–d). The monitoring of the construction and performance of these test sections has led to the following conclusions and recommendations (Battey 2002):

- Table 37 indicates that the smoothness of the UTW test section is less than satisfactory, suggesting the need for smoothness incentive provisions.

- In the construction of RMP, the gradation of the open-graded asphalt mix must be carefully controlled to ensure the target 30 percent air void level is obtained. Sufficient curing time (no less than 72 hours of above 50 ^oF temperature) should be provided for the grout to obtain its design compressive strength.

- The initial skid resistance of the RMP sections is less than satisfactory. However, as traffic begins to wear the top film of grout off the sections, the skid numbers begin to improve to more acceptable levels.

Table 37(a). International Roughness Index (IRI) Measured on the MS 1 Test Sections (mm/km)

DATE	ULTRATHIN WHITETOPPING	RESIN-MODIFIED PAVEMENT AT CASS	AT HINTON	HOT MIX ASPHALT
April 2001	5.42*	2.04*	2.36*	
July 2001	4.75**	1.78**	2.12**	1.41**
August 2001	4.54**	1.76**	2.01**	1.32**

* IRI was collected in the right wheelpath using MDOT's ARRB Transport Research Walking Profiler.
** IRI is the average IRI of both wheelpaths collected using MDOT's High Speed "South Dakota Type" Profiler.

Table 37(b). Profile Index Measured on the MS 1 Test Sections (in./mi)

DATE	ULTRATHIN WHITETOPPING $PI_{0.2"}$	ULTRATHIN WHITETOPPING $PI_{0.0}$	RESIN-MODIFIED PAVEMENT AT CASS $PI_{0.2"}$	RESIN-MODIFIED PAVEMENT AT CASS $PI_{0.0}$	RESIN-MODIFIED PAVEMENT AT HINTON $PI_{0.2"}$	RESIN-MODIFIED PAVEMENT AT HINTON $PI_{0.0}$	HOT MIX ASPHALT $PI_{0.2"}$	HOT MIX ASPHALT $PI_{0.0}$
April 2001	100.03	148.28	11.42	27.8	23.53	59.21		
June 2001							16.03	35.24

PI was collected in the right wheelpath using a "California Type" Profilograph, and analyzed with both 0 and 0.2 in. blanking bands.

Table 37(c). Friction Numbers Measured on the MS 1 Test Sections

DATE	ULTRATHIN WHITETOPPING	RESIN-MODIFIED PAVEMENT AT CASS	RESIN-MODIFIED PAVEMENT AT HINTON	HOT MIX ASPHALT
April 2001	36.8			
May 2001	33.2	24.4	23.4	
July 2001	33.3	29.8	31.8	35.8
August 2001	37.8	37.1	38.0	36.9
December 2001	44.2	38.7	39.4	34.9

Skid resistance tests were conducted in the outside lane of each section with a test speed of 40 mi/hr.

Table 37(d). Rutting Measured in the MS 1 HMA Test Section (in.)

DATE	RUTTING (IN.) LEFT WHEELPATH	RUTTING (IN.) RIGHT WHEELPATH
July 2001	0.09	0.04
December 2001	0.13	0.08

Point of Contact

Randy Battey
Mississippi Department of Transportation
P. O. Box 1850
Jackson, MS 39215-1850
(601) 359-7648

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Chapter 25. MISSOURI 1 (I-29, Rock Port)

Introduction

As part of the TE-30 program, the Missouri Department of Transportation (MoDOT) constructed an experimental unbonded concrete overlay in 1998. Located in the southbound lanes of I-29, just west of Rock Port (see Figure 72), the overlay includes test sections of conventional unbonded overlays, steel fiber-reinforced overlays, and polyolefin fiber-reinforced overlays, all with varying thicknesses (MoDOT 2000).

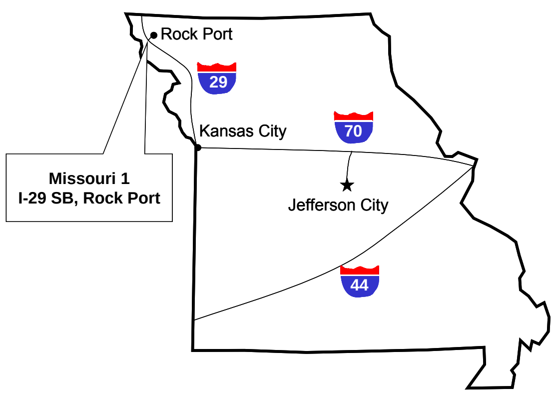

Figure 72. Location of MO 1 project.

Study Objectives

The overall objective of this study is to compare the performance of fiber-reinforced unbonded concrete overlays to that of conventional unbonded concrete overlays (MoDOT 2000). The addition of the fibers to the concrete mix is expected to increase the service life of the overlay by increasing the "toughness" and post-cracking behavior of the concrete (Mindess and Young 1981). Toughness is defined as the total energy required to break a specimen, and the addition of fibers gives concrete a considerable amount of apparent ductility before ultimate failure of the specimen (Mindess and Young 1981).

Project Design and Layout

This project, constructed in 1998, consists of eight test sections, each 762 m (2500 ft) long (MoDOT 2000). Of the eight test sections, three sections incorporate steel fibers in the concrete mix, three sections incorporate polyolefin fibers in the mix, and two sections use a conventional concrete mixture. The layout of these test sections is illustrated in Figure 73 (MoDOT 2000).

All of the test sections are unbonded concrete overlays placed over an existing 229-mm (9-in.) jointed reinforced concrete pavement (JRCP) that has 18.7-m (61.5-ft) transverse joint spacing (MoDOT 2000). A 25-mm (1-in.) asphaltic interlayer treated with white curing compound was used to isolate the concrete overlays from the underlying pavement (MoDOT 2000).

Based on a laboratory evaluation of fiber-reinforced mixes conducted by the DOT, and on the recommendations of the fiber manufacturers, two fibers were selected for the I-29 project (MoDOT 2000):

- 50-mm (2-in.) 3M polyolefin fibers. The fibers are straight with an aspect ratio (length/diameter) of 79 and were applied at a dosage rate of 14.8 kg/m^3 (25 lb/yd^3).
- 60-mm (2.4-in.) Dramix steel fibers. These fibers have hooked ends to promote bonding, an aspect ratio of 75, and were applied at a dosage rate of 44.5 kg/m^3 (75 lb/yd^3).

Each of the fiber-reinforced sections are differentiated by slab thickness, with thicknesses of 229 mm (9 in.), 152 mm (6 in.), and 127 mm (5 in.) included in the study. Furthermore, within each fiber-reinforced test section, four subsections with variable joint spacings (4.6 m [15 ft], 9.1 m [30 ft], 18.3 m [60 ft], and 61 m [200 ft]) are also included. The two control sections for the project are a 229-mm (9-in.) unbonded JPCP with 4.6-m (15-ft) transverse joints and a 279-mm (11-in.) unbonded JPCP with 4.6-m (15-ft) transverse joints. The experimental design matrix for the project is provided in Table 38.

Paraffin-treated, epoxy-coated steel dowel bars were included in all test sections. The 279-mm (11-in.) test section contained 38-mm (1.5-in.) diameter bars whereas the rest of the test sections contained 32-mm (1.25-in.) diameter bars. Transverse joints were sealed with a hot-poured elastic sealant.

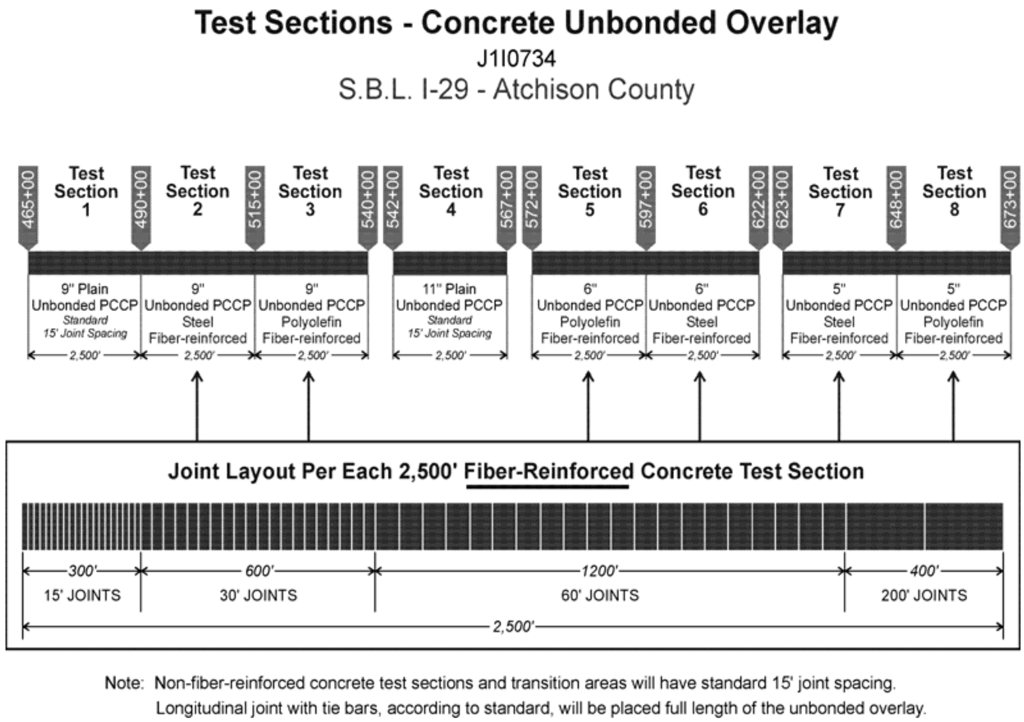

Figure 73. Layout of MO 1 test sections (MoDOT 2000).

Table 38. Experimental Design Matrix for MO 1

		UNBONDED OVERLAY PORTLAND CONCRETE CEMENT		
		CONVENTIONAL	**Steel Fiber**	**Polyolefin Fiber**
5 in. Slab Thickness	**15-ft Joint Spacing**		Section 7	Section 8
	30-ft Joint Spacing		Section 7	Section 8
	60-ft Joint Spacing		Section 7	Section 8
	200-ft Joint Spacing		Section 7	Section 8
6 in. Slab Thickness	**15-ft Joint Spacing**		Section 6	Section 5
	30-ft Joint Spacing		Section 6	Section 5
	60-ft Joint Spacing		Section 6	Section 5
	200-ft Joint Spacing		Section 6	Section 5
9 in. Slab Thickness	**15-ft Joint Spacing**	Section 1	Section 2	Section 3
	30-ft Joint Spacing		Section 2	Section 3
	60-ft Joint Spacing		Section 2	Section 3
	200-ft Joint Spacing		Section 2	Section 3
11 in. Slab Thickness	**15-ft Joint Spacing**	Section 4		

State Monitoring Activities

MoDOT is monitoring the performance of these sections annually for a minimum of 5 years, with additional monitoring thereafter conducted as appropriate. Data collection activities include pavement distress surveys, roughness measurements, surface friction testing, and FWD testing.

During construction, the properties of the materials were monitored. The fiber-reinforced concrete overlay mix had a w/c of 0.39 and utilized a limestone coarse aggregate with a 13 mm (0.5 in.) top size (MoDOT 2000). Nonuniform distribution of the fibers was observed, particularly for the polyolefin fibers (MoDOT 2000). Mixing times were increased, batch sizes were decreased, and the order of mixer loading was altered to address this concern, and these seemed to increase the uniformity of the fiber distribution in the concrete (MoDOT 2000).

Initial finishing of the overlays used a burlap drag, but this was later changed to an unweighted carpet drag because it was found that the fibers became caught in the burlap such that some fibers and aggregate were pulled from the top layer of the overlay (MoDOT 2000). In lieu of the conventional transverse tining texturing method, the final surface texture was established by diamond grinding the overlay 21 days after construction for smoothness and rideability (MoDOT 2000). Following grinding, profilograph readings averaged less than 0.17 m/km (11 in./mi) (0 blanking band), resulting in a smoothness bonus for the contractor (MoDOT 2000).

The in-place construction costs for these pavement sections are shown in Figure 74 (MoDOT 2000). This figure shows that the initial cost of the fiber-reinforced sections is higher than the cost of the conventional sections (the cost of furnishing the steel fiber concrete and the polyolefin fiber concrete was \$56.22 and \$71.77 more per m^2 [\$47 and \$60 more per yd^2], respectively, than the conventional concrete).

Preliminary Results/Findings

Preliminary results/findings are based on the first 2 years of performance monitoring of these test sections. After nearly 2 years of service, the overall performance of these sections was good, although a few of the sections performed poorly (MoDOT 2000). In particular, the thin 127-mm (5-in.) sections, both steel and polyolefin reinforced, exhibited a large amount of transverse cracking. In addition, the 152-mm (6-in.) steel fiber-reinforced section also showed significant transverse cracking. Figure 75 summarizes the cracking data collected up to 1999 on these test sections (MoDOT 2000).

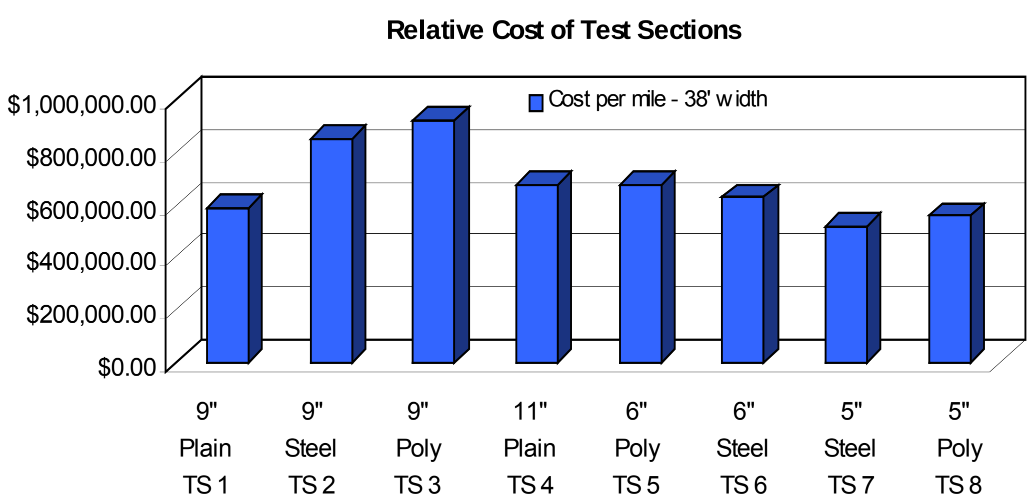

Figure 74. Relative cost of MO 1 test sections (MoDOT 2000).

Most of the transverse cracks that had developed were not located above joints or cracks in the existing pavement, so they did not appear to be reflective cracks (MoDOT 2000). In fact, most of the cracks on the thin steel fiber-reinforced sections were parallel to and located within 0.3 m (1 ft) of the transverse joints, whereas the cracks in the thin polyolefin fiber-reinforced sections were located away from the joints near mid-panel (MoDOT 2000). Because of the problems of the cracking and subsequent spalling, the test sections 7 and 8 were replaced with full-depth concrete in 2000.

Four general conclusions are drawn from the performance data collected up to 1999 (MoDOT 2000):

- The steel fiber-reinforced test sections exhibited more transverse cracking than the polyolefin fiber-reinforced test sections.
- The longer panels exhibited more cracking than the short panels.
- The thinner overlay sections exhibited more cracking than the thicker sections.
- Cracks that had developed in the steel fiber-reinforced test sections were tighter than those in the polyolefin fiber-reinforced test sections (3 mm [0.12 in.] vs. 6 mm [0.25 in.]).

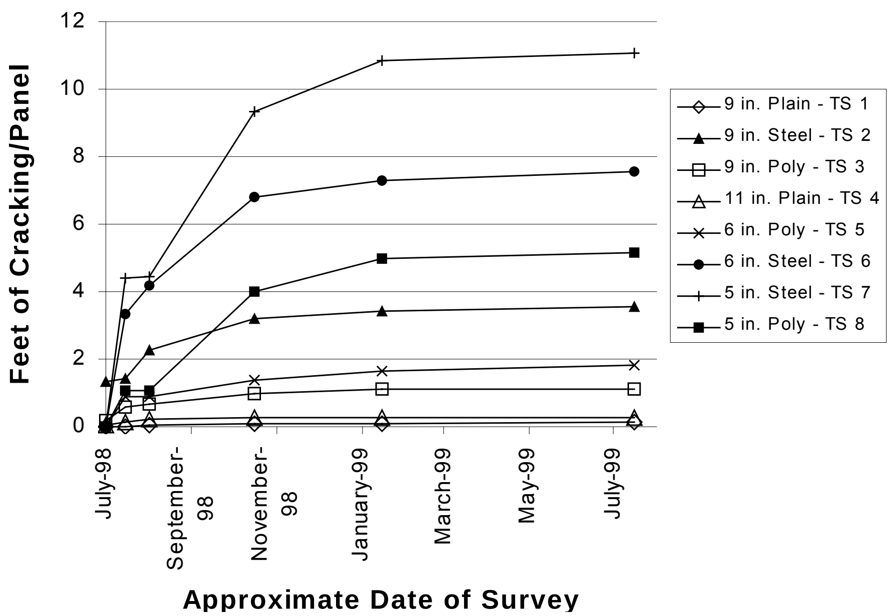

Figure 75. Transverse cracking on MO 1 test sections (MoDOT 2000).

Interim Results/Findings

Although no formal reports have been developed since the 2000 summary, MoDOT has provided additional performance data for inclusion in this report (Chojnacki 2004). Cracking surveys were conducted in December 2003 on these test sections, and the results are shown in Figure 76 (Chojnacki 2004). These data have been combined with the previous data to produce Figure 77.

In 2003, joint repairs were performed at several locations where transverse cracks existed near joints and spalling had occurred. The deteriorated areas were replaced with a full-lane-width concrete patch at least 1.8 m (6 ft) long. The patches were tied at one end with 19-mm (0.75-in.) epoxy-coated tie bars and doweled at the other end with 19-mm (0.75-in.) epoxy-coated dowel bars (Chojnacki 2004).

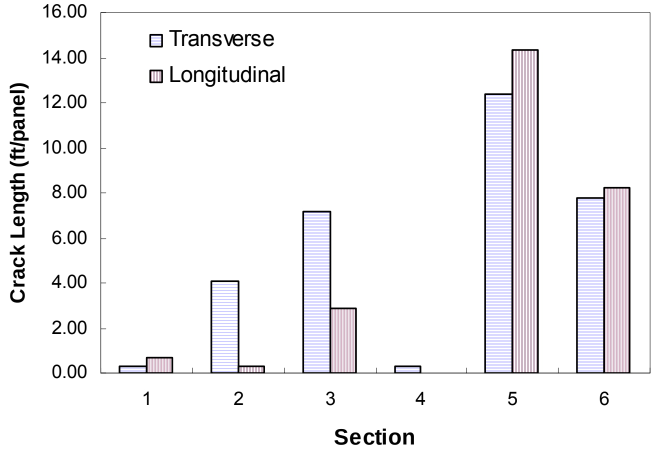

Figure 76. Transverse and longitudinal cracking on MO 1 test sections (Chojnacki 2004).

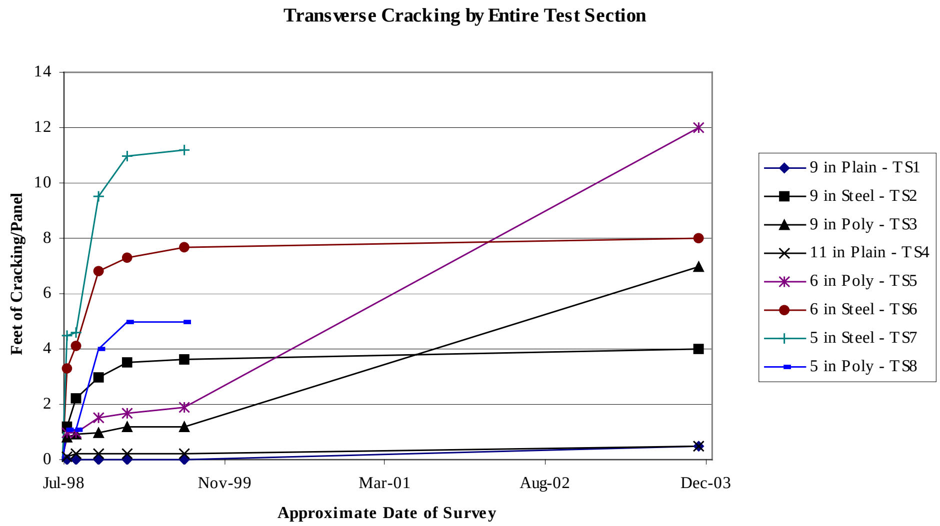

Figure 77. Transverse cracking on MO 1 test sections.

Point of Contact

Tim Chojnacki
Missouri Department of Transportation
1617 Missouri Boulevard
P.O. Box 270
Jefferson City, MO 65102
(573) 751-1040
Timothy.Chojnacki@modot.mo.gov

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Chapter 26. NEW HAMPSHIRE HPCP PROJECT

Introduction

Under the TE-30 program, the University of New Hampshire, in conjunction with the New Hampshire Department of Transportation, developed definitions of and showcase presentations for high-performance concrete in both bridge and pavement applications. A CD-ROM was produced containing the definitions as well as the presentation materials and accompanying speaker notes (Goodspeed 1999). On the pavement applications side, a demonstration computer program allowing the optimization of several concrete pavement variables was also developed.

Study Objectives

The objectives of this project include the development of performance classifications for high-performance concrete pavement (HPCP), the dissemination of information on HPCP through several showcase presentations, and the development of computer program that develops optimized pavement designs based on performance and variable constraints.

Results/Findings

HPCP Definitions

To aid in the development of high-performance concrete pavement designs, three HPCP classifications were developed. These classifications provide recommended values or ranges for different slab, base, and subgrade properties, and also maximum limits on various pavement performance parameters. The concept is that pavements subjected to higher traffic loadings or designed for longer service lives require stronger, more durable materials and more stringent performance requirements. Table 39 summarizes the recommended performance characteristics for three different HPCP classifications (Goodspeed 1999). It is noted that some of the performance characteristics are not defined, and also that the criteria for several of the design characteristics (e.g., permeability and ability and friction number) are not compatible with the selected performance classification.

Optimization Computer Program

The High Performance Concrete Pavement Optimization Program is available for use by State highway agencies (Goodspeed 1999). The computer program allows for one or more key design variables to be selected for optimization subject to certain constraints. Generally, slab thickness is the design variable that will be optimized, although other design variables, such as joint spacing or PCC compressive strength, can also be selected for optimization.

When optimizing, acceptable ranges must be entered for each variable being optimized. Furthermore, a cost equation must also be defined for each variable being optimized. The cost equation relates the cost of the variable to the design of the pavement system. Several default cost equations are provided for key variables, but users may also define their own unique cost equations (Goodspeed 1999).

Constraint parameters for various performance indicators must also be defined. These are generally equations that link the different design variables to pavement performance. Again, several default constraint parameters are contained in the program (for example, the 1993 AASHTO design procedure [AASHTO 1993] and the 1998 AASHTO Supplement [AASHTO 1998]), but users may define their own unique performance equations (Goodspeed 1999).

After defining the optimization variables and the constraint parameters, the user runs the optimization program to obtain the cost result and the values associated for each optimization variable. A summary report window is generated that summarizes the selected constraint equations, the optimization variable limits, the parameters, the cost equation results, and the optimized variable results (Goodspeed 1999).

Table 39. Summary of Design Characteristics for HPCP Classifications (Goodspeed 1999)

DESIGN CHARACTERISTICS	RECOMMENDED RANGE OR VALUE		
	PERFORMANCE CLASSIFICATION 1	PERFORMANCE CLASSIFICATION 2	PERFORMANCE CLASSIFICATION 3
System Parameters			
ESAL applications, millions	50–150	> 150	
Maximum bearing stress, lbf/in^2	2	4	6
Limiting IRI rating	1.5	1.75	
Limiting faulting, in./mi	20	15	10
Limiting transverse cracks/mi	30	20	
Design life, years	20	30	> 40
Design reliability, %	75–85	85–95	
Terminal serviceability index	2.50	3.0	
PCC Slab Materials			
Modulus of rupture, lbf/in^2	500–650	650–700	> 700
Elastic modulus, lbf/in^2	25–50	50–100	> 100
Freeze-thaw (ASTM C666), %	60–80	> 80	
Scaling (ASTM C672)	x = 4, 5	x = 2, 3	x = 0, 1
Abrasion (ASTM C944-90a)	1–2	0.5–1	< 0.5
Permeability (ASTM C1202), coulombs	1000	2000	3000
Coarse aggregate (AASHTO M80-87), class	D	C	B
Fine aggregate (AASHTO M6-93), class	B	A	
Slab Constructibility			
Fast track, hours to 3000 lbf/in^2	< 24	< 18	< 12
Slab Performance			
Mean friction number (ASTM E 274)	40–50	35–40	30–35
Initial smoothness (profilograph), in./mi	9	7	6
Texture (ASTM E965), in.	0.13	0.13–0.25	0.25–0.30
Joint Materials			
Rubberized asphalt (ASTM D1190, D3405), %	15–20	20–30	> 30
Low-Modulus RA (ASTM D3405), %	30–40	40–50	> 50
Nonself-leveling silicone (ASTM D3893), %	30–40	40–50	> 50
Self-leveling silicone (ASTM D3893), %	30–40	40–50	> 50
Preformed compression seal (ASTM D2628), %	45–65	65–85	> 85
Joint Constructibility			
Load transfer coefficient, J	3.0–3.2	2.8–3.0	< 2.8
Joint Performance			
Joint faulting, in.	< 0.13	< 0.10	
Base Materials			
Liquid limit (AASHTO T89)	25–28	28–32	> 32
Plastic limit (AASHTO T90)	< 10	< 20	
Abrasion (ASTM C131)	50 max		
Drainage coefficient, C_d	0.9–1.1	1.1–1.2	> 1.2
Stabilized base k-value, lbf/in^2/in.	100–200	200–300	> 300
Subgrade k-value, lbf/in^2/in.	100	150–200	200–250
Subgrade CBR, %	60–70	70–80	> 80
Base Constructibility			
In-place recycling, %	< 10	< 20	< 30
Speed, ft/day	< 500	< 1500	< 3000
Base Performance			
Erosion resistance (CTB), %	3–5	5–7	7–8
Friction coefficient, f	0.9–1.2	1.2–2.0	> 2.0

Program Availability

The software program is currently available and free upon request. The software can be requested from either the New Hampshire Technology Transfer (T^2) Center or the Florida T^2 Center:

Charles Goodspeed
University of New Hampshire
Kingsbury Hall
Room 236
Durham, NH 03824
(603) 862-1443
chgi@cisunx.unh.edu

Gib Peaslee
University of Florida
Civil Engineering Department
Transportation Research Center
P.O. Box 116585
512 Wiel Hall
Gainesville, FL 32611-6585
(352) 392-2371, ext 245
gib@ce.ufl.edu

Point of Contact

Charles H. Goodspeed
University of New Hampshire
Kingsbury Hall, Room 236
Durham, NH 03824
(603) 862-1443
chgi@cisunx.unh.edu

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Chapter 27. OHIO 1, 2, AND 3 (US Route 50, Athens)

Introduction

Under the TE-30 program, the Ohio Department of Transportation (ODOT) constructed three experimental pavement projects on US 50, approximately 8 km (5 mi) east of the city of Athens (see Figure 78). The projects incorporate a variety of experimental design features, including high-performance concrete mixtures utilizing ground granulated blast furnace slag (GGBFS) (OH 1), alternative dowel bar materials (OH 2), and alternative joint sealing materials (OH 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.

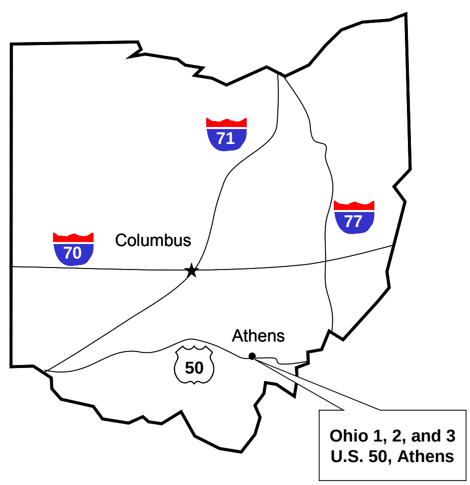

Figure 78. Location of OH 1, 2, and 3 projects.

Study Objectives

The study objectives for the overall US 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 the achievement of 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 US 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 40 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).

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).

Table 40. Comparison of Actual Base and Subbase Gradations Used on Ohio US 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

Project Layout Information

As described previously, the US 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 79. More detailed information on each project is provided in the following sections.

OH 1, Evaluation of Ground Granulated Blast Furnace Slag

The entire 10.5-km (6.5-mi) length of the US 50 project was constructed using a high-performance concrete mix consisting 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 41.

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 (type 304) 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.

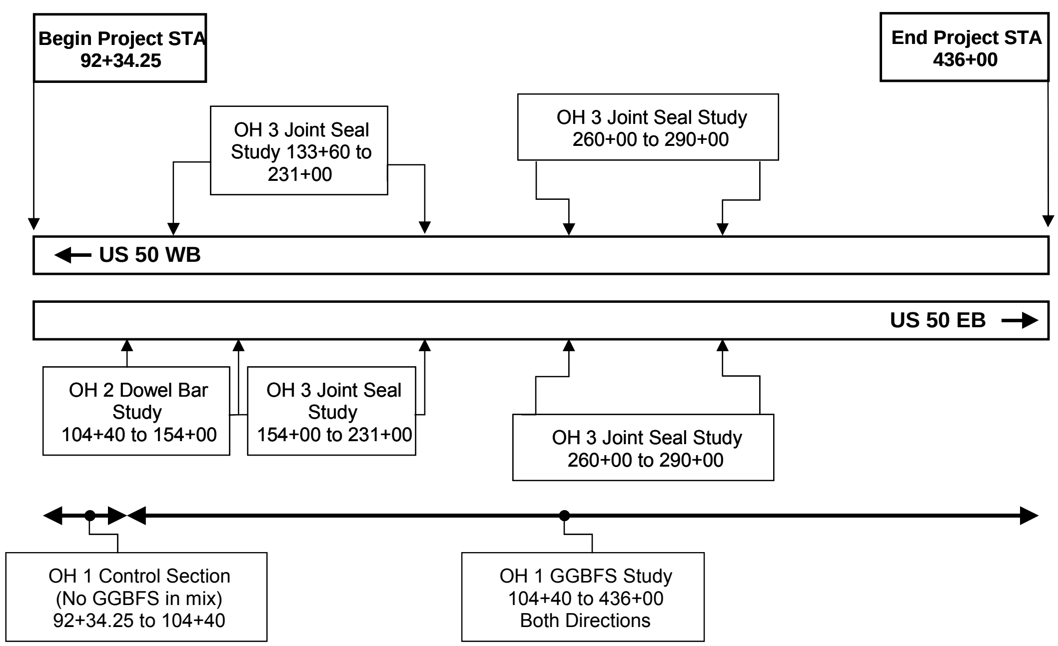

Figure 79. Layout of experimental projects on Ohio US 50.

Table 41. Concrete Pavement Mix Design Used on Ohio US 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
Ground granulated blast furnace slag	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.

Most of the US 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 80). 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 gauges (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 81 (Sargand 2000).

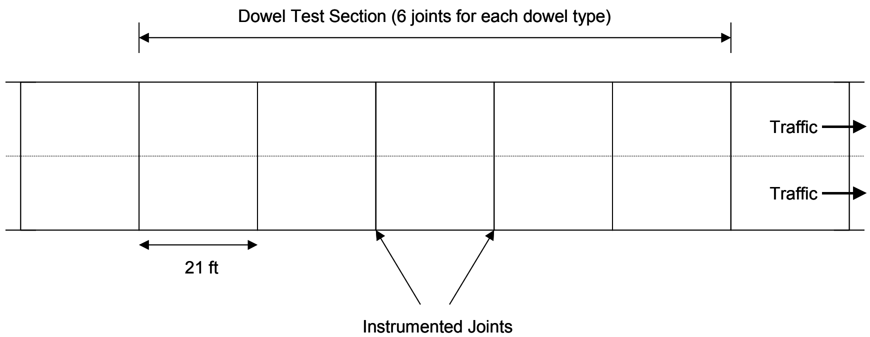

Figure 80. Layout of dowel test sections on Ohio US 50 project.

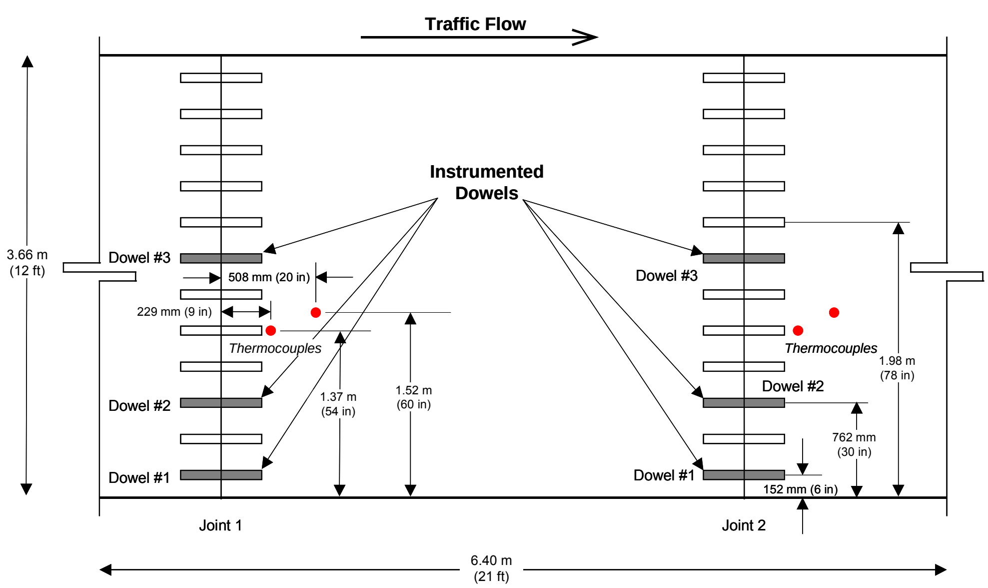

Figure 81. Dowel instrumentation layout for Ohio US 50 project (Sargand 2000).

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).

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, and the second unit consists of a single sensor measuring temperatures at a depth of 25 mm (1 in.) below the surface (Sargand 2000).

OH 3, Evaluation of Joint Sealing Materials

The joint sealant test sections are located in both the eastbound and westbound directions, and feature a total of nine different joint sealants. In addition, several pavement sections containing no sealant are included in the study.

Table 42 summarizes the location of the different sealant materials in each direction, as well as the joint channel configuration (see Figure 82) 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.

Results/Findings

Overall pavement performance on this project has been mixed, and may be related to the small top size of the coarse aggregate, the small percentage of reinforcing steel, and the poor support from the nonstabilized bases. Specific findings for each specific study are presented in the following sections.

OH 1, Evaluation of Ground Granulated Blast Furnace Slag

The final report by Sargand, Edwards, and Khoury (2002) provides the results for 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 ^{o}C, and developed cracking within 18 hours of placement. One HPC and one standard pavement section placed in October 1998 experienced gradients of only 5 ^{o}C, and did not develop cracking. The higher temperature gradient in 1997 resulted from a cold front that moved in shortly after the placement of the concrete.

- Large values of strain recorded with the vibrating wire strain gauges 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 1 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.

Table 42. Sealant Materials Used in Joint Sealant Study on Ohio US 50 Project (Hawkins, Ioannides, and Minkarah 2000)

SEALANT MATERIAL	SEALANT TYPE	BEGIN STATION	END STATION	JOINT CONFIGURA-TION	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

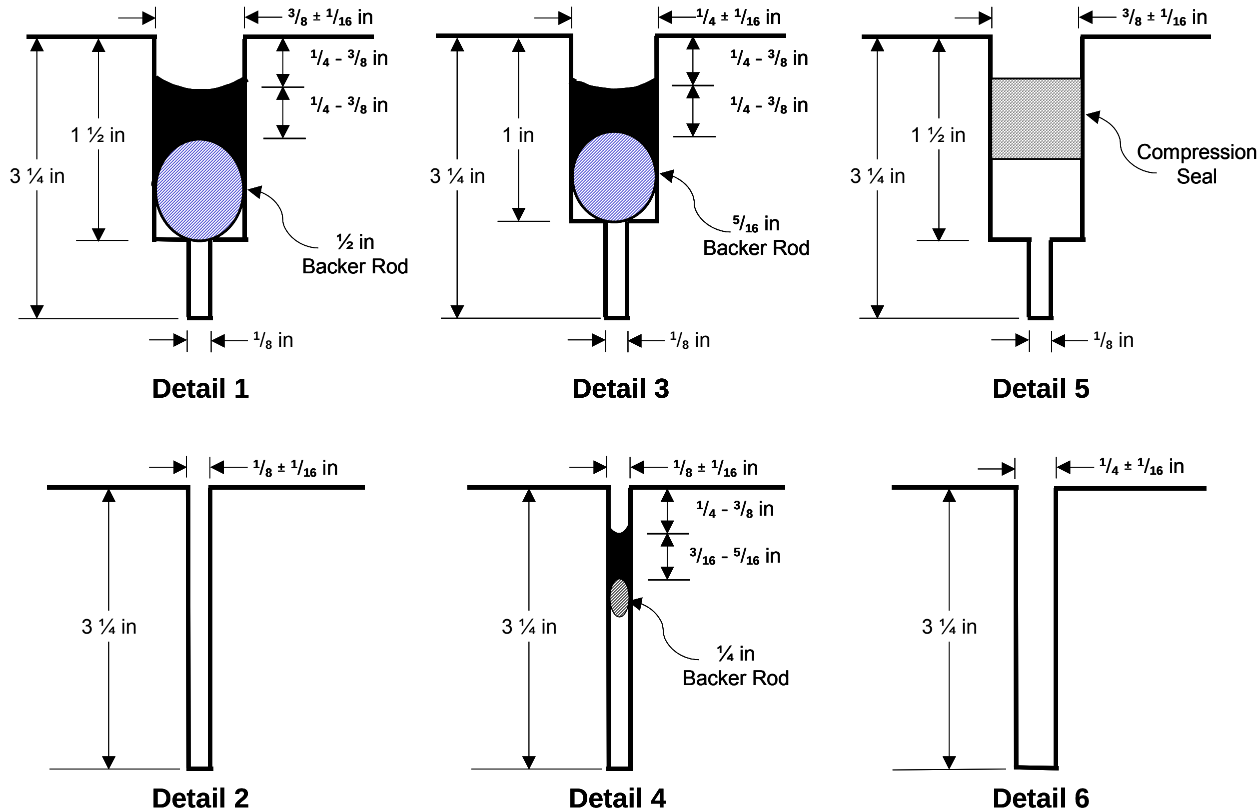

Figure 82. Joint channel configurations used in sealant study on Ohio US 50 project (Hawkins, Ioannides, and Minkarah 2000).

- 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, Edwards, and Khoury 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 HP 3 had less initial warping than did section SP constructed with standard ODOT class C concrete.

- FWD data indicated that, under similar loading conditions, the HP 3 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 testing, 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 1997) and WB (constructed in 1998). 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 43 (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 43. Summary of Sealant Costs on Ohio US 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

Points of Contact

Roger Green
Ohio Department of Transportation
Office of Pavement Engineering
1980 West Broad Street
Columbus, OH 43223
(614) 995-5993

Anastasios Ioannides
University of Cincinnati
Department of Civil and
Environmental Engineering
741 Baldwin Hall (ML-0071)
P.O. Box 210071
Cincinnati, OH 45221-0071
(513) 556-3137

Shad Sargand
Ohio University
Ohio Research Institute for
Transportation and the Environment
Department of Civil and
Environmental Engineering
Stocker Center
Athens, OH 45701
(740) 593-1467

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Chapter 28. Ohio 4 (US 35, Jamestown)

Introduction

The Ohio Research Institute for Transportation and the Environment at the Department of Civil Engineering at Ohio University, in conjunction with the Ohio Department of Transportation, have conducted extensive pavement research activities during the past decade. As a part of the TE-30 program, this study has undertaken a comparative evaluation of the available nondestructive test devices for measuring the support of subgrade and aggregate base layers of pavement sections. The study considers the means and results of data measurements of various testing devices, including their application, and ease of use. The test section selected for this project is located on US 35 in Jamestown, Ohio (see Figure 83).

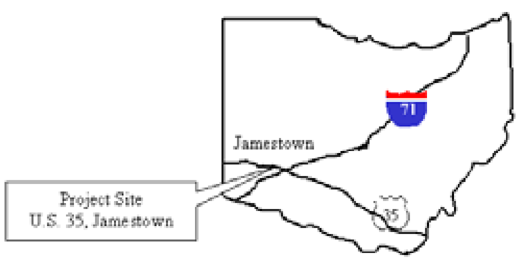

Figure 83. Location of OH 4 project.

Study Objectives

Variability in pavement support resulting from variations in subgrade and base layer support values result in significant variation in pavement performance. Deficiencies relative to design assumptions result in reduced pavement performance. Conventional laboratory testing has not been effective in capturing field variation. The use of nondestructive testing methods to assess pavement conditions and to predict pavement performance depends upon the quality and reliability of the data obtained.

The principal objective of the study is to measure the structural characteristics of the subgrade and base layers on a section of US 35 with various NDT devices during construction, and compare the output from the devices in the context of assessing structural conditions and variability. Nondestructive testing was performed using a nuclear density gauge, the Humboldt Stiffness Gauge, the German plate load device, a falling weight deflectometer, and a dynamic cone penetrometer.

Project Design and Layout

A 609.6-m (2000-ft) test section was selected in the eastbound lanes of a 8.5-km (5.3-mi) construction project on US 35 in Jamestown, Ohio. The test location was judged to have relatively uniform topographical and subsurface soil conditions. The pavement consists of a four-lane divided highway with two 3.6-m (12-ft) lanes, a 3-m (10-ft) outside shoulder and a 1.8-m (6-ft) inside shoulder. The pavement cross section consists of 228.6-mm (9-in.) JRCP, a 101.6-mm (4-in.) unstabilized drainable base, and a 152.4-mm (6-in.) dense-graded aggregate base, all over a prepared subgrade.

Subgrade samples were collected and identified as silty clay (AASHTO Classification A-6) with a liquid limit of 22.8 percent, a plastic limit of 16.7 percent, and a plasticity index of 6.1 percent. Laboratory resilient modulus tests were conducted in accordance with SHRP Protocol P-46.

State Monitoring Activities

Initial moisture and density data were collected along the centerline and right wheelpath of the eastbound driving lane at 15.2-m (50-ft) intervals with a nuclear density gauge. Other nondestructive testing was also typically conducted at intervals of 15.2 m (50 ft), except for the German plate load test, whose testing frequency was increased to 30.5 (100 ft) because of the time required to conduct each test.

Laboratory resilient modulus testing was performed on material samples from the project materials to provide a comparison of results. Tests were performed at several confining pressure levels including 20.7, 34.5, 68.9, 103.4, and 137.9 kPa (3, 5, 10, 15, and 20 lbf/in^{2}). The moisture content of the materials is also known to affect the results.

Results/Findings

Overall results indicated that each device has a useful function in evaluating subgrade and base uniformity conditions. The laboratory resilient modulus test is limited to materials sampled at specific designated locations. Additionally, the results are very much a function of test conditions. The level of confining pressure used during the testing was found to have an effect on the computed modulus values, as described by Sargand, Edwards, and Salimath (2001).

The nuclear density gauge is limited to a layer thickness measurement of 304.8 mm (12 in.), and greatly affected by nonuniformity within the layers tested. It is a quick means of controlling the uniformity of material density during construction. The density measurements recorded can be correlated with material stiffness.

The DCP is a quick automated field test method for evaluating the in situ stiffness of layers in a highway pavement structure. It measures the strength and stiffness of subgrade and unstabilized base layers. The DCP's ability to penetrate into underlying layers and accurately locate zones of weakness represent its greatest advantage over other tests considered. The automated device includes software for storing and reporting the collected data.

The Humboldt Gauge measures stiffness of the upper 152.4 mm (6 in.) of material by electrical impedance. In this respect, it is quite different from the other NDT devices considered in this study, as the other devices measure the composite response of the upper layer measured, and any supporting layers beneath. The Humboldt gauge was considered effective for monitoring the integrity of individual material layers as they are being placed.

The remaining devices identified significant pavement support variation along the length of the test. Because the Humboldt Gauge only measures to a depth of 152.4 mm (6 in.), its variation represented is much smaller than that indicated by the other devices. Both stiffness and calculated moduli values were evaluated for each device, with sample results provided in Table 44.

The large load FWD represented 2948.3- to 4082.3-kg (6500- to 9000-lb) loads, while the small load represented 1587.6- to 2041.2-kg (3500- to 4500-lb) loads.

Table 44. Stiffness and Modulus Sample Results from OH 4 Testing

Nondestructive Test	Stiffness, lbf/in.	Modulus, lbf/in^2
Subgrade (15 stations)		
Humboldt Gauge	88,758	18,750
FWD, Large load	249,703	22,610
FWD, Small load	210,785	19,090
German Plate	1st Cycle	
	131,889	11,960
	2nd Cycle	
	153,795	13,930
Composite Base (16 stations)		
Humboldt Gauge	129,730	27,410
FWD, Large load	252,747	36,220
FWD, Small load	257,114	40,970
German Plate	1st Cycle	
	67,793	16,870
	2nd Cycle	
	206,533	44,500

FWD = falling weight deflectometer

The FWD and the German plate load test are considered effective for measuring the total composite stiffness of the in situ pavement structures. Comparisons of the devices, and the Humboldt Gauge, are difficult since each generates load differently, to a different depth, and uses different equations to convert surface deflections to layer modulus. The dynamic loading applied by the FWD typically results in higher material stiffness than static loads used in the German plate test. The Humboldt Gauge produces small excitations, which limits its depth of effectiveness.

The FWD has a definite advantage over the plate load testing because of its speed of testing. The plate load test is much more labor intensive and requires more test time at a single location. The DCP is considered useful for identifying and locating the cause(s) of low stiffness identified with FWD results, which will likely cause premature failure within a pavement structure.

Points of Contact

Roger Green
Ohio Department of Transportation
Office of Pavement Engineering
1980 West Broad Street
Columbus, OH 43223
(614) 995-5993

Shad Sargand
Ohio University
Ohio Research Institute for
Transportation and the Environment
Department of Civil and
Environmental Engineering
Stocker Center
Athens, OH 45701
(740) 593-1467

Reference

Sargand, S. M., W. F. Edwards, and S. Salimath. 2001. *Evaluation of Soil Stiffness Via Non-destructive Testing*. Final Report. Ohio Department of Transportation, Columbus.

Chapter 29. PENNSYLVANIA 1 (SR 22, Murrysville)

Introduction

Pennsylvania's first project to be funded, in part, under the TE-30 program will be on Section B01 on SR 22 in Murrysville just east of Pittsburgh (see Figure 84). This project is unique in that is a cooperative effort with the public (Pennsylvania Department of Transportation [PENNDOT] and FHWA) and private sector (Mascaro Construction), as well as academia (University of Pittsburgh). The project will be constructed in the fall of 2004.

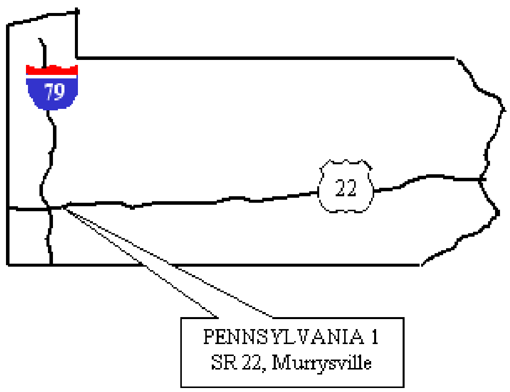

Figure 84. Location of PA 1 project.

Study Objectives

One of the primary objectives for this project is to evaluate HIPERPAV, a computer program that can be used to evaluate the early-age behavior of concrete pavements. This study will evaluate the ability of HIPERPAV to predict strength gain and to model early-age stress development. Other objectives of particular interest to PENNDOT include addressing needs for the new mechanistic-empirical pavement design guide. Six additional objectives will be addressed in this study:

1. Characterize thermal gradients throughout the pavement structure.
2. Characterize moisture gradients throughout the pavement structure.
3. Quantify construction curling and seasonal and diurnal curling.
4. Develop suitable strength correlations.
5. Monitor slab performance over time.
6. Characterize slab response to environmental and applied loads.

Project Design and Layout

Four test cells will be constructed on a four-lane divided highway. The project layout is provided in Figure 84 and a summary of the design features for each cell is provided in Table 45.

The types of instrumentation to be installed on this pavement project include the following:

- Environmental Sensors
  - Moisture gauges
  - Thermocouples
  - Time domain reflectometers
- Static Sensors
  - Static strain gauges
  - Static pressure cells
- Dynamic Sensors
  - Dynamic Strain gauges
  - Dynamic pressure cells

Environmental and static data collection will be automated and collected every 15 minutes. Dynamic data will be collected manually. All data will be stored in a database housed at the University of Pittsburgh.

State Monitoring Activities

The test sections will be monitored seasonally by the University of Pittsburgh. This will include making Dipstick measurements, performing distress surveys and collecting dynamic strain data from various truck types, weights and axle configurations. Ride data and FWD testing will be performed by PENNDOT. Seasonal monitoring will be performed for 4 years.

Preliminary Results/Findings

No preliminary results are available.

Table 45. Design Features for the PA 1 Project

DESIGN FEATURE	CELL 1	CELL 2	CELL 3	CELL 4
Pavement type	JPCP	JPCP	JPCP	JPCP
Tie bars	Yes	No	No	Yes
Dowel bars	1.25 in.	No	No	1.25 in.
Slab thickness	12 in.	12 in.	12 in.	12 in.
Base type	Asphalt treated	Asphalt treated	Asphalt treated	Asphalt treated
Lane width	12 ft	12 ft	12 ft	12 ft
Dynamic sensors	Yes	Yes	No	No
Static and environmental sensors	No	No	Yes	Yes

Point of Contact

Julie M. Vandenbossche
University of Pittsburgh
934 Benedum Hall
3700 Ohara St.
Pittsburgh, PA 15261
(412) 624-9879
jmv@engr.pitt.edu

Chapter 30. SOUTH DAKOTA 1 (US Route 83, Pierre)

Introduction

In the early 1990s, the South Dakota Department of Transportation (SDDOT) investigated the use of non-metallic fiber reinforced concrete (NMFRC) on several small bridge and highway applications (Ramakrishnan 1995; Ramakrishnan 1997). Although the NMFRC performed well in each application, a strong need for additional information on the design, construction, and performance of NMFRC pavements was identified. Thus, this project, located on US 83 northeast of Pierre (see Figure 85), was constructed in 1996 under the TE-30 program to allow, on a larger scale, the evaluation of NMFRC pavements with different slab thicknesses, joint spacings, and the need for transverse joint load transfer (Ramakrishnan and Tolmare 1998).

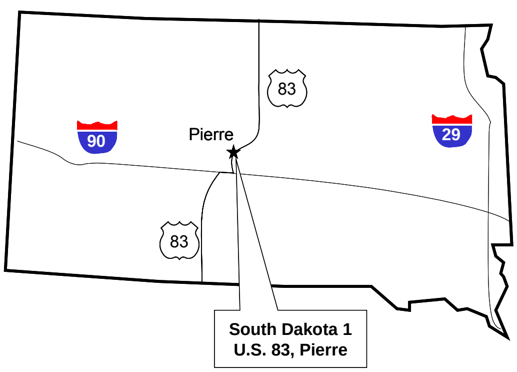

Figure 85. Location of SD 1 project.

Study Objectives

In order to more fully assess the suitability of NMFRC in full-depth concrete, this project was constructed to accomplish the following objectives (Ramakrishnan and Tolmare 1998):

- Develop NMFRC full-depth pavement designs that will enhance PCC pavement performance, including appropriate slab thicknesses, joint load transfer designs, and joint spacings.
- Evaluate the constructibility and performance of NMFRC full-depth pavement.
- Evaluate the economic impacts of using NMFRC full-depth pavement.

Project Design and Layout

This project was constructed in 1996 on US 83 northeast of Pierre, between mileage reference markers (MRMs) 144 and 145 (Ramakrishnan and Tolmare 1998). The new pavement is a two-lane roadway (each lane 4.3 m [14 ft] wide), and was constructed on an existing gravel base course. Eight different test sections are included in the project: one control section and seven non-metallic, fiber reinforced concrete pavement (NMFRCP) sections. Figure 86 illustrates the layout of the test sections, and Table 46 summarizes the design features of the test sections. Table 47 presents the experimental design matrix for the project.

The NMFRCP sections used polyolefin fibers, manufactured by the 3M company. These fibers are purported to provide mechanical improvements to the concrete and are also non-corrodible and resistant to chemicals (Ramakrishnan and Tolmare 1998). The NMFRCP mix design used in this project is shown in Table 48 (Ramakrishnan and Tolmare 1998).

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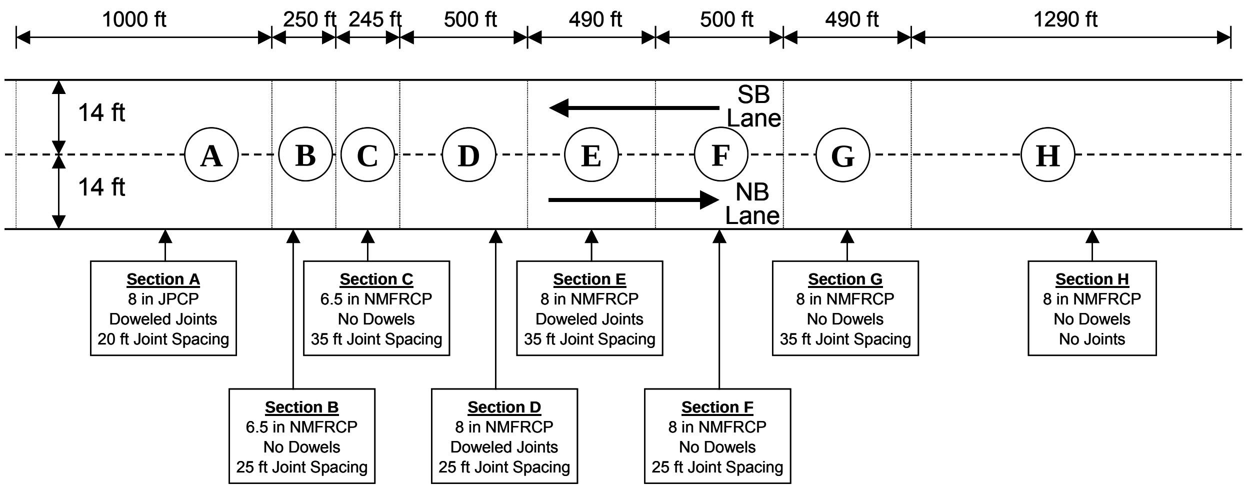

Figure 86. Layout of SD 1 test sections (Ramakrishnan and Tolmare 1998).

Table 46. Design Features of SD 1 Test Sections (Ramakrishnan and Tolmare 1998)

TEST SECTION	LENGTH, FT	TYPE	THICKNESS, IN.	DOWELED	JOINT SPACING, FT	VOLUME OF FIBER CONCRETE, YD^3
A	1000	JPCP	8.0	Yes	20	0
B	250	FRCP (25 lb/yd^3)	6.5	No	25	173
C	245	FRCP (25 lb/yd^3)	6.5	No	35	170
D	500	FRCP (25 lb/yd^3)	8.0	Yes	25	346
E	490	FRCP (25 lb/yd^3)	8.0	Yes	35	339
F	500	FRCP (25 lb/yd^3)	8.0	No	25	346
G	490	FRCP (25 lb/yd^3)	8.0	No	35	339
H	1290	FRCP (25 lb/yd^3)	8.0	No	None	892

Table 47. Experimental Design Matrix for SD 1

		JPCP		NMFRCP	
		Doweled	**Nondoweled**	**Doweled**	**Nondoweled**
6.5-in. Slab	**20 ft Joints**				
	25 ft Joints				Section B
	35 ft Joints				Section C
	No Joints				
8-in. Slab	**20 ft Joints**	Section A			
	25 ft Joints			Section D	Section F
	35 ft Joints			Section E	Section G
	No Joints				Section H

Table 48. NMFRCP Mix Design Used in SD 1 Project (Ramakrishnan and Tolmare 1998)

MIX COMPONENT	QUANTITY
Cement (Type II)	510 lb/yd^3
Fly ash (Type F)	112 lb/yd^3
Water	264 lb/yd^3
Limestone coarse aggregate	1417 lb/yd^3
Fine aggregate	1417 lb/yd^3
Polyolefin fibers	25 lb/yd^3
Slump	1–2 in.
Air content	6 $\pm$ 1.5 percent

Preliminary Results/Findings

The monitoring of the design, construction, and early performance of these pavement sections had led to the following conclusions regarding the use of NMFRC pavements (Ramakrishnan and Tolmare 1998):

- The same construction techniques and construction equipment can be used in the batching and placement of fiber-reinforced concrete pavements. The only modification needed in the batching process is the addition of a plastic tube to facilitate the introduction of the fibers. However, some additional mixing time may be required for NMFRC pavements.
- No differences in the riding quality of the pavement sections could be established. All of the pavement sections met the smoothness criteria for new pavement construction.
- During the construction of these test sections, both cylinders and beams were cast for later laboratory testing. A comparison of the flexural strengths for beams cast from concrete used on different days of paving is shown in Figure 87; these are the average of three beam breaks for each of the following specimens:

  – "P1" indicates specimens collected from the northbound NMFRC paving operations on August 15, 1996.

  – "P2" indicates specimens collected from a portion of the southbound NMFRC paving operations on August 26, 1996.

  – "P3" indicates specimens collected from the rest of the southbound NMFRC paving operations on August 27, 1996.

  – "Control" indicates specimens collected from the northbound control section paved on August 15, 1996.

  The strengths of the NMFRC specimens are observed to be greater than that of the control section, although on average the percent increase is only about 20 percent. However, other laboratory tests of toughness, impact, fatigue, endurance limit, and post-crack load carrying capacity suggested that the structural properties of the fiber-modified concrete had been improved.

- After about 3 years of service, condition surveys of the pavement sections revealed that transverse cracks had occurred only in the section without joints (section H). These cracks occurred at approximately 26-m (85-ft) intervals. No other distresses were noted within the pavement sections.

- Data from FWD testing showed that the load transfer was less in the NMFRC pavement sections compared to the control section (see Figure 88). In general, the load transfer was less in sections with longer joint spacings, thinner slabs, or nondoweled joints. The longer joint spacings of the nondoweled NMFRC pavements reduce the effectiveness of the aggregate interlock at the joints, even though they contain fibers. This suggests that the doweling recommendations for conventional PCC pavements also apply to fiber reinforced concrete pavements. Unfortunately, direct comparisons of load transfer could not be made between the control section and the NMFRC sections because joint spacing was not held constant in the experimental design.

- The initial cost of the 203-mm (8-in.) JPCP was \$18.36/m^2 (\$15.35/yd^2), whereas the initial cost of the 203-mm (8-in.) NMFRC was \$34.57/m^2 (\$28.90/yd^2) (both based on an 8.5-m [28-ft] wide pavement). A life cycle cost analysis of both designs showed that the conventional design was 61 and 31 percent cheaper than the NMFRC design for analysis periods of 40 and 60 years, respectively. Nevertheless, there may be special design situations in which longer joint spacings, thinner slabs, or more efficient performance may dictate the use of NMFRC pavements.

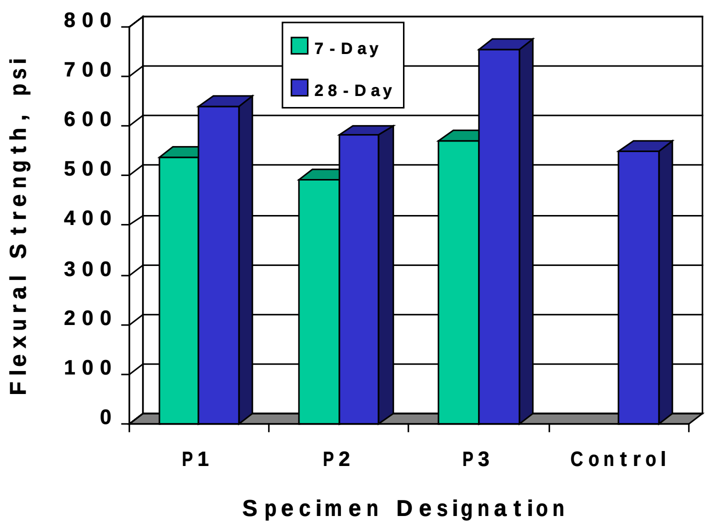

Figure 87. Summary of flexural strength tests for SD 1 (Ramakrishnan and Tolmare 1998).

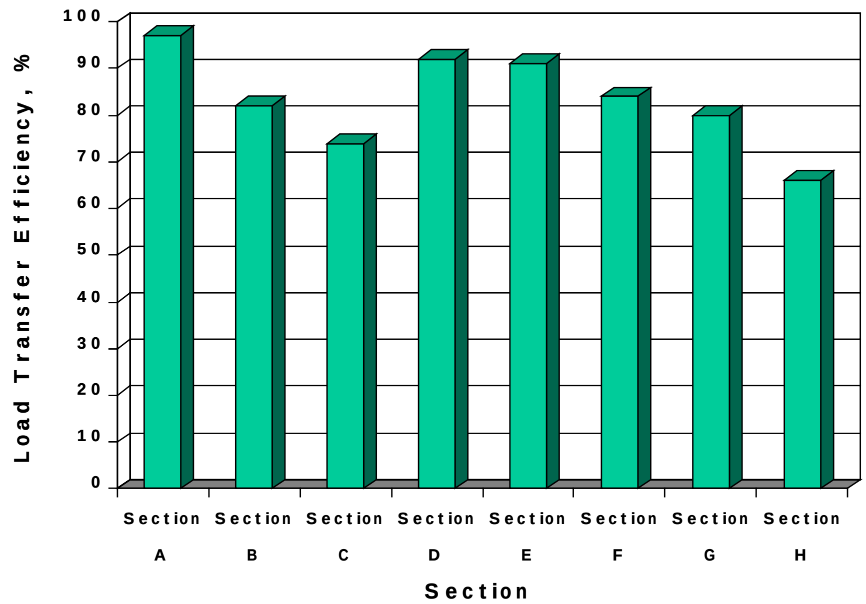

Figure 88. Load transfer efficiencies for SD 1 sections.

Interim Results/Findings

Distress surveys conducted on these test sections in February 2004 revealed no apparent distress of any kind except for the uncontrolled transverse cracks in section H (Johnston and Huft 2004). The uncontrolled transverse cracks showed no faulting, spalling, etc. and remained narrow. The entire pavement section was uniform in appearance and in good condition (Johnston & Huft 2004).

Points of Contact

Dave Huft
South Dakota Department of Transportation
700 East Broadway Avenue
Pierre, SD 57501-2586
(605) 773-3292
Dave.Huft@state.sd.us

Venkataswamy Ramakrishnan
Department of Civil and
Environmental Engineering
SD School of Mines and Technology
501 E St. Joseph Street
Rapid City, SD 57701-3995
(605) 394-2403

References

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Ramakrishnan, V., and N. S. Tolmare. 1998. *Evaluation of Non-Metallic Fiber Reinforced Concrete in New Full Depth PCC Pavements*. Report No. SD96-15-F. South Dakota Department of Transportation, Pierre.

Chapter 31. TENNESSEE 1 (I-65, Nashville)

Introduction

Over the past decade, significant progress has been made in the development of performance-related specifications (PRS) for the acceptance of newly constructed jointed plain concrete pavement (JPCP). A PRS is able to relate the construction acceptance quality characteristics (AQCs) (e.g., smoothness, thickness, strength, air content, and percent consolidation around dowels) to the levels of expected pavement performance and future life cycle costs (LCCs), thus providing the basis for rational acceptance and price adjustment decisions. The PRS methodology has been verified through the use of shadow specifications on actual construction projects.

In this study, the Tennessee Department of Transportation (TDOT) is implementing PRS on a field trail basis under a FHWA contract. Should the implementation of PRS in this project be successful, TDOT would be able to use PRS in more projects without outside assistance and would be able to achieve the full benefits of PRS.

Study Objectives

The main objective of this project is to evaluate the practicality and effectiveness of PRS for JPCP based on a real-world field trial in Tennessee. The long-term benefits of PRS expected to be derived from this project include (Gharaibeh 2003):

- Better linkage between design and construction.
- Higher quality pavements (through incentives).
- Testing that focuses on key quality characteristics that relate to the pavement's long-term performance.
- Incentives and disincentives that are justified through reductions or increases in life cycle costs.
- Specifications that give contractors more flexibility, responsibility, and accountability.
- Incentive to contractors to be more innovative and more competitive.
- Lower "fear factor" for contractors and less administrative complexity for the agency than warrantees.

Project Design and Layout

TDOT provided the following candidate projects for the implementation of PRS (Gharaibeh 2003):

- I-65 north of Nashville. This project is ongoing and consists of widening a 5.6-km (3.5-mi) long roadway from 6 to 10 lanes, with the existing concrete pavement to be reconstructed. Paving is expected to take place in March or April 2004.
- I-75 north of Chattanooga and I-65 north of Nashville. These projects are expected to be let in December 2004. However, paving could take place 2 to 3 years after letting.

After a project coordination meeting, it was recommended to TDOT that the most practical option is to implement PRS on a portion of the ongoing I-65 project.

State Monitoring Activities

No final decisions have been made regarding the State monitoring activities.

Preliminary Results/Findings

Acceptance test results from three concrete pavement projects that were constructed in Tennessee in the past 5 years were obtained and analyzed. The project-wide results are summarized in Table 49. These results will assist in establishing the target values (mean and standard deviation) for the PRS project, considering the quality levels achieved in these past projects (Gharaibeh 2003).

Point of Contact

Sam Tyson
Federal Highway Administration
Office of Infrastructure
Office of Pavement Technology (HIPT)
400 7th Street, SW
Washington, DC 20590
(202) 366-1326

Reference

Gharaibeh, N. G. 2003. *Performance-Related Specifications (PRS) Field Trial in Tennessee.* Quarterly Progress Report to the Federal Highway Administration.

Table 49. Summary of Past Concrete Pavement Projects in Tennessee

ATTRIBUTE		PROJECT 1	PROJECT 2	PROJECT 3
Tennessee Department of Transportation Identification		S.P. 33003-4154-04 (Contract No. 4359)	IM-40-2(71)87, 57001-8172-44 (Contract No. 4559)	NH-I-75-1(95)3, 33005-3161-44 (Contract No. 5356)
Location		I-24, Hamilton County	I-40, Madison and Henderson Counties	I-75, Hamilton County
Approximate length, mi		2.76	8.02	3.2
Project period		1997-2000	1997	1999-2001
28-day compressive strength, lbf/in^2	Field average	6432	5247	6046
	Field standard deviation	892	315	625
	Specifications	Min 3000	Min 3000	Min 3000
Thickness, in.	Field average	N/A	N/A	12.04
	Field standard deviation	N/A	N/A	0.11
	Specifications	N/A	N/A	12.00
Air content, %	Field Average	5.46	5.11	5.14
	Field standard deviation	0.51	0.11	0.44
	Specifications	3-8	3-8	3-8
P.I, in./mi	Field Average	2.55	N/A	2.53
	Field standard deviation	0.88	N/A	0.71
	Specifications	5	N/A	4

Chapter 32. VIRGINIA 1 (I-64, Newport News)

Introduction

Concrete pavements constructed in Virginia over 20 years ago commonly incorporated a 50-mm (2-in.) maximum coarse aggregate size in the PCC mix design (Ozyildirim 2000). These pavements generally performed well, but over time concerns with the availability, stockpiling, and segregation of the aggregate led to a reduction in the specified maximum coarse aggregate size (Ozyildirim 2000). However, while smaller coarse aggregate size is inherently more durable, concrete using smaller coarse aggregate commonly exhibits greater shrinkage (and increased potential for slab cracking) because of increased paste requirements. Larger maximum coarse aggregate sizes, on the other hand, require less paste, less cementitious material, and less water, thereby resulting in reduced shrinkage; they also provide increased mechanical interlock at joints and cracks (Ozyildirim 2000).

To investigate the effect of maximum coarse aggregate on concrete material properties and pavement performance, the Virginia Department of Transportation (VDOT) constructed an experimental concrete pavement project as part of the TE-30 program. The project is located on I-64 near Newport News (see Figure 89) and was constructed in 1998 and 1999 (Ozyildirim 2000).

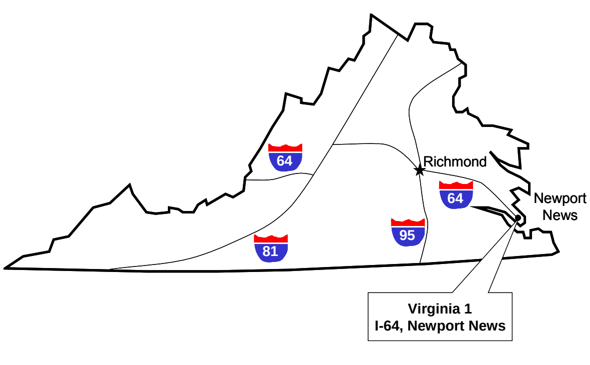

Figure 89. Location of VA 1 project.

Study Objectives

The objectives of this project are (Ozyildirim 2000):

- To develop concrete mixtures that have low shrinkage and high flexural strength.
- To determine the properties of such concretes.
- To instrument and test jointed pavement slabs for volumetric changes.
- To determine the air void distribution (indicative of consolidation) from cores.

Project Design and Layout

This project was constructed on I-64 in Newport News, beginning about 2.4 km (1.5 mi) west of Route 143 and ending about 1.6 km (1 mi) east of Route 143 (Ozyildirim 2000). The project involved removing and replacing the two existing traffic lanes in both the eastbound and westbound directions, and constructing an additional lane in each direction (Ozyildirim 2000). The nominal pavement design used for these sections is as follows (Ozyildirim 2001):

- 280-mm (11-in.) JPCP.
- 75-mm (3-in.) asphalt-stabilized open-graded drainage layer (OGDL).
- 150-mm (6-in.) cement-treated subbase.
- 4.6-m (15-ft) transverse joint spacing.
- Silicone sealant in all transverse joints.
- 32-mm (1.25-in.) diameter epoxy-coated steel dowel bars (511 mm [18 in.] long and spaced 300 mm [12 in.] apart).
- Widened slabs (4.3 m [14 ft]) in the outer traffic lane.

Three experimental sections were constructed in the westbound lanes in November 1998, and then replicated in the construction of the eastbound lanes in July 1999 (Ozyildirim 2000). Each of the three sections incorporated a different concrete mix design (Ozyildirim 2001):

- Section 1 EB/WB—concrete mixture using a 50-mm (2-in.) maximum coarse aggregate and including ground granulated blast furnace slag (GGBFS).
- Section 2 EB/WB—concrete mixture using a 25-mm (1-in.) maximum coarse aggregate and including GGBFS.
- Section 3 EB/WB—concrete mixture using a 25-mm (1-in.) maximum coarse aggregate and including Class F fly ash. This section represents VDOT's current conventional concrete mix design.

Figure 90 illustrates the layout of these test sections, while Table 50 summarizes the concrete mix design information. The mix designs for sections 1 and 2 contain 30 percent slag, whereas the mix design for section 3 contains 25 percent fly ash. The coarse aggregate was a crushed biotite gneiss and granite, whereas the fine aggregate was a natural sand (Ozyildirim 2001). The coarse aggregate used in section 1 was a blend of No. 3 and No. 57 aggregates (Ozyildirim 2001).

Comparing the mix design of section 1 (the larger aggregate) with the mix designs for the other two sections, it is observed that the use of the larger aggregate did slightly reduce the required quantities of water and cementitious materials (Ozyildirim 2000).

Dowel bars were placed in baskets at the predetermined joint locations prior to the paving operations (Ozyildirim 2001). Transverse joints were cut initially 4.75 mm (0.19 in.) wide to a depth of one-third of the slab thickness (Ozyildirim 2001). A secondary cut was made 2 weeks later to establish the joint reservoir for the silicone sealant.

A water-based curing compound was used to cure the pavement, except that the appropriate materials could not be obtained during the placement of the westbound lanes (Ozyildirim 2001). Consequently, a white plastic sheeting was used to cure the pavement for 10 days (Ozyildirim 2001).

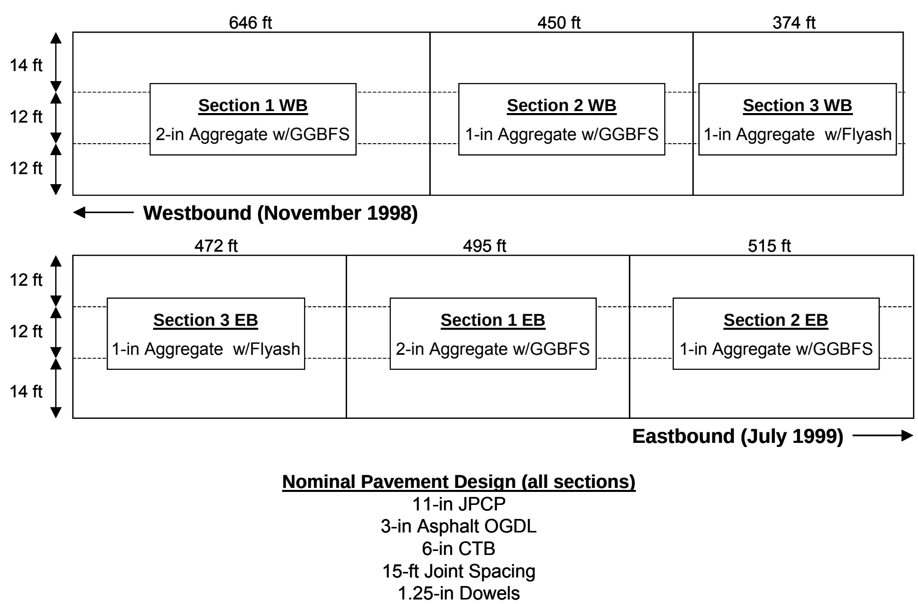

Figure 90. Layout of VA 1 test sections (Ozyildirim 2000).

Table 50. Concrete Mix Designs Used in VA 1 Project (Ozyildirim 2000)

MATERIAL	SECTION 1 (SLAG MIX #2)	SECTION 2 (SLAB MIX #1)	SECTION 3 (FLY ASH)
Portland cement quantity, lb/yd^3 (kg/m^3)	374 (222)	394 (234)	423 (251)
GGBFS quantity, lb/yd^3 (kg/m^3)	160 (95)	169 (100)	—
Class F fly ash quantity, lb/yd^3 (kg/m^3)	—	—	142 (84)
Pozzolan quantity, lb/yd^3 (kg/m^3)	30 percent	30 percent	25 percent
Course aggregate No.	357	57	57
Maximum top size, in (mm)	2 in (50 mm)	1 in (25 mm)	1 in (25 mm)
Course aggregate quantity, lb/yd^3 (kg/m^3)	1935 (1148)	1841 (1092)	1841 (1092)
Fine aggregate quantity, lb/yd^3 (kg/m^3)	1171 (695)	1217 (722)	1229 (729)
Water, lb/yd^3 (kg/m^3)	243 (144)	249 (148)	249 (148)
Water-to-cementitious materials ratio (w/cm)	0.45	0.44	0.44
Admixtures	Air entraining agent + retarder	Air entraining agent + retarder	Air entraining agent + retarder

During construction, one outside slab in each of the three sections in the westbound lanes was instrumented to record strains, displacements, and temperatures in the slab (Ozyildirim 2000). Two vibrating wire strain gauges were placed in the middle of each slab, one 38 mm (1.5 in.) from the top of the slab and one 38 mm (1.5 in.) from the bottom of the slab (Ozyildirim 2000). Two additional vibrating wire strain gauges were tied to two stainless steel stakes and driven into the base (Ozyildirim 2001). The gauges were placed 2 m (6.5 ft) from the outside edge to avoid the vibrator of the paver (Ozyildirim 2001).

Two linear variable differential transformers (LVDTs) were installed in each of the slag concrete pavement sections to measure vertical displacements due to curling (Ozyildirim 2001). One LVDT was placed at the center of the slab and the other 280 mm (11 in.) from the joint (Ozyildirim 2001).

Type T thermocouples were placed at each location in the slab where the vibrating gauges were placed. The thermocouples were placed at 6 mm (0.25 in.), 76 mm (3 in.), 140 mm (5.5 in.), 203 mm (8 in.), and 273 mm (10.75 in.) above the base (Ozyildirim 2001). In addition, the vibrating wire gauges included thermistors that provide temperatures at the each location (Ozyildirim 2001).

Finally, 10 consecutive joints in each section were instrumented to monitor transverse joint movements (Ozyildirim 2001). Gauges were placed on either side of the joints 1 week after paving (Ozyildirim 2001).

State Monitoring Activities

During and immediately after construction, VDOT conducted an evaluation of the fresh and hardened concrete properties of each pavement section. Table 51 summarizes some of the selected concrete properties obtained from the monitoring. Both the fresh concrete properties and the hardened concrete properties generally met design requirements, but there was a noticeable difference in the properties of the mixtures between the westbound and eastbound lanes. Generally speaking, the westbound lanes had lower strengths and higher air contents than the eastbound lanes.

Comparisons of the different mixes show that the slag mixes (sections 1 and 2) have higher strengths than the fly ash mixes. Interestingly, the section 2 mixes had higher strengths than the section 1 mixes, even though the section 1 mixes had a larger coarse aggregate size.

The chloride ion permeability test measures the electrical conductance of a sample, and VDOT specifies a maximum of 3500 coulombs (Ozyildirim 2001). All of the mixtures exhibit permeabilities much less than that value, with the fly ash mixtures the lowest of all of the mixtures. All of the sections exhibited similar shrinkage values, with the section 1 mixture (containing the largest maximum coarse aggregate size) exhibiting the least amount of shrinkage.

The acceptance criteria for freeze-thaw data shown in Table 51 are a weight loss of 7 percent or less and a durability factor of 60 or more (Ozyildirim 2001). All mixtures complied with these requirements except the fly ash mixtures exceeded the allowable weight loss. However, this is a severe test and the fly ash mixtures are expected to perform satisfactorily in the field provided that they have adequate strength and an adequate air void system (Ozyildirim 2001).

Monitoring of the instrumented slabs was conducted during the first several weeks after construction, along with transverse joint movements. Generally, larger thermal gradients were observed for section 2, but all of the differences were really quite small (Ozyildirim 2001). Section 1 (larger aggregate) showed less curling than section 2, but again the differences were small (Ozyildirim 2001). Limited FWD testing showed nearly identical load transfer efficiencies of 85, 85, and 88 percent for section 1, 2, and 3, respectively (Ozyildirim 2001).

Table 51. Summary of Concrete Properties Measured on VA 1 Project (Ozyildirim 2000).

CONCRETE PROPERTY	SPECIMEN SIZE, IN.	SECTION 1 (2-IN. AGG W/SLAG)		SECTION 2 (1-IN. AGG W/SLAG)		SECTION 3 (1 IN. AGG W/FLY ASH)	
		EB	WB	EB	WB	EB	WB
Fresh Concrete							
Slump, in.		0.88	1.50	1.88	1.25	1.50	1.25
Air, %		4.30	5.25	4.95	5.80	4.20	5.45
Concrete temperature, ^oF		86.5	67	85.5	68	82.0	67
Unit weight, lb/yd^3		145.6	144.2	147.0	145.6	147.6	143.8
Hardened Concrete							
28-day compressive strength, lbf/in^2	6 x 12	5446	4530	5540	4625	4612	3920
28-day flexural strength, lbf/in^2	6 x 6 x 20	704	670	783	685	—	—
Permeability, coulombs	4 x 4	1364	1774	1548	1672	680	1265
1-year shrinkage, %	6 x 6 x 200	0.041	0.052	0.044	0.059	—	—
Freeze-Thaw Data at 300 cycles (ASTM C 666, Procedure A, except air dried 1 week and 2% NaCl in test solution)							
Weight loss, %		5.3	4.0	2.3	6.0	11.6	14.1
Durability rating		69	91	88	89	82	83

After 2 years and 6 months of service, respectively, both the westbound sections and the eastbound sections are in excellent condition, with no distress or scaling (Ozyildirim 2001). VDOT will continue monitoring the performance of these pavements and will produce a final report in 2004 (Ozyildirim 2000).

Preliminary Results/Findings

This project has illustrated that air-entrained paving concrete with satisfactory strength, low permeability, and volume stability can be prepared using concrete with Class F fly ash or slag, and with 25- and 50-mm (1- and 2-in.) maximum size aggregates (Ozyildirim 2000; Ozyildirim 2001). The larger maximum size aggregate is expected to provide better performance in the field, and it will be monitored over the next 5 years. Although the reduction in water and cement contents was minimal for the mix with the larger sized aggregate, the use of a more uniform combined grading is expected to reduce water and paste demands (Ozyildirim 2000; Ozyildirim 2001). The instrumentation did not provide any strong results regarding the relative performance of the different pavement sections (Ozyildirim 2000; Ozyildirim 2001).

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Project Status

No interim field testing results have been published, nor were they available for this update. However, an additional field evaluation will be conducted in the spring of 2004 that will include FWD and automated distress measurements. Results of these field reviews will be included in the final project report that is slated to be complete in August 2004.

Point of Contact

Celik Ozyildirim
Virginia Transportation Research Council
530 Edgemont Road
Charlottesville, VA 22903
(434) 293-1977
celik@vdot.state.va.us

References

Ozyildirim, C. 2000. *Evaluation of High-Performance Concrete Pavement in Newport News, VA.* Draft Interim Report. Virginia Transportation Research Council, Charlottesville.

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Chapter 33. VIRGINIA 2 (Route 288, Richmond) and VIRGINIA 3 (US 29, Madison Heights)

Introduction

VDOT constructed two additional high-performance concrete pavements under the TE-30 program. Virginia 2 is located on Route 288 near Richmond, and Virginia 3 is located on US 29 near Madison Heights (see Figure 91). These pavements are both continuously reinforced concrete pavement (CRCP) designs that are incorporating concrete mixtures expected to provide high flexural strengths and low shrinkage.

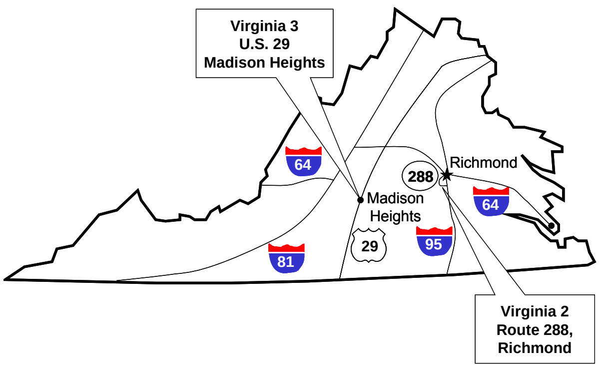

Figure 91. Location of VA 2 and VA 3 projects.

Study Objectives

These two pavement projects share the following objectives:

- Develop concrete mixes that have low shrinkage and high flexural strength.
- Assess aggregate particle shape using a video grader.
- Determine the material properties of high performance concrete, including compressive strength, flexural strength, elastic modulus, coefficient of thermal expansion, permeability, and drying shrinkage.
- Continuously monitor the speed of the paver and the frequency of the vibrators during the construction of the pavement.
- Investigate the effect of longitudinal steel content on the development of cracking.

Curing of the sections (curing compound vs. burlap wet cure) may also be investigated as part of the investigation.

Project Design and Layout

The project on Route 288 was paved in mid-December 2003. Samples and data were gathered during the construction. The paving materials will be tested as per the AASHTO 2002 guidelines. No specific information regarding the actual pavement construction or the results from any followup testing is currently available.

The Madison Heights project (located on US Route 29) has not yet been constructed; consequently, no information is currently available.

State Monitoring Activities

Once these projects are constructed, the State will monitor the performance of the sections for 5 years.

Preliminary Results/Findings

No preliminary results or findings are available.

Point of Contact

Celik Ozyildirim
Virginia Transportation Research Council
530 Edgemont Road
Charlottesville, VA 22903
(434) 293-1977
celik@vdot.state.va.us

Chapter 34. Washington 1 (SR 395, Kennewick)

Introduction

Traditionally, reconstruction of urban intersections with portland cement concrete pavement (PCCP) requires several weeks and complex traffic management plans. However, with accelerated paving construction techniques, concrete can meet opening strengths in less than 12 hours, providing quick public access to a high-quality, long-lasting pavement. In the fall of 2000, the Washington State Department of Transportation (WSDOT) reconstructed an asphalt concrete (AC) intersection with PCC over a weekend in a 70-hour period. The intersection is located at SR 395 and West Kennewick Avenue in the City of Kennewick, Washington (see Figure 92). The successful completion of this project demonstrates that concrete can be used successfully for the rapid reconstruction of urban intersections.

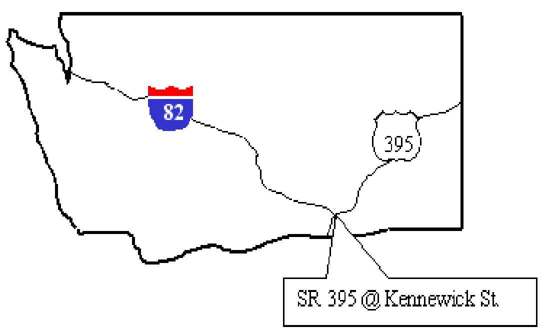

Figure 92. Location of WA 1 project.

Study Objectives

The goal of this project is to develop a body of knowledge and tools for accelerated reconstruction of urban intersections using PCC. To achieve this goal, three classes of activities were investigated: methods to accelerate the rate of concrete strength gain, methods to minimize the construction time, and traffic control strategies to minimize user delay (Nemati et al. 2003).

Project Design and Layout

Before reconstruction, the HMA pavement exhibited severe rutting, as much as 50.8 to 101.6 mm (2 to 4 in.) (Nemati et al. 2003). Traffic volume passing through the intersection is as high as 30,000 vpd, including 20 percent heavy trucks (Nemati et al. 2003). A 3-day weekend closure, beginning on Thursday evening, was adopted for the intersection reconstruction. Efforts were focused on the following aspects:

Public Relations and Traffic Management

Many individuals, especially business owners in the affected areas, were contacted personally by WSDOT personnel. Pre-construction meetings were held with the City council and the public to encourage active involvement of all the affected parties. The media was utilized to alert the public to the upcoming construction and to keep them up to date on the schedule. A multistaged traffic detour plan was implemented that provided local access, access to commercial sites, and special routes for heavy trucks passing through the area. These activities were aimed at making the closures as organized and painless as possible for the public.

Concrete Mixture Properties

For this project, the concrete mix design was critical in maintaining the accelerated schedule. ASTM C 150 Type III portland cement was used to provide rapid strength gain. Table 52 shows the concrete mix design used in this project, while Table 53 shows the characteristics of the concrete delivered to the site (Nemati et al. 2003). Figure 93 shows the strength-maturity relationship that was developed for the specific concrete mixture used in the intersection reconstruction (Nemati et al. 2003). According to WSDOT requirements, a minimum concrete compressive strength of 17,236.9 kPa (2,500 lbf/in^{2}) must be achieved in order for the intersection to be opened to traffic. Figure 93 indicates that this mixture is capable of reaching the opening strength approximately 8 hours after placement.

Construction

The contractor prepared and executed an hourly progress schedule during the intersection closure. Figure 94 shows the contractor's critical path method schedule (Nemati et al. 2003). The approach legs were rebuilt in the days prior to the complete intersection closure. A rotomill was used to remove the existing pavement and base at the intersection to a depth of 304.8 mm (12 in.), which was accomplished in a single pass. Concrete was placed in alternate sections, and then the sides of the newly placed pavements acted as forms for the interim sections. In this way, a significant amount of time was saved by not erecting and removing side forms for the interim sections.

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In this project, liquid curing compounds meeting ASTM C309 requirements were applied to the surface and exposed edges of the concrete pavement at a rate of one gallon per 13.9 m^2 (150 $ft^{2)}$. The sawcutting at the intersection typically began within 6 hours of concrete placement. After sawing, the joints were sealed with a hot-poured asphaltic material.

Maturity meters were used in this project to monitor the strength gain in the concrete and determine the appropriate time for stripping forms and opening the pavement to traffic. The intersection was opened to traffic on Sunday between 4:00 p.m. and 6:00 p.m., well ahead of the originally scheduled 6:00 a.m. Monday morning opening.

Table 52. Concrete Mix Design Used on WA 1 Project

MATERIAL	TYPE	QUANTITY
Cement (lbs/yd^3)	ASTM C150 Type III	705
Aggregate (lbs/yd^3)	1.5 in.	940
	0.75 in.	799
	0.375-in. Pea Gravel	140
Sand (lbs/yd^3)	Coarse	590
	Fine	481
Water (lbs/yd^3)	–	254
Air-entraining admixture (oz/yd^3)	ASTM C260	11
Water-reducing admixture (oz/yd^3)	ASTM C494	30.3
Set-retarding admixture (oz/yd^3)	ASTM C494 / Delvo	17.6

Table 53. Characteristics of Concrete Used on WA 1

CHARACTERISTIC	TEST RESULT
Slump	3.25 in.
w/c ratio	0.36
Air content	6.3 percent
Unit weight	149.8 lb/ft^3
Concrete temperature	85 °F
Air temperature	82 °F

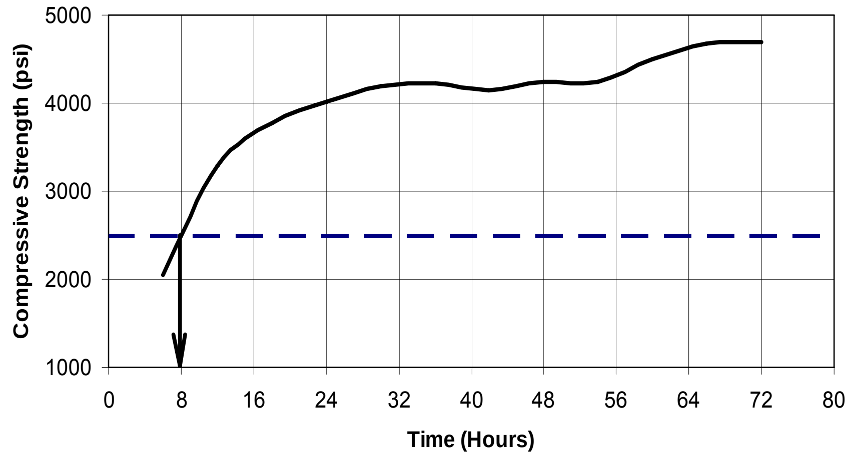

Figure 93. Strength-time relationship developed for WA 1 project.

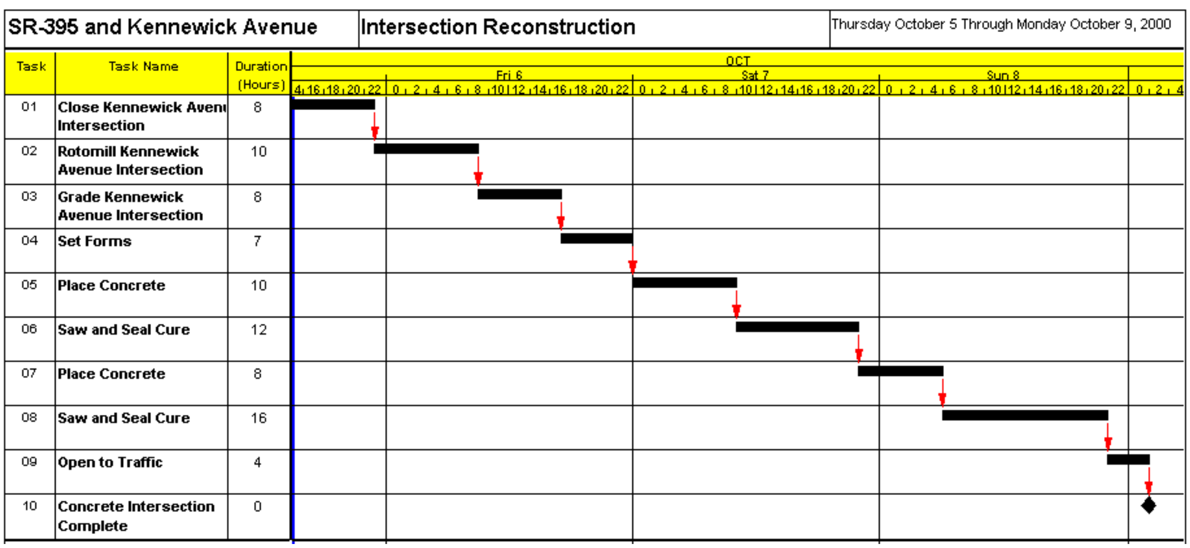

Figure 94. Contractor’s critical path method schedule for WA 1 project.

Results/Findings

The reconstruction of the intersection at SR 395 and Kennewick Avenue using accelerated construction techniques and complete weekend closure was completed successfully. The intersection was opened to traffic 16 hours ahead of the scheduled opening time. The breakdown of the time actually spent on each activity during the reconstruction is shown in Figure 95 (Nemati et al. 2003). As a result of the traffic control and public relation management, complaints from the public were reduced by over 70 percent compared to a project constructed 2 years before (Nemati et al. 2003). The unit price for reconstruction of the intersection at SR 395 and Kennewick Avenue is 168.26/m^2 (\$15.58/ft^2) (Nemati et al, 2003).

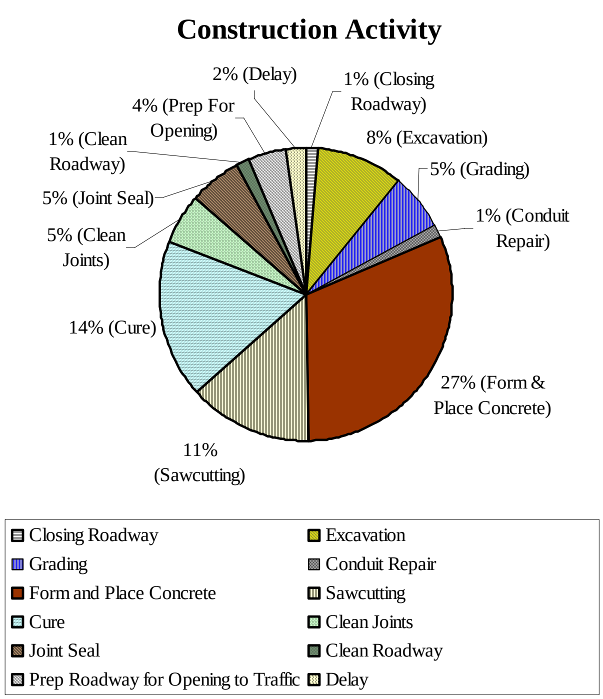

Figure 95. Breakdown of time consumed by each construction activity on WA 1 project.

Points of Contact

Kamran M. Nemati
University of Washington
P.O. Box 95233
Seattle, Washington 98145-2233
(206) 685-4439
nemati@uwashington.edu

Jeff S. Uhlmeyer
(360) 709-5485
UhlmeyJ@wsdot.wa.gov

Linda M. Pierce
(360) 709-5470
PierceL@wsdot.wa.gov

Washington State Department of Transportation
P.O. Box 47365
Olympia, WA 98504-7365

Reference

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Chapter 35. WEST VIRGINIA 1

Introduction

As part of a comprehensive research program investigating the use of glass fiber-reinforced plastic (GFRP) reinforcing bars in concrete pavements, the West Virginia University initiated a research study consisting of three research elements: (1) construction and evaluation of GFRP-doweled pavement sections; (2) investigation of moisture diffusion in GFRP dowels; and (3) construction and evaluation of a CRCP pavement section containing GFRP rebars. The use of GFRP bars in lieu of steel bars is expected to alleviate the problems associated with steel corrosion and steel distress. At the conclusion of the project, a workshop and technology transfer initiative are scheduled to be held.

Study Objectives

As mentioned above, three separate investigations are being conducted under this research project. Specific objectives of each investigation are described below.

GFRP Dowel Study

The objectives of this study are to (Li 2004; Vijay, GangaRao, and Li 2004):

1. Evaluate the use of GFRP dowel bars as load transfer devices in JPCP under HS-25 static and fatigue loads and compare their response with JPCP consisting of steel dowels. This includes the conduct of laboratory and field evaluations of JPCP with both GFRP and steel dowels, as well as the conduct of analytical modeling of dowel response in terms of maximum bending deflection, relative deflection, and bearing stress.

2. Evaluate the long-term performance of field rehabilitation of JPCP using GFRP dowels.

Moisture Diffusion Study

This is a laboratory study with the following objectives (Gupta, Rana, and GangaRao 2004):

1. Measure the diffusion coefficients and equilibrium moisture contents, and establish the mechanism of water diffusion through neat and glass fiber-reinforced DERAKANE 411-350 Momentum vinyl ester resin bars.

2. Reduce the diffusion coefficient through GFRPs by dispersing nanoclay in the matrix.

3. Study the effect of clay loading on the diffusion properties of GFRPs by varying the amount of clay (1, 2, and 5 percent by weight).

4. Study the effect of alkaline and saline solutions on the diffusion of water through GFRPs.

5. Take scanning electron microscope (SEM) images of damage at the surface of GFRP after prolonged immersion in distilled water.

6. Determine the changes in tensile strength and impact strength of vinyl ester-clay nanocomposites with varying amounts of nanoclay and correlate the results with available theories.

7. Use dynamic mechanical analysis (DMA) to study the structural changes due to immersion in distilled water for extended periods.

GFRP Rebars in CRCP Study

The objectives of this study are to (GangaRao, Chen, and Vijay 2003; Choi and Chen 2005):

1. Design and construct a CRCP that used GFRP bars for longitudinal reinforcement.

2. Evaluate the performance of GFRP-reinforced CRCP in relation to conventional steel-reinforced CRCP during the winter and summer seasons in terms of:

    - Crack spacing and crack width.

    - Crack-cluster ratio and percentage of "Y" cracking.

Both the GFRP-reinforced CRCP and the conventional steel-reinforced CRCP will be constructed on the same project and will share the same pavement thickness, base design, traffic loadings, and environmental conditions.

3. Check the existing design and construction methodologies given by the 1993 AASHTO Pavement Design Guide for the performance of conventional steel-reinforced CRCP and suggest necessary modifications for CRCP with GFRP reinforcement.

Project Design and Layout

GFRP Dowel Study

The study of GFRP dowels consists of both laboratory and field investigations. Laboratory tests conducted in this study include static and repeated load applications on 279.4- and 304.8-mm (11- and 12-in.) thick slabs containing both 25.4- and 38.1-mm (1.0- and 1.5-in.) diameter steel and GFRP dowels placed at 152.4- and 304.8-mm (6- and 12-in.) spacings.

For the field investigation, GFRP dowel bars were included the Corridor H project on Rt. 219, near Elkins WV. Both 25.4- and 38.1-mm (1.0- and 1.5-in.) diameter GFRP dowels were installed in the field at 152.4-,203.2-,228.6-, and 304.8-mm (6-, 8-, 9-, and 12-in.) spacings. Load calibrated field tests were conducted on these pavements in 2002 and 2003 using a WVDOT truck. The 38.1-mm (1.5-in.) diameter GFRP dowel bars were also utilized for pavement rehabilitation near the junction of Rt. 857 and Rt. 119 on University Avenue in Morgantown, WV. Field data collected through automatic data acquisition system included strain and joint deflections (which were used for assessing joint load transfer efficiency), relative deflections across the joints, and overall pavement performance.

Moisture Diffusion Study

Diffusion of water through GFRPs is being studied gravimetrically by performing transient water uptake experiments on glass fiber-reinforced and nonreinforced nanocomposites in distilled water, alkaline solutions, and saline solutions. A diffusion mechanism will be proposed for both glass fiber-reinforced and nonreinforced nanocomposites based on the experimental results. Equations will also be proposed to predict the changes in diffusion behavior with temperature. SEM images will be taken to estimate the damage of the fiber surface after extended exposure to distilled water, tensile tests and fracture tests will be carried out on nonreinforced nanocomposites with various clay loadings, and DMA tests will be carried out to study changes in glass transition temperature (T_g), storage modulus and loss modulus with clay loading. Results of these laboratory experiments will be correlated with available theories.

GFRP Rebars in CRCP Study

Analytical modeling and finite element (FE) analysis of CRCP reinforced with GFRP bars have been carried out at WVU (Vijay and GangaRao 1999; Chen and Choi 2002, 2003, 2004). Based on the laboratory and FE analysis results, designs for a GFRP-reinforced CRCP section and a conventional steel-reinforced CRCP control section have been developed, but a suitable project site has not yet been identified.

Current Project Status, Results, and Findings

Some preliminary findings and recommendations are available from the study of GFRP dowels (Li 2004; Vijay, GangaRao, and Li 2004). These are presented below.

Findings from GFRP Dowel Study

1. In this research, GFRP dowels were found to be acceptable alternatives to traditional steel dowels for transferring joint loads in JPCP pavements. Joints with GFRP dowels provided adequate deflection load transfer efficiency exceeding the minimum values recommended by AASHTO (75 percent) and ACPA (60 percent) in both laboratory tests and field tests.

2. GFRP dowel-concrete interfaces in slab #1 and slab #4 after 5 million load cycles were found to be in excellent condition with no visible damage, microcracking, or separation between the GFRP dowel and surrounding concrete.

3. The greater flexibility of the GFRP dowels results in a shorter distance to the point of inflection (no bending in the GFRP dowel) compared to steel dowels. The required length of 38.1-mm (1.5-in.) GFRP dowels is 65 percent of that for

a steel dowel with same diameter. The required length of a 25.4-mm (1-in.) GFRP dowel is 69 percent of that for a steel dowel with same diameter. The bearing stress at the joint face is greater with the GFRP dowel compared to a steel dowel.

4. Under static load testing, slabs with smaller diameter GFRP dowel and shorter spacings provided lower relative deflections than GFRP dowels with larger diameter and longer spacings. During fatigue loading up to 5 million cycles, the relative deflection of the slab with smaller GFRP diameter bars and shorter spacings and a 10.16-mm (0.4-in.) joint width increased from 0.3251 mm (0.0128 in.) to 2.0396 mm (0.0803 in.) over a range from 2 million to 5 million cycles, whereas the relative deflection of the slab with larger GFRP diameter bars and longer spacings and a 6.35-mm (0.25-in.) joint width decreased from 0.635 mm (0.025 in.) to 0.4318 mm (0.017 in.) over the range from 2 million to 5 million cycles.

5. The LTE of slabs with 25.4-mm (1-in.) diameter GFRP bars spaced at 152.4-mm (6-in.) intervals was 72 percent after 5 million load cycles. The LTE of slabs on a strong base and with 38.1-mm (1.5-in.) diameter GFRP bars spaced at 304.8-mm (12-in.) intervals was greater than 80 percent, whereas the same setup on slabs with poor base support conditions exhibited an LTE of 55 percent. Hence, it is very important to ensure adequate support conditions.

6. The LTE affects the slab integrity and slab stresses, whereas relative deflection affects ride comfort and impact on slab at joints. It is important to consider both LTE and relative deflections when the performance of JPCP is evaluated. For example, at 5 million cycles, the slab with 38.1-mm (1.5-in.) diameter GFRP dowels spaced at 304.8-mm (12-in.) intervals exhibited 55 percent LTE, but its relative deflection (0.4318 mm [0.017 in.]) was less than that of the slab with 25.4-mm (1-in.) diameter GFRP dowels spaced at 152.4-mm (6-in.) intervals (LTE of 72 percent and a relative deflection of 2.0396 mm [0.0803 in.]).

Findings from GFRP CRCP Study

The concrete stresses induced in a GFRP-reinforced concrete slab due to concrete shrinkage or temperature variations have been analytically calculated in comparison to those induced in a steel reinforced concrete slab, and the validity of the analytical model has been verified numerically and experimentally. The analytical solution indicates that a low Young's modulus of GFRP rebars results in a stress reduction in concrete slabs. The thermal stress in concrete can be either tensile or compressive, depending on temperature variations, the coefficient of thermal expansions (CTEs) of the concrete, and the type of GFRP reinforcement.

From comparison with the FE calculation, the analytical solution of average axial concrete stresses is shown to be valid throughout the longitudinal (x-) direction, especially in the vicinity of the slab's middle section, where the maximum average axial stresses in concrete appeared. Meanwhile, the comparison also reveals the applicability of the FE method to the CRCP analysis. A 5-ft CRCP segment FE model was built to simulate the behavior of the CRCP and to study the effects of the CRCP-design considerations (such as the CTE of concrete, the subbase friction, and the bond-slip between concrete and reinforcement) on the stress development and crack width in the CRCP. It is shown that using concrete with a lower CTE reduces the concrete's tensile stress level when exposed to a temperature drop, and a weaker bond between the concrete and reinforcement also decreases the concrete tensile stress level in the CRCP and the tensile reinforcement stress level at its cracks. With a higher subbase friction, the CRCP is more likely to crack from its bottom area, since the concrete stress level at the bottom will be higher than that at the top. This suggests that one can control crack spacing and crack widths of GFRP-reinforced CRCP by increasing the amount of reinforcement, reducing the bond between the concrete and reinforcement, or reducing the bond between the slab and the base.

From the FE study, a feasible longitudinal reinforcement design for a 254-mm (10-in.) GFRP-reinforced CRCP has been developed by evaluating concrete stress levels in 1-m (3.5-ft), 1.5-m (5-ft), and 2.4-m (8-ft) long CRCP-slab segments. Using #6 longitudinal GFRP rebars at 152.4-mm (6-in.) spacings at the mid-depth of the slab is shown to be an economically feasible design for GFRP-CRCP on the flexible subbase (or lime-treated clay subbase). In addition, #5 GFRP-rebars spaced at 48 in will be adequate as transverse reinforcement.

Higher CTE, higher Young's modulus, lower tensile strength, and greater drying shrinkage of the concrete appear to reduce the crack spacing in the CRCP. However, even though the crack width is generally narrower as the crack spacing is smaller, it can open wider after the crack formation stabilizes. This occurs when the ambient temperature and air humidity levels drop enough to cause significant concrete volume changes such that the crack width of those smaller crack spacing sections will become larger. Increased subbase friction levels always cause shorter crack spacings followed by a narrower crack width for all cases. From the mechanistic analysis results, either of the GFRP-CRCPs with the limestone, sandstone, and siliceous river gravel concrete on asphalt-stabilized subbase seems to perform satisfactorily without raising the GFRP-reinforcement ratio. In addition, the subbase that can provide a bond-slip of about 178.9 kPa/mm (5,000 lbf/in^2/in.) is able to guarantee a satisfactory performance of the GFRP-reinforced CRCP with granite concrete.

The designs of a 279.4-mm (11-in.) GFRP-reinforced CRCP and a conventional steel-reinforced CRCP sections were prepared in 2003. The final designs of the sections may vary depending upon the location of the experimental sections.

Points of Contact

Hota V. S. GangaRao, Ph.D., P.E., Professor
(304) 293-7608, ext. 2634
Hota.Gangarao@mail.wvu.edu

Hung-Liang (Roger) Chen, Ph.D., Professor
(304) 293-3031, ext. 2631
hchen@wvu.edu

P. V. Vijay, Ph.D., Assistant Professor
(304) 293-7608, ext. 2660
pvijay@wvu.edu

Constructed Facilities Center
College of Engineering and
Mineral Resources
West Virginia University
Morgantown, WV 26506

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Chapter 36. WISCONSIN 1 (Highway 29, Abbotsford)

Introduction

In the early 1990s, the Wisconsin Department of Transportation (WisDOT) began investigating noise levels produced by concrete pavement finishing practices (Kuemmel et al. 2000). In 1994, Wisconsin constructed 16 concrete pavement sections incorporating different experimental textures, the results of which led to some preliminary recommendations on concrete pavement surface texturing (Kuemmel et al. 1996).

At about the same time, several other states (Colorado, Iowa, Michigan, Minnesota, and North Dakota) constructed concrete pavements with experimental surface texturing, and there arose an acute need to uniformly collect and analyze the data from all of the test sections and develop national guidelines for recommended surface texturing practices. To address this need, WisDOT and FHWA contracted with Marquette University and the HNTB Corporation to collect noise, texture, and friction characteristics from the now six states and prepare a final report documenting the findings (Kuemmel et al. 2000). Concurrently, under the TE-30 program, additional concrete pavement test sections were constructed in Wisconsin to afford a more complete range of surface textures (Kuemmel et al. 2000). These new test sections were constructed in the westbound lanes of Highway 29, west of Abbotsford (see Figure 96), and are located just east of the original 1994 Wisconsin test sections (Kuemmel et al. 2000).

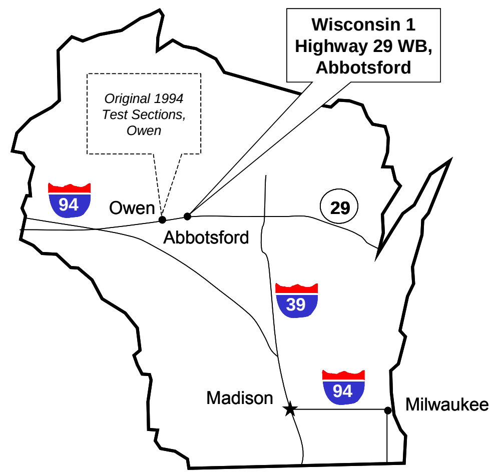

Figure 96. Location of WI 1 project.

Study Objectives

The major objective of this project is to develop national guidelines for texturing PCC pavements based on national experience (Kuemmel et al. 2000). This objective was to be accomplished by performing the following (Kuemmel et al. 2000):

- Construction of 10 additional test sites in Wisconsin.
- Measurement of interior and exterior noise levels, surface friction, and texture on test sites in Wisconsin, Colorado, Iowa, Michigan, Minnesota, and North Dakota, as well as on 8 of the original 16 Wisconsin test sections.
- Application of Fast Fourier Transform (FFT) methods for both interior and exterior noise measurements to help resolve discrepancies in subjective noise measurements.
- Assessment of public perception to road noise through the use of standard audiology testing by a panel of non-highway agency raters.
- Development of recommended pavement textures and demonstration at an open house and workshop near the new Wisconsin test sections.

Project Design and Layout

New Wisconsin Sections

The 10 new Wisconsin test sections were constructed in 1997 in the westbound lanes of Highway 29, west of Abbotsford (Kuemmel et al. 2000). These sections are all 1.6 km (1 mi) long and share the same pavement design, differing only surface texture. On this project, the following 10 surface textures were constructed (Kuemmel et al. 2000):

1. Random transverse tining, 25-mm (1-in.) average spacing (Random 1).
2. Random transverse tining, 19-mm (0.75-in.) average spacing (Random 2).
3. Uniform transverse tining, 25-mm (1-in.) spacing.
4. Random 1 skewed tining, 1:6 left hand forward (LHF).
5. Random 2 skewed tining, 1:6 LHF.
6. Random 1 skewed tining, 1:4 LHF.
7. Random 2 skewed tining, 1:4 LHF.
8. Random 1 longitudinal tining.
9. Random 2 longitudinal tining.
10. Uniform longitudinal tining, 25-mm (1-in.) spacing.

All texturing was preceded by a longitudinal turf drag (LTD) and all tining depths were specified to be 3 mm (0.12 in.) (Kuemmel et al. 2000).

Additional Analysis Sections

Several additional sections, both within Wisconsin and from other states, were brought into the study to expand the analysis. Characteristics of these sections are described below.

Additional Wisconsin Sections

As previously noted, 8 of the 16 original 1994 Wisconsin sections were included in this evaluation; six other experimental pavement sections from I-43 and four other sections from throughout the State were also included (Kuemmel et al. 2000). The surface texturing of these additional Wisconsin sections are described below (Kuemmel et al. 2000):

- Highway 29, Owen
  - 25 mm (1 in.) uniform longitudinal
  - 25 mm (1 in.) uniform skewed, 1:6 LHF, 1.5 mm (0.06 in.) depth.
  - 13 mm (0.5 in.) uniform transverse.
  - 13 mm (0.5 in.) uniform transverse, 1.5 mm (0.06 in.) depth.
  - 19 mm (0.75 in.) uniform transverse.
  - Manufactured random.
  - 25 mm (1 in.) uniform transverse (former WisDOT standard)
  - Skidabrader (a blasted, set, but uncured PCC pavement with longitudinal turf drag only prior to treatment).
- I-43, Milwaukee
  - SHRP AC Pavement (built in 1992)
  - Standard AC dense-graded pavement (1992)
  - Standard AC dense-graded pavement (1993).
  - Stone matrix asphalt (SMA) pavement, 16 mm (0.62 in.) top size (1993).
  - Diamond ground PCC pavement (built in 1978, ground in 1993).
  - SMA pavement, 9 mm (0.38 in.) top size aggregate (1992).
- Highway 29 eastbound, random transverse tining, 21 mm (0.83 in.) average spacing (1994).
- US 51 north of Merrill, random transverse tining, 25 mm (1 in.) average spacing (1996).
- US 151 near Beaver Dam, random transverse tining, 25 mm (1 in.) average spacing (1996).
- Highway 26 near Jefferson, random transverse tining, 25 mm (1 in.) average spacing (1996).

Colorado Sections

The Colorado DOT constructed nine test sections on I-70 near Deertrail in 1994, six of which were included in this study (Kuemmel et al. 2000):

- 25 mm (1 in.) uniform transverse tining (Colorado DOT's then standard).
- 19 mm (0.75 in.) average random transverse tining (repeating pattern of 16, 22, and 19 mm [0.63, 0.87, and 0.75 in.]).
- 13 mm (0.5 in.) uniform transverse tining.
- 19 mm (0.75 in.) average random transverse saw cut (repeating pattern of 16, 22, and 19 mm [0.63, 0.87, and 0.75 in.]).
- 19 mm (0.75 in.) uniform longitudinal saw cut.
- 19 mm (0.75 in.) uniform longitudinal tining.

LTD preceded all surface texturing, and the impressions were specified to be 3 mm (0.12 in.) deep and 3 mm (0.12 in.) wide (Kuemmel et al. 2000).

Iowa Sections

In 1993, the Iowa DOT constructed nine test sections on Highway 163 northeast of Des Moines, and the following seven sections were selected for inclusion in this study (Kuemmel et al. 2000):

- 13 mm (0.5 in.) uniform transverse, 3 to 5 mm (0.12 to 0.2 in.) deep.
- 19 mm (0.75 in.) uniform transverse, 3 mm (0.12 in.) deep.
- 19 mm (0.75 in.) uniform longitudinal, 1.5 mm (0.06 in.) deep.
- 19 mm (0.75 in.) uniform longitudinal, 3 to 5 mm (0.12 to 0.2 in.) deep.
- 19 mm (0.75 in.) variable transverse, 3 to 5 mm (0.12 to 0.2 in.) deep.
- Milled PCC pavement (carbide ground).
- 13 mm (0.5 in.) uniform transverse, sawed.

All texturing was preceded by LTD.

Michigan Sections

The two sections in the Michigan I-75 experimental project (see chapter 10) were selected for inclusion in this study (Kuemmel et al. 2000). These include both a standard texture (25-mm [1-in.] uniform transverse tining) and a European exposed aggregate surface.

Minnesota Sections

Eight pavement sections with various surface texturings were included from Minnesota. LTD preceded all texturing activities for the 8 selected sections (Kuemmel et al. 2000):

- US 169 section with 19 mm (0.75 in.) uniform longitudinal tining (built in 1996).US 169 section with 19 mm (0.75 in.) random transverse tining (1996).
- US 12 section with 19 mm (0.75 in.) random transverse tining (1996)
- Highway 55 with 38 mm (1.50 in.) random transverse tining (built in 1994).
- I-494 section with longitudinal turf drag only (1990).
- US 169 section with 38 mm (1.50 in.) random transverse tining (1994).
- US 169 section with LTD only (1996).
- US 169 section with 19 mm (0.75 in.) uniform longitudinal tining (1996).

North Dakota Sections

In 1994, North Dakota constructed nine test sections incorporating different surface texturing on I-94 near Eagle's Nest, of which six were selected for this study (Kuemmel et al. 2000):

- 25 mm (1 in.) uniform skewed tining, 1:6 RHF.
- 19 mm (0.75 in.) uniform transverse tining.
- Variable (26, 51, 76, and 102 mm [1, 2, 3, and 4 in.]) random transverse tining.
- 13 mm (0.5 in.) uniform transverse tining.

- 19 mm (0.75 in.) uniform longitudinal tining.
- 25 mm (1 in.) uniform transverse tining (for control).

All surface texturing was preceded by LTD.

State Monitoring Activities

A total of 57 pavement test sections from 6 states were ultimately included in the study. Representing a wide variety of surface textures, each of these pavement sections was evaluated in 1999 for the following properties (Kuemmel et al. 2000):

- Interior and exterior noise levels measured with a real-time acoustical analyzer using the FFT analysis.
- Subjective rating of interior noise levels by 24 people on 21 of the test sections. The subjective ratings were performed on digital recordings of interior noise levels made during the interior noise measurements.
- Surface texture measurements using the FHWA's Road Surface Analyzer (ROSAN). Computations of mean profile depth (MPD) and estimated texture depth (ETD) were obtained from the measurements. Sand patch testing, providing an estimate of the texture depth, was also conducted on most test sections in Wisconsin.
- Surface friction collected by the participating SHAs. This testing was conducted in accordance with ASTM E274 and was conducted in close proximity to the time that the texture and noise measurements were obtained.

Results/Findings

A comprehensive analysis of the noise, texture, and surface friction data collected from the 57 sections was conducted, and yielded the following conclusions (Kuemmel et al. 2000):

- The depth of the tining varies considerably between pavement test sections, as well as within a test section. In many cases, specified tining depths were not achieved.
- Uniformly tined pavements exhibit discrete frequencies that produce an annoying "whine" to travelers.
- Transversely tined pavements with the widest and deepest textures were often among the noisiest.
- AC and longitudinally tined PCC pavements exhibit the lowest exterior noise levels while still providing adequate texture. Longitudinal uniform spacings of 19 mm (0.75 in.) reduce the impact on motorcycles and compact vehicles. However, splash and spray have been noted to be greater on longitudinally tined pavements.
- The longitudinally tined PCC pavements, the randomly skewed (1:6) tined pavements, and an AC pavement exhibit the lowest interior noise levels while still providing adequate texture.
- Random transverse tining is very sensitive to the spacing pattern. When the spacings become more uniform, discrete frequencies may develop, resulting in an objectionable whine.
- Randomly skewed (1:6) tined pavements can be constructed relatively easily, exhibit low interior noise levels and no discrete frequencies, and have the best subjective ranking. They have higher levels of exterior noise than longitudinally tined PCC and AC pavements, but lower than random transverse PCC pavements. They also exhibit good friction and texture.
- The diamond-ground PCC pavement, although not as quiet as other PCC pavements, did not exhibit any predominant frequency or spike.

Figure 97 summarizes the surface friction data (measured at 64 km/hr [40 mi/hr] and with a bald tire) collected for the Wisconsin 1 sections (Kuemmel et al. 2000). All but one of these sections exhibit friction numbers greater than 40, with the lowest exhibited by Section 8 (random longitudinal tining).

Figure 98 shows the relative ranking of the noise levels for all of the different pavement surface textures in terms of subjective ratings, interior noise levels, and exterior noise levels (Kuemmel et al. 2000).

Based on the results of the data analysis, the following primary recommendations were developed (Kuemmel et al. 2000):

- If overall noise considerations are paramount, longitudinal tining that provides satisfactory friction may be considered. A uniform tine spacing of 19 mm (0.75 in.) will provide adequate friction and, according to other studies, will minimize effects on small tired vehicles. However, the safety aspects of longitudinal tining have not been documented.

- If subjective perceptions and texture considerations are paramount, a randomly skewed (1:6) textured pavement, offset the opposite of any skewing of the transverse joints, may be used. This pattern achieves the texture and friction of a conventional transversely tined pavement while also obtaining most of the noise reductions associated with longitudinal tining.

- If texture considerations are paramount, and a skewed pattern is impractical, randomly spaced transverse tining may be employed. However, this should be carefully designed and built using a highly variable spacing. A 3-m (10-ft) long rake with spacings between 10 and 76 mm (0.4 and 3.0 in.), designed using spectral analysis, is recommended, and has been successfully tested by three states.

- Diamond grinding, if sufficiently deep to remove most of the uniform transverse texture, can be considered a treatment for PCC pavements with excessive whine.

It should be noted that at least one State evaluated the surface texture recommendation presented above, but did not achieve low-noise pavements (Scofield 2003). A new technical advisory on pavement surface texture is currently under development by the FHWA.

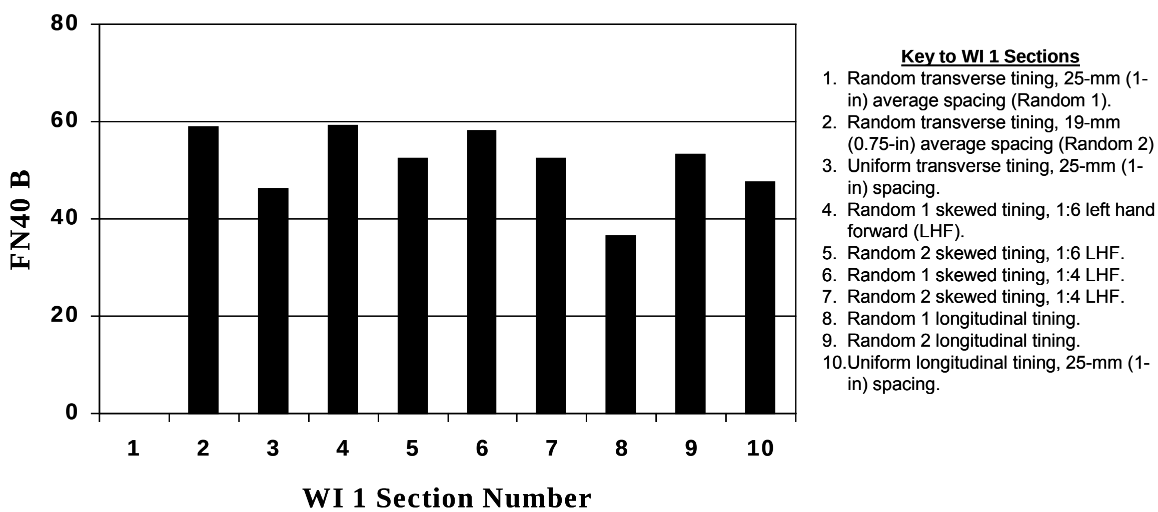

Figure 97. Surface friction of WI 1 test sections (Kuemmel et al. 2000).

Points of Contact

Debbie Bischoff
Wisconsin Department of Transportation
3502 Kinsman Boulevard
Madison, WI 53704
(608) 246-7957
debra.bischoff@dot.state.wi.us

David A. Kuemmel
(414) 288-3528
david.kuemmel@marquette.edu

Ronald C. Sonntag
(414) 288-5734
ronald.sonntag@marquette.edu

Marquette University
Department of Civil and
Environmental Engineering
P.O. Box 1881
Milwaukee, WI 53201-1881

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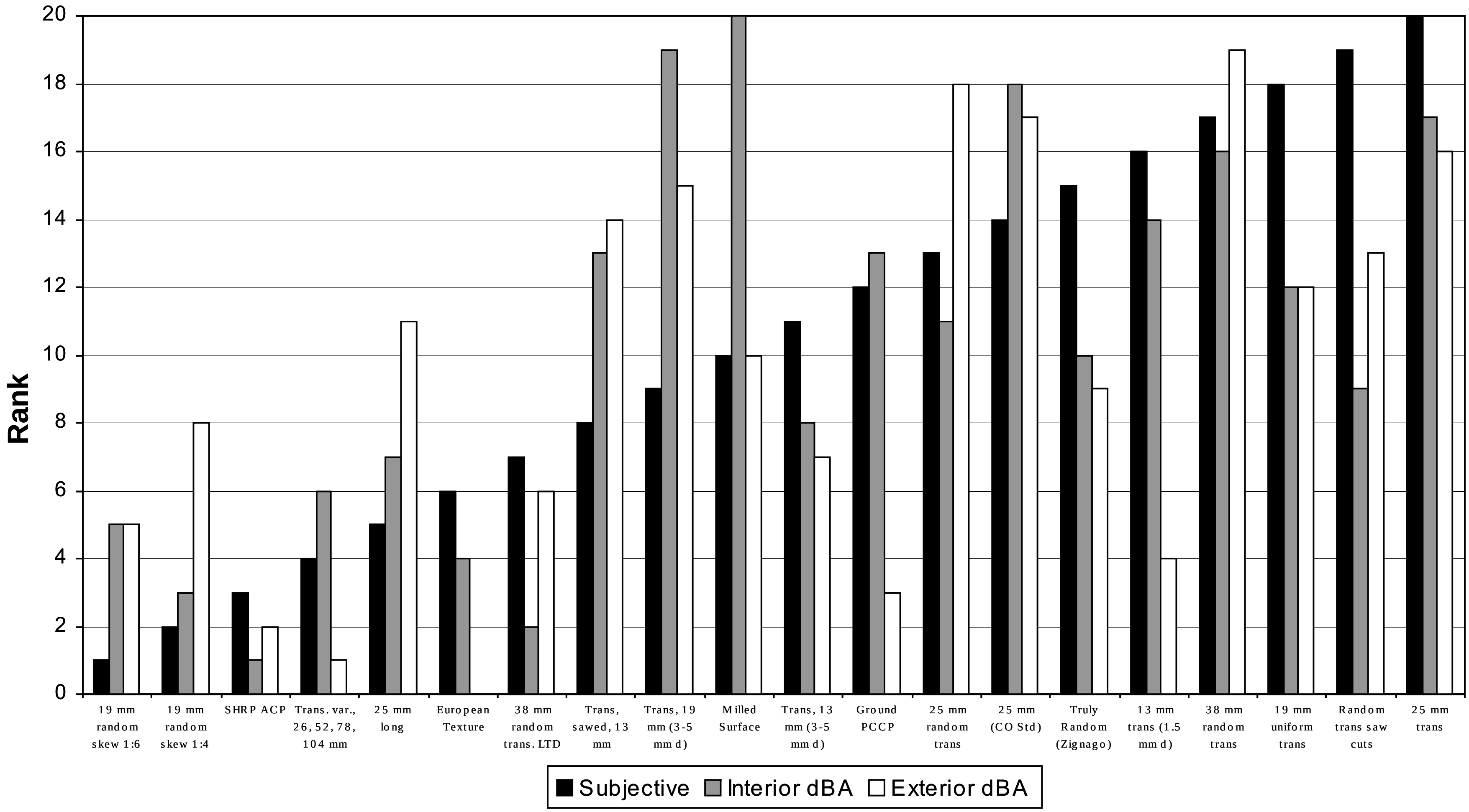

Figure 98. Relative noise rankings of pavement surface texturings (Kuemmel et al. 2000).

Chapter 37. 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 constructibility 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 99). 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.

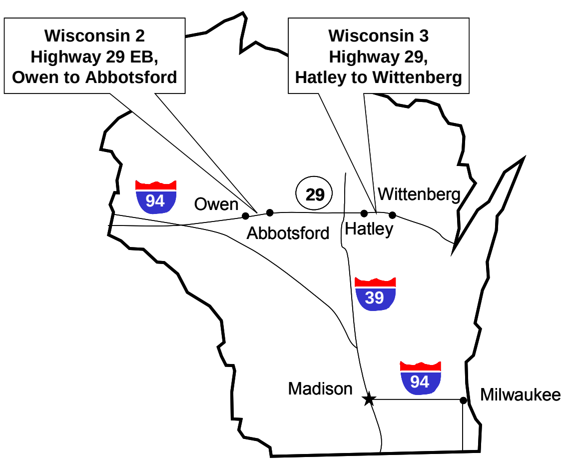

Figure 99. Location of WI 2 and WI 3 projects.

Study Objectives

The overall objective of these projects is to evaluate the constructibility 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 used on this project are illustrated in Figure 100. These layouts were selected to reduce dowel bar requirements while still maintaining standard placement locations used in Wisconsin (Crovetti 1999).

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 101 shows the approximate layout of the 11 test and 2 control sections in the WI 2 project, using the original section nomenclature adopted by the researchers. Nominal 161-m (528-ft) long pavement segments generally consisting of 29 joints were selected from within each test section for long-term monitoring (Crovetti 1999). Table 54 provides the experimental design matrix for the project.

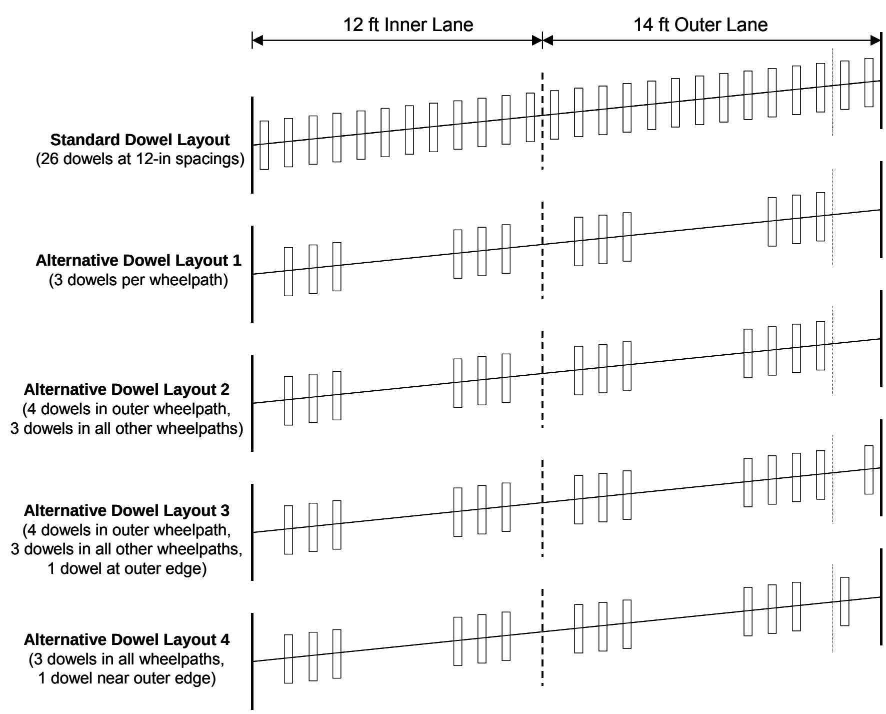

Figure 100. Alternative dowel bar layouts used on WI 2.

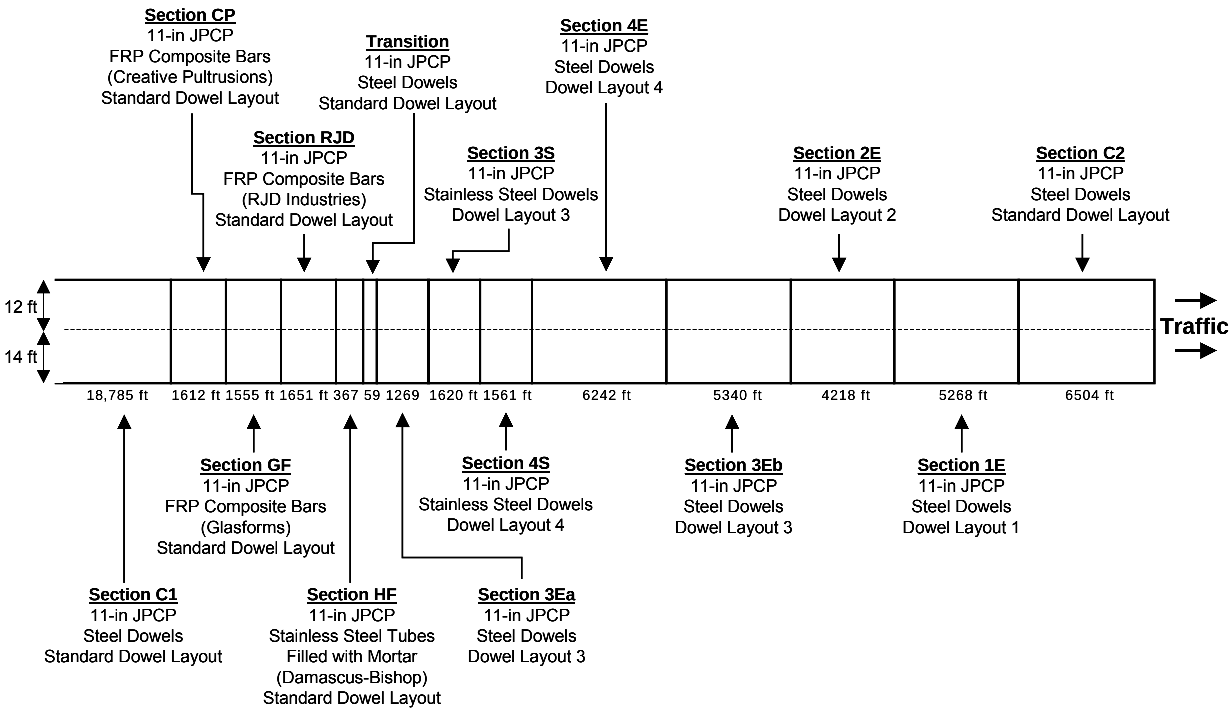

Figure 101. Approximate layout of WI 2 test sections.

Table 54. 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 102). The goal is the more efficient use of materials in areas subjected to greater traffic loading, resulting in more cost-effective designs.

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.

Six sections are included in the WI 3 project. The approximate layout of the WI 3 sections being monitored is shown in Figure 103. All dowel bars were placed on baskets prior to paving (Crovetti 1999). 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 103.

The experimental design matrix for the WI 3 project is shown in Table 55. 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 55 for each section.

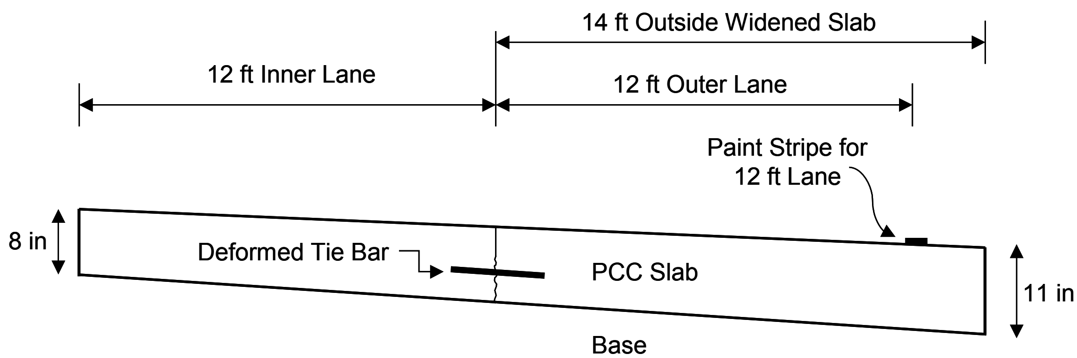

Figure 102. Variable cross section used on WI 3.

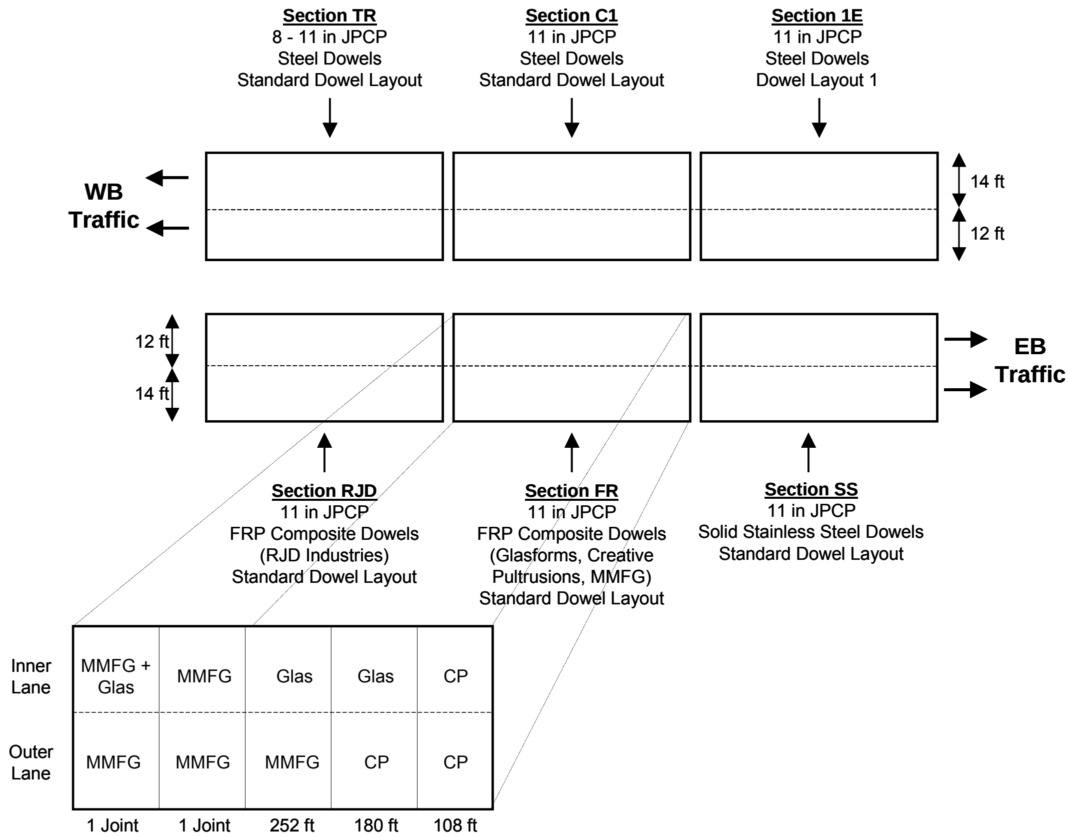

Figure 103. Approximate layout of WI 3 monitoring sections.

Table 55. 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. Four types of monitoring activities are used (Crovetti 1999; Crovetti and Bischoff 2001):

- Dowel bar location study—2 months after construction.
- FWD testing—immediately prior to paving, immediately after paving, and after 6 and 12 months of trafficking.
- Distress surveys—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—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 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 study results are provided in Table 56 (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 56. 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 57 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.

Transverse joint load transfer efficiencies were also measured on all test sections using the FWD. Figure 104 illustrates the average transverse joint load transfer for the outermost wheelpath of the WI 2 project, while Figure 105 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). The 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). Overall, the low LTE values of the FRP bars (between 60 and 75 percent in many cases) is a cause of concern.

For WI 3, Figure 105 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). Again, however, the lower LTE values for the FRP bars are a concern.

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 106 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).

Table 57. Summary of FWD Test Results for WI 2 and WI 3 Projects (Crovetti 1999).

	WI 2			WI 3				
	EB LANES			EB LANES			WB LANES	
PROPERTY	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

Points of Contact

Debbie Bischoff
Wisconsin Department of Transportation
3502 Kinsman Boulevard
Madison, WI 53704
(608) 246-7957

James A. Crovetti
Marquette University
Department of Civil and Environmental Engineering
P.O. Box 1881
Milwaukee, WI 53201-1881
(414) 288-7382

References

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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.

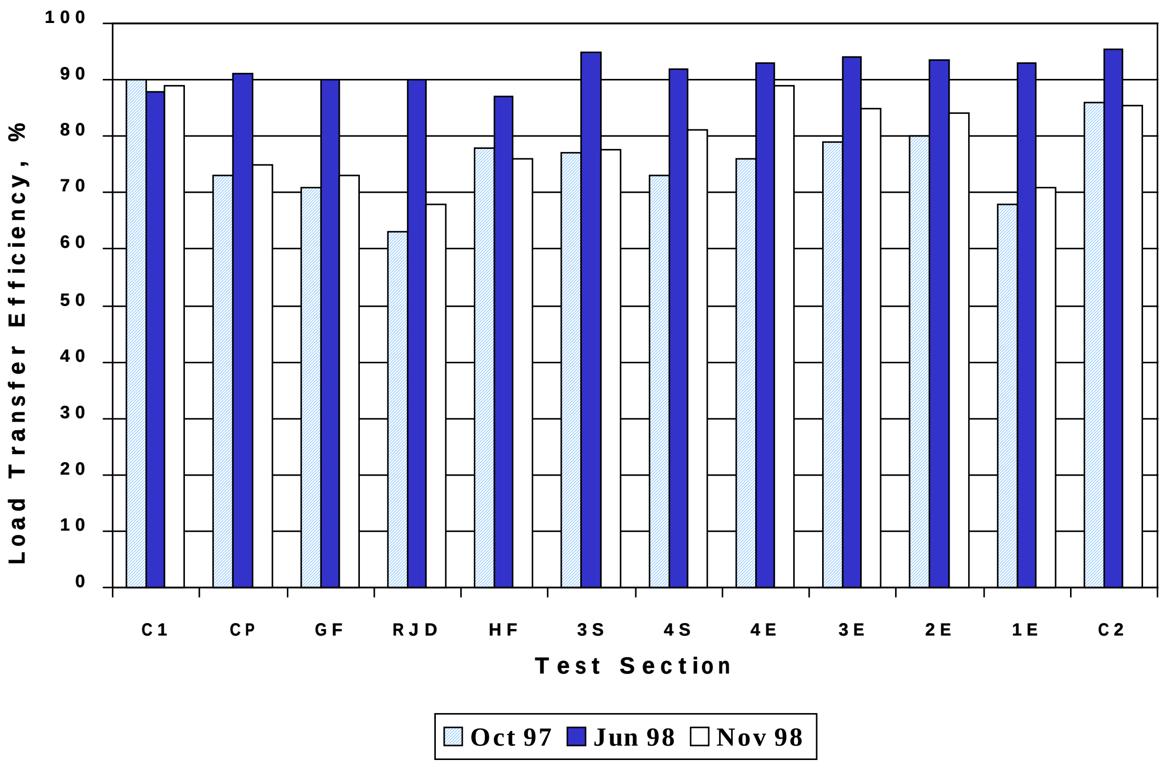

Figure 104. Transverse joint load transfer for outermost wheelpath on WI 2 (Crovetti 1999).

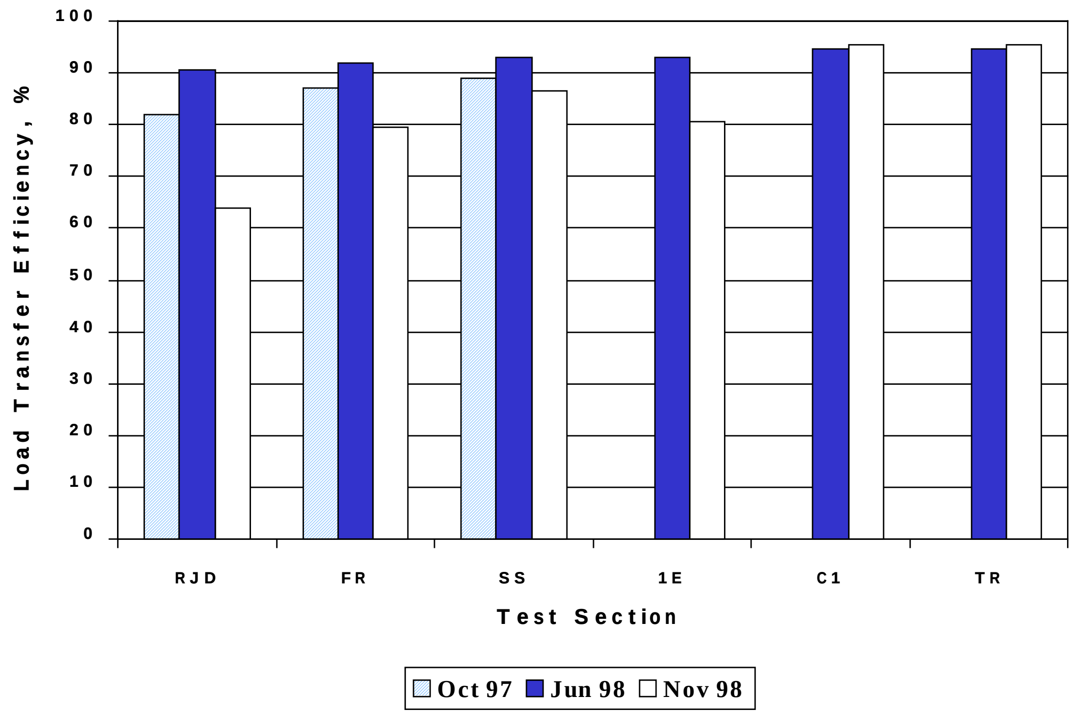

Figure 105. Transverse joint load transfer for outermost wheelpath on WI 3 (Crovetti 1999).

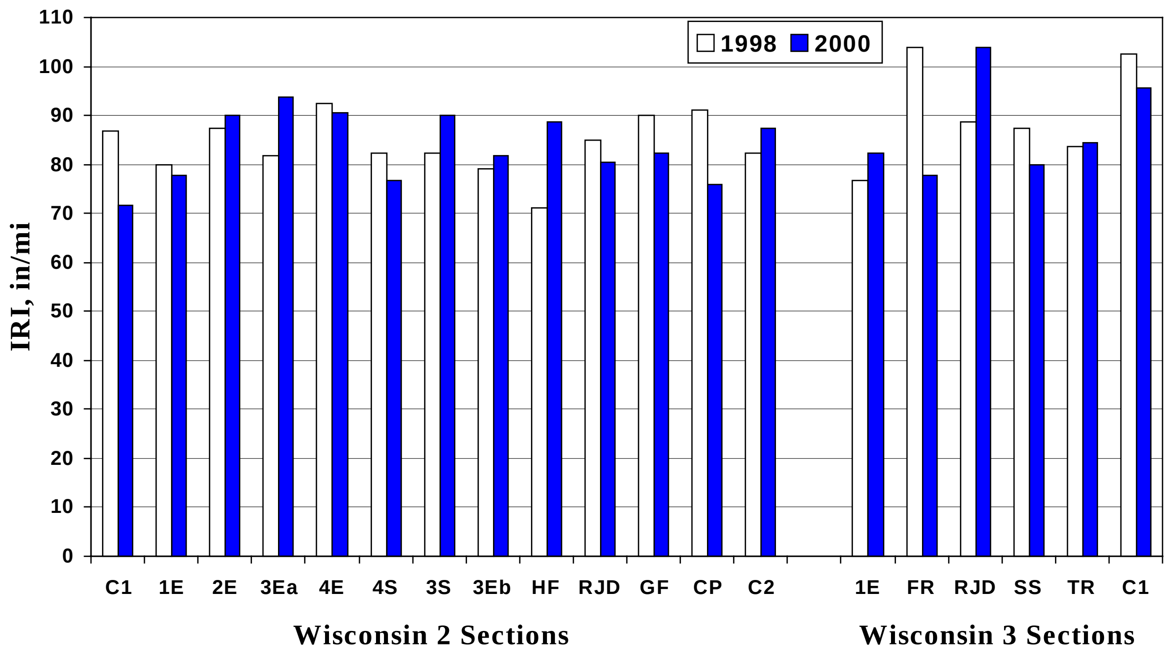

Figure 106. Average IRI values in the outer traffic lanes of WI 2 and WI 3 pavement sections (Crovetti and Bischoff 2001).

Chapter 38. WISCONSIN 4 (I-90, Tomah)

Introduction

In September 2002, the Wisconsin Department of Transportation (WisDOT), constructed a high performance concrete pavement section with a design life of 50 years, with no maintenance anticipated over that period. The project is located on I-90 in Monroe County, immediately west of the intersection with STH 16 and approximately 12.9 km (8 mi) west of the split in Tomah, of I-90 and I-94. The test sections are located in the eastbound lanes with the termination of the test sections at structure B-41-111. The general project location is illustrated in Figure 107.

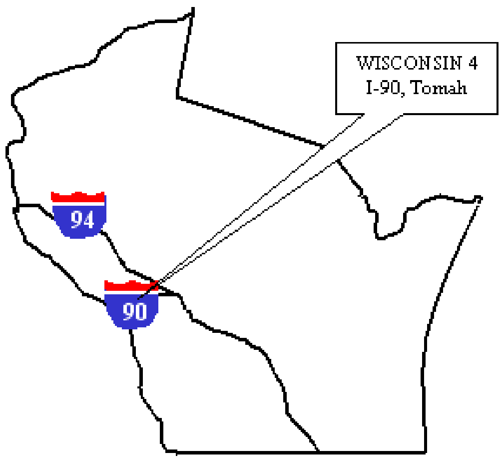

Figure 107. Location of WI 4 project.

Study Objectives

The objective of this study was to construct a pavement that would last for 50 years. Specific design factors considered were:

- The use of deicing agents, especially salts, may cause premature pavement failure due to determination of the load transfer devices made of plain and epoxy-coated steel.
- Slab curling is also a cause of premature pavement failure. One key to a long-life pavement is that the pavement is built upon a firm foundation and remains in contact with such. Concrete pavements respond to the environment by changing in three ways: built-in curvature, temperature curling, and moisture warping. When the slab is curled away from the foundation premature cracking may result from stresses incurred.

Some of the benefits of providing this type of technology to Wisconsin are the anticipated lower long-term pavement maintenance costs and reduced rehabilitation and replacement costs due to a longer initial service life. Wisconsin is also beginning to focus on user cost delays. If a longer lasting pavement is developed and placed, user cost delays will decrease as well as increasing safety by reducing the exposure of construction and maintenance crews to traffic.

Project Design and Layout

The test sections are located in the two eastbound lanes of I-90. The project was split into two test sections and one control section as follows (Kemp 2004):

- Test section 1 is 227 m (745 ft) long, with two 3.6-m (12-ft) lanes and tied concrete shoulders (inside shoulder 1.2 m (4 ft) wide and outside shoulder 3 m (10 ft) wide).
- Test section 2 is 242.3 m (795 ft) long, with two 3.6-m (12-ft) lanes and tied concrete shoulders (inside shoulder 1.2 m [4 ft] wide and outside shoulder 3 m [10 ft] wide). Stainless steel dowels were used for the load transfer devices utilizing a non-corrodible basket system for the test sections. Shoulders were tied to the pavement using stainless steel tie bars.
- The control section is 242.3 m (795 ft) long, with two 3.6-m (12-ft) lanes and asphaltic shoulders (inside shoulder 1.2 m [4 ft] wide and outside shoulder 3 m [10 ft] wide). Epoxy-coated dowels placed in standard steel baskets were used for load transfer at the joints.

The current WisDOT pavement design guidelines specify a structure with the following characteristics will be suitable for pavements with a 20-year pavement life:

- 304.8-mm (12-in.) thick doweled JPCP.
- 101.6-mm (4-in.) open-graded base course.
- 152.4-mm (6-in.) dense-graded base course.

The 50-year design had the following characteristics:

- 342.9-mm (13.5-in.) doweled JPCP.
- 101.6-mm (4-in.) open-graded base course (No.2).
- 152.4-mm (6-in.) dense-graded base course.

In addition to the standard design, an increased base section was engineered to carry the improved pavement section. This is to provide a more static base than the standard base and original subgrade. The section has the following properties:

- 342.9-mm (13.5-in.) doweled JPCP.
- 101.6-mm (4-in.) open-graded base course (No. 20).
- 254-mm (10-in.) dense-graded base course (a two-layer system with the first 152.4 mm [6 in.] being a 76.2-mm [3-in.] maximum size and the remaining 101.6-mm [4-in.] being a 38.1-mm [1.5-in.] maximum size, well graded base course).
- 406.4-mm (16-in.) of breaker run.

State Monitoring Activities

The test section is monitored using Wisconsin's Pavement Monitoring Van. This van is equipped with laser sensors to measure the profile. The laser measurements are used to calculate an International Roughness Index (IRI). The van is also equipped with video cameras that are able to log the section of the highway of interest, and later analyzed for pavement distress. The results of the analysis are used to calculate a Pavement Distress Index (PDI)

A fatigue life will be calculated based upon a back-calculated elastic module. This is calculated from a FWD testing results performed on the various sections.

Rapid chloride permeability tests will be performed at 28, 90, and 365 days to indicate the concrete's resistance to infiltration of salts. In addition, the standard tests of air content, slump, and compressive strength will be monitored at 3, 7, 28, 90, and 365 days. Thermocouples were installed to monitor the temperature of the slab during the curing period along with documentation of the temperature of the base course.

The FHWA Office of Pavement Technology will evaluate the slab in relationship to warpage and curl during the initial curing period. In addition to these tests, regular visual inspections will be conducted and traffic and weather data will be recorded from an automated site located near the project.

Preliminary Results/Findings

The pavement sections were constructed on September 24, 2002. The concrete was a standard mix design utilizing local materials. Type I cement was used in conjunction with a Class C fly ash at a substitution rate of 25 percent. Dowel bars were 38.1-mm (1.5-in.) diameter Type 316 solid stainless steel, 457.2 mm (18 in.) long, and spaced across the joints at 304.8-mm (12-in.) spacings. The transverse joints are perpendicular and spaced at fixed 4.6-m (15-ft) intervals.

The dowels were placed in baskets on the grade. The baskets were constructed from stainless steel wire. The wire had a diameter of 3.2 mm (0.125 in.), which made the baskets flexible. To compensate for this flexibility, the baskets were cut in half on the jobsite. Epoxy-coated steel tie bars were then driven in the ground in front of the baskets to keep the frame from moving forward when the paver passed over them (see Figure 108). Additionally, a concrete spreader deposited 1.5 to 2.3 m^3 (2 to 3 yd^3) of wet concrete on top of the basket assemblies prior to the slip-form paver moving over the dowel baskets. Visual inspection during the paving along with verification from a dowel bar locater confirmed that the dowels had stayed in place. Dowel baskets, for future projects, should be constructed with a 4.8-mm (0.1875-in.) minimum diameter wire to avoid these issues.

Test section 1 was paved shorter than originally planned, and totaled 227 m (745 ft). A construction joint extended into the test section by 15.2 m (50 ft). This joint is to allow the doweled JPCP to abut to the existing CRCP. Stainless steel tie bars for adjoining the shoulder were used in the test sections. The spacing was unintentionally altered from 30 in center-to-center spacing to 24 in center-to-center spacing. This caused the addition used of epoxy-coated tie bar in the last 21.3 m (70 ft) of test section 2.

Points of Contact

Peter Kemp
New Products/New Methods Engineer
(608) 246-7953
peter.kemp@dot.state.wi.us

Steven Krebs
Chief Pavements Engineer
(608) 246-5399

David Leo Larson
(608) 246-7950

Wisconsin Department of Transportation
3502 Kinsman Boulevard
Madison, WI 53704

Reference

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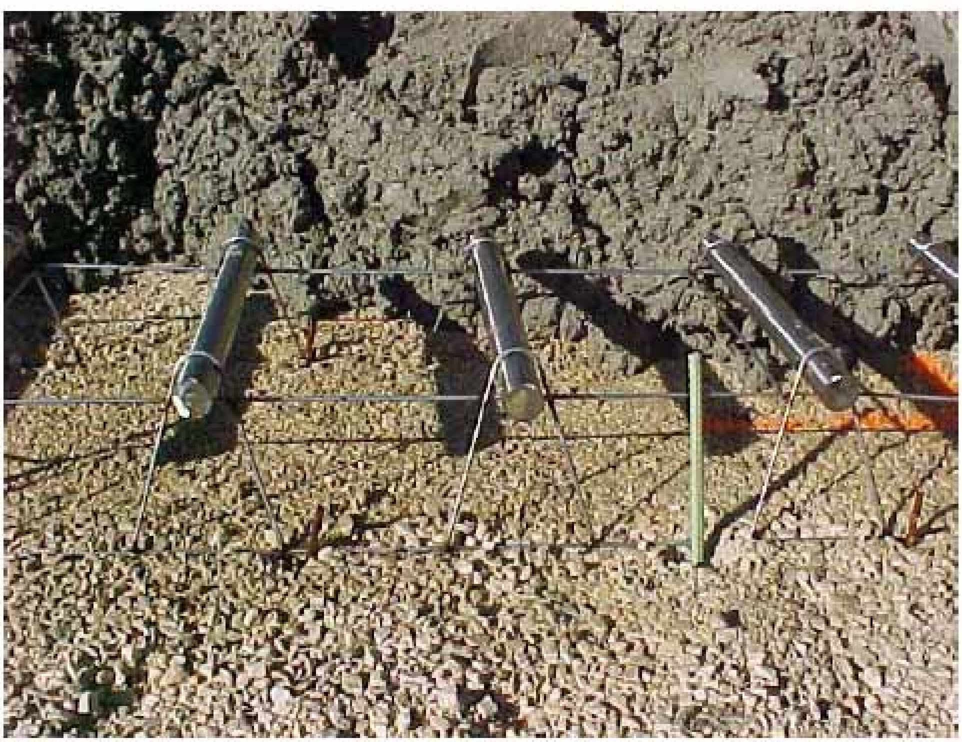


Figure 108. Stainless steel dowels and basket on WI 4 (Kemp 2004).

Chapter 39. FEDERAL HIGHWAY ADMINISTRATION 1 (FHWA's Pavement Test Facility, McLean, VA)

Introduction

A joint venture between FHWA and the American Concrete Pavement Association (ACPA) was initiated in 1998 to study the design and performance of ultrathin whitetopping (UTW) overlays. The research consisted of constructing eight 15.2-m (50-ft) test lanes of UTW placed on an existing hot mix asphalt (HMA) surface. Loading of the test sections occurred between May 1998 and November 1999.

Study Objectives

The objective of the study was to validate the design equations and performance prediction models used in the ACPA UTW design procedure. The specific research objectives were as follows:

- Evaluate UTW performance under controlled wheel-loads and temperature.
- Study the effects of a range of design features (thickness, joint spacing, and fiber reinforcement) on the performance of UTW.
- Measure pavement response and develop mechanistic models based on these responses.
- Verify existing models and develop new models for predicting load-carrying capacity.

Project Design and Layout

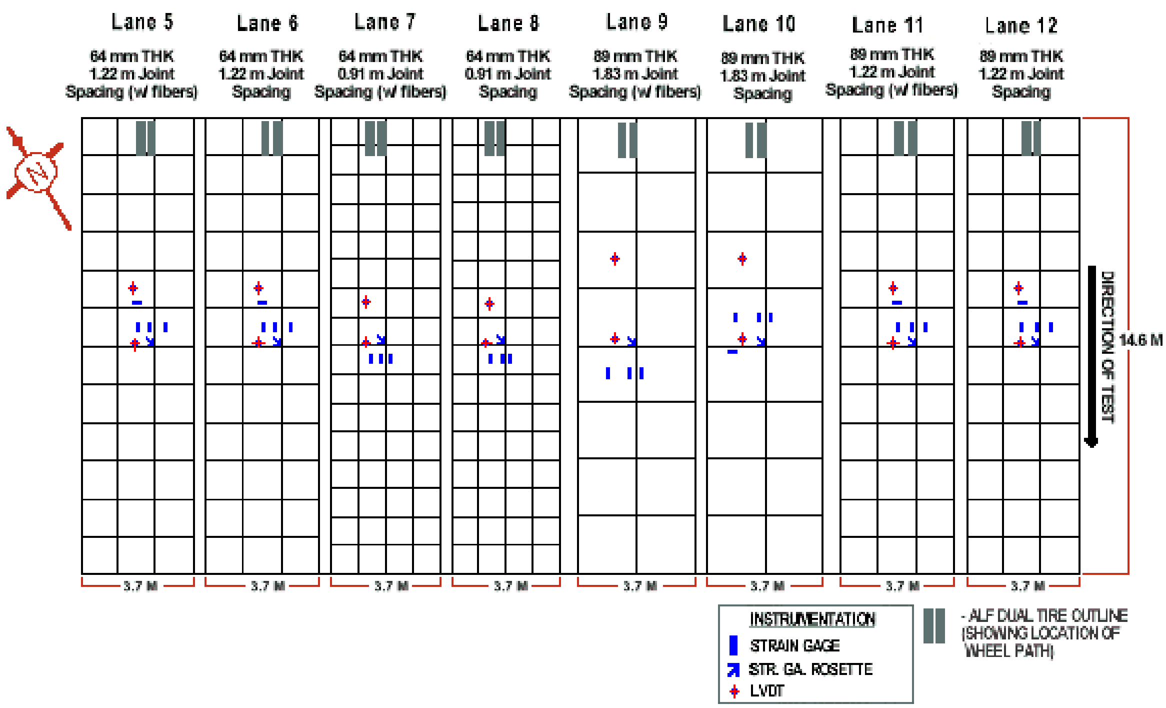
The test sections were loaded by two accelerated load testing machines capable of applying simulated truck loads under a controlled pavement temperature. A single wheel load of either 10 or 12 kips was applied, and the speed of the loaded tire was a constant 16 km/hr (10 mi/hr). An average of 35,000 loads were applied per week. The HMA was 5 years old and a range of severities of rutting were present prior to milling. The HMA was milled to a depth of either 63.5 or 88.9 mm (2.5 or 3.5 in.) depending on the depth of the rutting that was present. The thickness of the overlay was equal to the milled depth so the existing elevation could be maintained. The design parameters for each test section are provided in Table 58, and a layout of the test sections is provided in Figure 109.

Table 58. Test Section Design Parameters for FHWA 1 Project (CTL 2001)

LANE NO.	PCC OVERLAY THICK (IN.)	HMA THICK (IN.)	HMA MIXTURE, BINDER[1]	JOINT SPACING (FT)	FIBERS IN CONCRETE MIXTURE	DESIGN 12-KIP LOAD APPLICATIONS[2]
5	2.5	5.5	SM, AC-10	4 x 4	Yes	210,000
6	2.5	5.5	SM, AC-20	4 x 4	No	140,000
7	2.5	5.5	SM, PM	3 x 3	Yes	420,000
8	2.5	5.5	SM, PM	3 x 3	No	350,000
9	3.5	4.5	SM, AC-5	6 x 6	Yes	350,000
10	3.5	4.5	BM, AC-20	6 x 6	No	245,000
11	3.5	4.5	BM, AC-5	4 x 4	Yes	455,000
12	3.5	4.5	BM, AC-20	4 x 4	No	350,000

[1]SM = surface mixture; BM = base mixture; PM = polymer modified asphalt.

[2]Estimated design life was calculated using ACPA design procedure based on as-built strengths, overlay design thicknesses, and 12-kip loads.



ALF = accelerated loading facility; LVDT = linear value displacement transducers; THK = thick

Figure 109. Layout of test sections and instrumentation for FHWA 1 project (FHWA 2004).

The test sections were instrumented with between 15 and 18 dynamic strain gauges. The gauges were installed near the top and bottom of the concrete overlay and on the surface of HMA. Deflections were measured in the interior of the slab and adjacent to the joint using linear variable displacement transducers.

State Monitoring Activities

Monitoring activities included collecting distress data and measuring faulting and roughness. Deflections and strains induced by the applied wheel loads were also recorded. Laboratory data collected included layer modulus values, flexural strengths, and bond strengths.

Preliminary Results/Findings

A summary of the performance of the test sections is provided in Table 59. Although a few of the sections exhibited significant cracking, many experienced very little cracking. The primary types of distress documented were corner, transverse, and longitudinal cracks with corner cracking being the most predominant distress. A small amount of faulting along some of the longitudinal and transverse joints was measured, but overall very little faulting developed. Test sections that contained an HMA with binders having a higher penetration tended to exhibit a slightly higher level of cracking. Reducing the panel size when overlaying an HMA with a soft binder tended to increase the performance of the overlay.

Design recommendations based on the findings from this study are being prepared, and the database of information produced will be available to the public in conjunction with a report on the contents of the database.

Current Project Status, Results, and Findings

The cracked slabs in Lane 6 and Lane 10 were repaired on April 28, 2000. Mostly panels with multiple cracks or loose or missing pieces of concrete were replaced. Five panels were replaced in Lane 6 and three panels in Lane 10. The steps followed in performing the repairs appear below.

Table 59. Performance of UTW Test Sections (CTL 2001; FHWA 2004)

LANE NO.	ACTUAL LOAD APPLICATIONS	NUMBER OF PANELS LOADED	NUMBER OF PANELS EXHIBITING CRACKING	RANGE OF LONGITUDINAL FAULTING (IN.)	RANGE OF TRANSVERSE FAULTING (IN.)
5	194,500 @ 12 kips	12	18	0.1-0.7	None
6	359,000 @ 12 kips	12	11	0.1-0.3	None
7	283,500 @ 12 kips	16	4	0-0.1	None
8	628,000 @ 12 kips	16	3	0.1-0.2	0-0.1
9	266,000 @ 12 kips	8	10	0.1-0.0.3	0-0.2
10	441,000 @ 12 kips	8	4	0.1-0.2	0-0.2
11	310,000 @ 10 kips 762,630 @ 12 kips	12	6	0.1-0.6	None
12	310,000 @ 10 kips 762,630 @ 12 kips	12	4	0-0.0.3	0-0.4

1. Identify panels to be repaired.
2. Mark boundaries for saw cuts, which should be at located at least 101.6 mm (4 in.) on the inside of each joint.
3. Perform saw cuts.
4. Used jackhammer to break-up slabs and to dislodge the bonded portions of the concrete overlay from the HMA.
5. Remove debris and clean surface in the repair area.
6. Place new concrete.
7. Saw joints.

The average depth of the repair area after the panels were removed for Lane 6 was 96.5 mm (3.8 in.) and 119.4 mm (4.7 in.) for Lane 10. This resulted in an increase in overlay thickness of 33 mm (1.3 in.) for the repairs in Lane 6 and 33 mm (1.3 in.) for the repairs in Lane 10. More information on the repairs can be found in the report prepared by CTL (2001).

The slump, air content, concrete temperature, and unit weight of the repair concrete were measured. The compressive strength, split tensile strength, modulus of rupture, and modulus of elasticity of the concrete were also measured. Information on the test results is available in the report prepared by CTL (2001).

Four panels were also removed from Lane 9 to investigate the removal of slabs using a small front-end loader. Additional loadings were not applied to Lane 9.

Additional 12-kip dual wheel loads were applied to each lane beginning 8 days after the repair and continuing until early August 2000. During this time, Lane 6 accumulated 400,000 load repetitions and Lane 10 427,000 repetitions. Two of the five panels repaired in Lane 6 exhibited cracking, and two of the three panels repaired in Lane 10 cracked. These cracks were primarily corner cracks.

Point of Contact

James Sherwood
Office of Infrastructure
Research and Development
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, VA 22101
james.sherwood@fhwa.dot.gov
(202) 493-3150

References

Construction Technologies Laboratories, Inc. (CTL). 2001. *UTW Pavement Repair Demonstration*. Final Report. Innovative Research Foundation, Falls Church, VA.

Federal Highway Administration (FHWA). 2004. *Ultra-Thin Whitetopping (UTW) Project.* http://www.tfhrc.gov/pavement/ utwweb/utw.htm.

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Chapter 40. Various States, MIT Scan-2

Introduction

Dowel bar placement is critical for providing proper load transfer for concrete pavements. Misaligned dowels can lock up the joint and cause premature failure. However, the ability to identify the accuracy of dowel bar placement is difficult. A new device, MIT Scan-2, developed by MIT GmbH, Dresden, Germany, was created for the specific purpose of locating steel dowel and tie bars in concrete pavements. Figure 110 provides a close-up view of the device (ERES 2003). In 2003, the FHWA conducted laboratory and field evaluation of the MIT Scan-2 technology to assess its performance in determining dowel bar location. It is anticipated that MIT Scan-2 will become an important monitoring tool in QA/QC procedures.

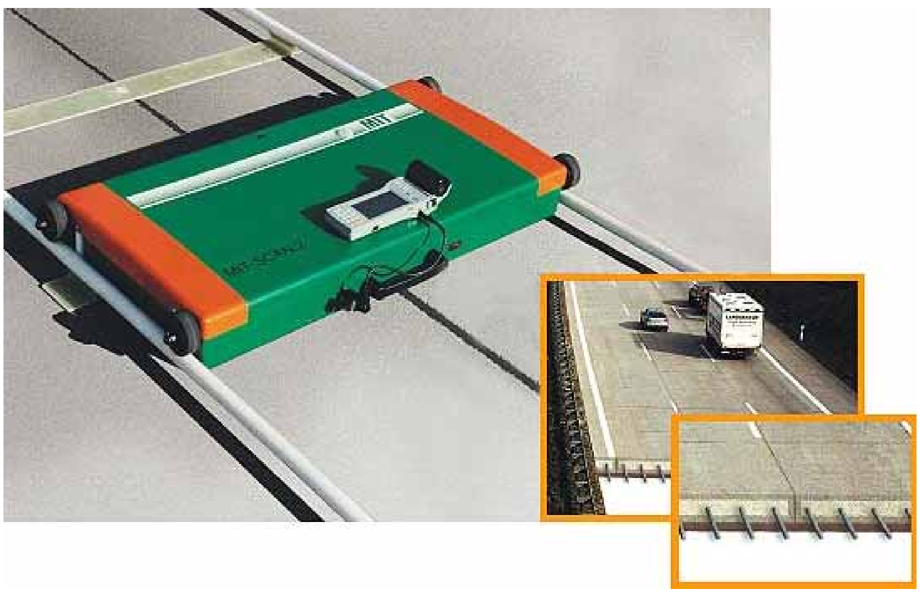

Figure 110. Close-up view of MIT Scan-2.

Study Objectives

The objectives of this project are the following (ERES 2003):

- Provide laboratory and field evaluation of MIT Scan-2 to assess accuracy and repeatability of measurements of dowel position.
- Compare MIT Scan-2 measurements with measurements of other devices such as cover meter and ground penetrating radar (GPR).
- Demonstrate MIT Scan-2 to contractors and State DOT personnel and collect their comments on usability of this device.
- Develop recommendations for use of MIT Scan-2 in QA/QC procedures by contractors and State DOTs and develop comprehensive training material.

Project Design and Layout

The laboratory testing program was designed to verify the measurement accuracy, determine the overall standard deviation of measurement error, and identify factors affecting measurement results for MIT Scan-2. The following laboratory tests were identified (ERES 2003):

- Determine repeatability of measurements.
- Determine the absolute measurement error.
- Document that the measurements are not affected by cover material or water.
- Determine the effects of common factors (variability in bar length, presence of metal objects, and water) encountered in the field.
- Determine the effects of dowel baskets—properly cut baskets only.

Other than the basic objectives, no specific plans were developed for the field testing, since the feasibility of field testing depends on the availability of projects (ERES 2003).

Preliminary Results/Findings

Literature Review Summary

A comprehensive literature search was conducted to evaluate various types of cover meters, GPR, and MIT Scan-2 in determining dowel bar alignment. The cover meters and MIT Scan-2 were noted to have the following advantages over GPR (ERES 2003):

- The weather conditions (dry vs. wet) do not affect measurement results.
- Testing can be conducted on "green" concrete.

The main disadvantage of cover meters and MIT Scan-2 is that the presence of other metal affects the measurement results. In addition to the advantages mentioned above, however, MIT Scan-2 is able to scan the entire joint and determine the location and alignment of all dowels in one measurement (ERES 2003).

Laboratory Testing Results

The first series of laboratory testing was completed during August 2003. Preliminary results from the laboratory testing confirmed the MIT's claim on the accuracy of Scan-2 (ERES 2003):

- Depth: ±2 mm (0.0787 in.)
- Horizontal misalignment: ±2 mm (0.0787 in.)
- Vertical misalignment: ±2 mm (0.0787 in.)
- Side shift: ±5 mm (0.1968 in.)
- Repeatability: ±2 mm (0.0787 in.)

With the rail fixed, the measurements from one series of repeated testing had the following standard deviations (ERES 2003):

- Horizontal misalignment: 0.6 mm (0.0236 in.)
- Vertical misalignment: 0.4 mm (0.0157 in.)
- Side shift: 0.5 mm (0.0197 in.)

These results are for the bars with misalignment less than 13 mm (0.5118 in.). The variability is somewhat higher for bars with greater misalignment.

Field Testing Results

Limited field testing conducted in Reno, Nevada showed that for dowel bars placed in baskets, the presence of the metal basket interferes with the MIT Scan-2 measurement results. However, if the transport ties for the basket are cut, good results can be obtained (ERES 2003). MIT is in the process of developing software that can compensate for the presence of the basket.

Even without the basket software, however, the field testing results showed that MIT Scan-2 can be a useful tool for detecting problems with dowel placement. Figure 111a shows an example of dowel basket that is pulled apart during construction, resulting in severely misaligned bars. Figure 111b is an example of properly placed dowel basket (ERES 2003).

Point of Contact

Sam Tyson
Federal Highway Administration
Office of Infrastructure
Office of Pavement Technology (HIPT)
400 7th Street, SW
Washington, DC 20590
(202) 366-1326

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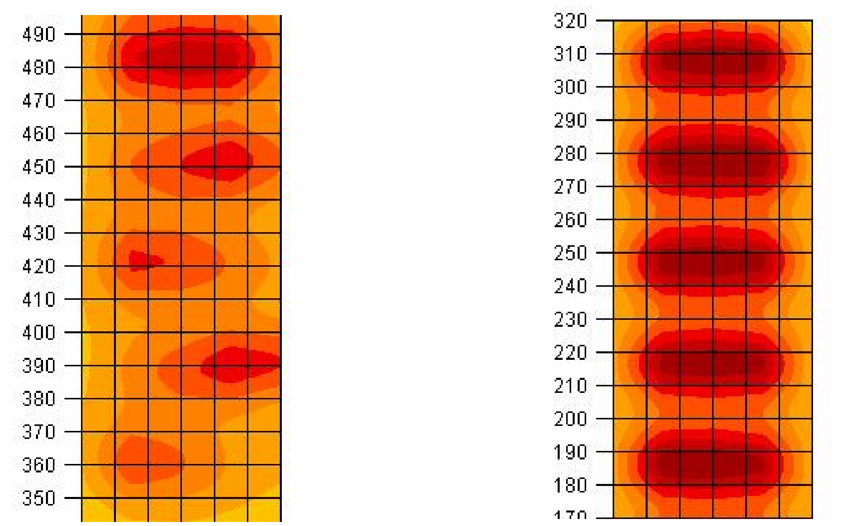
Reference

ERES Consultants, Inc. (ERES). 2003. *Use of Magnetic Tomography Technology to Evaluate Dowel Placement.* Quarterly Progress Report to Federal Highway Administration.



(a) (b)

Figure 111. Example sectional contour map (tomography) of electro-magnetic signal detected by MIT Scan-2: a) severely distorted basket with bars pulled out; b) properly placed dowel basket.

APPENDIX A

TE-30 PROJECT SUMMARY TABLE

APPENDIX A. TE-30 PROJECT SUMMARY TABLE

STATE AND PROJECT DESIGNATION		TE-30?	DESIGN FEATURES EVALUATED	RANGE OF STUDY VARIABLES	LOCATION	NUMBER OF SECTIONS	YEAR BUILT	PROJECT LENGTH	NO. OF LANES (1-DIR)	SLAB THICKNESS	BASE TYPE	JOINT SPACING	LOAD TRANSFER	JOINT SEALANT	SHOULDER TYPE
California	CA 1	No	Precast, Post-Tension Concrete Pavement	Single Application of TX Design Standards	Pending	1				10 in.		8 ft			
Colorado	CO 1	No	Thin Whitetopping	Thickness (3.9 in., 4.5 in., 6.6 in.) Jt Spacing (4 x 4, 6 x 6, 6 x 9 ft)	SH 121 Wadsworth	4	2001	4 mi	2	Varies	HMA	Varies	No		
	CO 2	No	Precast Concrete Repairs	Const ruction Methods (Michigan & Colorado)	I-25, Loveland	1	2004	450 panels	2			12 ft	Yes	Yes	Tied PCC
Illinois	IL 1	No	Alternative Dowel Bar Materials	Fiber composite bars (1.5 in.) Epoxy coated steel bars (1.5 in.)	I-55 SB, Williamsville	4 Joints with Fiber Bars, 3 control	1996	315 ft	1	11.25 in. JRCP	BAM	45 ft (15 ft hinges)	1.5 in. dowels (various types)	Preformed (HP in hinge joints)	Widened Slab + PCC Shldr
	IL 2	Yes	Alternative Dowel Bar Materials Sealed/Nonsealed Joints	Epoxy-coated steel bars (1.5 in.) Polyester and fiberglass bars (1.5 and 1.75 in.) Epoxy-resin and fiberglass bars (1.5 in.) Sealed and nonsealed Hinge Joint JRCP and Conventional JRCP	IL 59 SB, Naperville	8	1997	2355 ft	3	10 in. JRCP 10 in. JPCP	12 in. AGG over porous embankment	45 ft (15 ft hinges) 15 ft	1.5 to 1.75 in. dowels (various types)	Preformed HP None	Tied curb and gutter
	IL 3	Yes	Alternative Dowel Bar Materials Sealed/Nonsealed Joints	Polyester and fiberglass bars (1.5 in.) Vinyl ester and fiberglass bars (1.5 in.) Cement-filled fiberglass and polyester resin tubes Carbon steel rods with stainless steel cladding (1.5 in.) Epoxy-coated steel dowels (1.5 in.) Sealed and nonsealed joints	US 67 WB, Jacksonville	7 (5 for dowels, 2 for sealing)	1999	1065 ft	2	10 in. JPCP	4 in. CAM II 12 in. LSS	15 ft	1.5 in. dowels (various types)	HP No Seal	Tied PCC
	IL 4	No	Alternative Dowel Bar Materials	Cement-filled fiber composite tubes Cement-filled stainless steel tubes (1.5 and 1.75 in.) Carbon steel rods with stainless steel cladding (1.5 and 1.75 in.)	SR 2 NB, Dixon (driving lanes only)	5	2000		2	9.5 in. JPCP	12 in. AGG	15 ft	1.5 to 1.75 in. dowels (various types)	HP	Tied curb and gutter
Indiana	IN 1	Yes	Mixed design	Low shrinkage and Low curling and warping Improve fracture properties	I-65 at SR-60, Clark County	3	2002	1.55 mi	3	14 in..	AGG	18 ft. and 15 ft. (experimental)	Dowels (1 in.)	N/A	concrete
Iowa	IA 1a	Yes	PCC Mixing Times	Iowa DOT and contractor mixes 45-, 60-, and 90-second mixing times	IA 5, Carlisle		1996	3.6 mi		JPCP					
	IA 1b	Yes	PCC Mixing Times	30- and 45-second mixing time	US 30, Carroll		1996	2.2 mi		JPCP					
	IA 2	Yes	Alternative Dowel Bar Materials Alternative Dowel Bar Spacings	Fiber composite (1.5 and 1.88 in. at 8 and 12 in. spacing) Stainless steel (1.5 in. at 8 and 12 in. spacing) Epoxy-coated steel (1.5 in. at 12 in. spacing)	US 65 Bypass, Des Moines	8	1997	2432 ft	2	12 in. JPCP	6 in. AGG	20 ft (skewed)	1.5 to 1.88 in. dowels (various types)	HP	HMA
	IA 3	Yes	Subgrade Stabilization	Fly Ash Stoke Ash	US 151 in Linn/Jones Counties	4 (test sections plus control)	2000–2001	7,100 m	2 lanes–dir.	240 mm thick pavement	Fly-ash stabilized base	6m joint spacing	Doweled joints	HP bitumi-nous	Granular
	IA 4	No	Alternative Dowel Bar Shapes	Heavy Elliptical Bars (2.084 sq in.) Medium Elliptical Bars (1.473 sq in.) Standard Round Bars (1.5 in. dia)	IA 330	21	2002	10,500 ft	2			20 ft	Dowels (various types)		
	IA 5	No	Elliptical Fiber Reinforced Polymer Dowel Bars	Dowel Bar Spacing (10, 12, 15 in.)	IA 330 Melbourne	3	2002	180 m	2			6 m	FRP Elliptical Dowels		

STATE AND PROJECT DESIGNATION		TE-30?	DESIGN FEATURES EVALUATED	RANGE OF STUDY VARIABLES	LOCATION	NUMBER OF SECTIONS	YEAR BUILT	PROJECT LENGTH	NO. OF LANES (1-DIR)	SLAB THICKNESS	BASE TYPE	JOINT SPACING	LOAD TRANSFER	JOINT SEALANT	SHOULDER TYPE
	IA 6	No	Field Trials of Guidelines for Soil Stabilizers of Non-Uniform Subgrade Soils	Raw, Hydrated & Conditioned Fly Ash	Various (pending)	Various (pending)									
	IA 7	No	Implementation of Total Environ-mental Management for Paving (TEMP)	Up to 3 Implementation Sites	Pending	3	Pending								
Kansas	KS 1	Yes	Alternative Dowel Bar Materials Fiber PCC Alternative PCC Mix Designs Joint Sawing Alternatives Joint Sealing Alternatives Surface Texturing Two-Lift Construction	Epoxy dowels, fiber composite bars, and X-Flex Polyolefin fibers Lower w/c mix Early-cut saws (3 types) Preformed, HP, and no seal Longitudinal and random tining Two-lift construction (3 types) High solids curing compound	Highway K-96 EB, Haven	13	1997	6.1 mi	2	10 in. JPCP 10 in. FRCP	CTB	15 ft	1.25 in. to 2 in. dowels (various types) "X-Flex" device	Preformed HP None	HMA
	KS 2	Yes	Smoothness Monitoring of Plastic Concrete	Smoothness monitoring of plastic concrete	Hutchinson, KS	5	2001	12,401 m	1	220 mm	Dense cement treated base	5 m	1 ¼ in. dowels at 12 in. centers	Transverse-Preformed neoprene compression; Longitudinal-low modulus hot pour	Tied concrete
Maryland	MD 1	Yes	PCC Mix Design Fiber PCC	Conventional PCC Polypropylene fiber reinforced PCC Low shrinkage PCC (large coarse aggregate size)	US 50, Salisbury Bypass	3	2001			JPCP FRCP					
Michigan	MI 1	No	Two-Lift Construction Exposed Aggregate Surface Thick Foundation	JRCP and JPCP OGDL and LCB Subbase thickness Surface texturing	I-75 NB (Chrysler Freeway), Detroit	2	1993	2.1 mi	2 to 3	11 in. JRCP 10 in. JPCP	4 in. OGDL 6 in. LCB	41 ft 15 ft	1.25 in. dowels (variably spaced on JPCP)	HP Preformed	PCC Shldr Widened Slab + PCC Shldr
Minnesota	MN 1	Yes	Alternative Dowel Bar Materials PCC Mix Design	High performance PCC mix (high air content, low w/c, high quality aggregate, GGBFS) Stainless steel dowels (1.5 and 1.7 in.) Epoxy-coated steel bars (1.5 in.)	I-35W, Richfield	5	2000	1 mi	3	13.4 in. JPCP	5 in. AGG	15 ft	1.5 to 1.7 in. dowels (various types)	HP	PCC
	MN 2	Yes	Alternative Dowel Bar Materials Doweled/Nondoweled Joints PCC Mix Design	5 and 7.5 in. JPCP High performance PCC mix with GGBFS Doweled/nondoweled joints Epoxy-coated dowels (1 and 1.25 in. diameter) Fiber composite dowels (1.25 and 1.50 in. diameter)	Mn/Road Low-Volume Road Facility, Albertville	3	2000	870 ft	2	5 in. JPCP 7.5 in. JPCP	7 in. AGG 5 in. AGG	10 ft 15 ft	None 1 to 1.5 in. dowels (various types)	HP Silicone None	Widened Slab + Granular
	MN 3	No	Ultrathin Whitetopping	Instrumented sections with varying thickness and joint spacings	Mn/ROAD, Mainline Road Facility and US 169, Albertville	9	1997	788 ft per section		3 in. 4 in. 6 in.					
Mississippi	MS 1	Yes	Alternative PCC Paving Material (Resin Modified Pavement)	Resin modified pavement Ultrathin whitetopping Polymer-modified HMA overlay	US 72, Corinth	4 (2 RMP, 1 UTW, 1 HMA)	2001	Approx. 900 ft	2	2 in. RMP 3 in. UTW 4 in. HMA	Existing HMA pavement	RMP: N/A UTW: 3 ft HMA: N/A	RMP: N/A UTW: None HMA: N/A	RMP: N/A UTW: None HMA: N/A	
Missouri	MO 1	Yes	Fiber PCC Slab Thickness Joint Spacing	Conventional JPCP (9 and 11 in.) Steel fibers (5, 6, and 9 in.) Polyolefin fibers (5, 6, and 9 in.) 15, 30, 60, and 200 ft joint spacings	I-29 SB, Rock Port	8	1998	3.9 mi	2	9 and 11 in. JPCP 5, 6, 9 in. FRCP	Underlying 9 in. JRCP with 61.5 ft joint spacing	15 ft 30 ft 60 ft 200 ft	1.25 and 1.5 in. epoxy coated steel dowels	HP	AC
New Hampshire	NH 1	Yes	Definition of HPCP Design Optimization	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

STATE AND PROJECT DESIGNATION		TE-30?	DESIGN FEATURES EVALUATED	RANGE OF STUDY VARIABLES	LOCATION	NUMBER OF SECTIONS	YEAR BUILT	PROJECT LENGTH	NO. OF LANES (1-DIR)	SLAB THICKNESS	BASE TYPE	JOINT SPACING	LOAD TRANSFER	JOINT SEALANT	SHOULDER TYPE
Ohio	OH 1	Yes	PCC Mix Design	PCC mix with GGBFS / Conventional PCC mix	US 50, Athens	2	1997	10.5 mi	2	10 in. JRCP	4 in. Perm AGG 6 in. AGG subbase	21 ft	1.5 in. epoxy coated steel bars	HP	PCC
	OH 2	Yes	Alternative Dowel Bar Materials	Fiberglass bars (1.5 in.) Epoxy coated steel bars (1.5 in.) Stainless steel tubes filled with concrete	US 50, Athens	3	1997	378 ft	2	10 in. JRCP	4 in. Perm AGG 6 in. AGG subbase	21 ft	1.5 in. dowels (various types)	HP	PCC
	OH 3	Yes	Sealed/Nonsealed Joints	Hot Poured Sealants Silicone Sealants Neoprene Sealants No Seal	US 50, Athens	14 (28 including replicates)	1997- 1998	2.0 mi	2	10 in. JRCP	4 in. Perm AGG 6 in. AGG subbase	21 ft	1.5 in. epoxy coated steel bars	HP Silicone Preformed None	PCC
	OH 4	No	Evaluation of nondestructive test devices for measuring the support of subgrade and aggregate base layers	Nuclear Gauge Humboldt Gauge FWD, Large Load FWD, Small Load German Plate	US 35, James- town	1	2000	5.26 mi	2	9 in. JRCP	4 in. drainable base, 6" dense graded aggregate				
Pennsylvania	PA 1	Yes	Evaluation of HIPERPAV	Tie Bars (Yes/No) Dowel Bars (Yes/No) Static Sensors (Yes/No) Dynamic Sensors (Yes/Not)	SR 22, Murrys- ville	4	Fall 2004	210 ft	2	12 in.	Asphalt Treated	15 ft	Dowels (1.25 in., when used)		
South Dakota	SD 1	Yes	Fiber PCC Doweled/Nondoweled Joints Joint Spacing	Polyolefin fiber reinforced concrete (doweled and non-doweled, 6.5 and 8 in. thick, 0, 25, and 35 ft joints) Conventional concrete (8 in. thick, 20 ft doweled joints)	US 83, Pierre	8	1996	4765 ft	1	6.5 and 8 in. FRCP 8 in. JPCP	Gravel	20 ft 25 ft 35 ft None	Dowels (1 in. epoxy coated steel bars) No Dowels	HP	Widened Slab + Granular
Virginia	VA 1	Yes	PCC Mix Design	2 in. top size aggregate with GGBFS 1 in. top size aggregate with GGBFS 1 in. top size aggregate with Class F fly ash	I-64, Newport News	3	1998- 1999	2.38 mi	3	11 in. JPCP	Asphalt-treated OGDL	15 ft	1.25 in. epoxy coated steel bars	Silicone	Widened Slab + HMA
	VA 2	Yes	PCC Mix Design Steel Contents	Low shrinkage and high flexural strength mixes Various longitudinal steel contents	VA 288, Richmond		2000			CRCP		N/A	N/A	N/A	
	VA 3	Yes	PCC Mix Design Steel Contents	Low shrinkage and high flexural strength mixes Various longitudinal steel contents	US 29, Madison Heights		2000			CRCP		N/A	N/A	N/A	
Washington	WA 1	No	PCC Mix Design and Construction at Intersections	Evaluate rapid reconstruction of urban intersections by looking at methods to accelerate the rate of concrete strength gain, methods to minimize the construction time, and traffic control strategies to minimize user delay	SR 395, Kenne- wick	1	2000	Intersec- tion							
West Virginia	WV 1	No	GFRP Dowel Bars GFRP Reinforced CRCP	Evaluate the potential use of GFRP dowel bars in. JPCP and GFRP reinforcing for CRCP	Pending										
Wisconsin	WI 1	Yes	Surface Texturing	Random, transverse tining (2 levels) Uniform, transverse tining Random, skewed tining (4 levels) Random, longitudinal tining (2 levels) Uniform, longitudinal tining	WI 29, Abbotsford	10 (+47 from other states)	1997	10 mi	2	JPCP		17-20-18-19 ft variable	Dowels	None	Widened Slab + HMA
	WI 2	Yes	Alternative Dowel Bar Materials Alternative Dowel Bar Spacings	Fiber reinforced polymer composite dowels Solid stainless steel dowels Hollow core/mortar filled stainless steel dowels Epoxy-coated dowels Dowel placement (5 layouts)	WI 29 EB, Owen	13	1997	10.6 mi	2	11 in. JPCP	2 in. AGG 5 in. AGG (in place) 9 in. AGG (in place)	17-20-18-19 ft variable	1.5 in. Dowels (various types)	None	Widened Slab + HMA

STATE AND PROJECT DESIGNATION		TE-30?	DESIGN FEATURES EVALUATED	RANGE OF STUDY VARIABLES	LOCATION	NUMBER OF SECTIONS	YEAR BUILT	PROJECT LENGTH	NO. OF LANES (1-DIR)	SLAB THICKNESS	BASE TYPE	JOINT SPACING	LOAD TRANSFER	JOINT SEALANT	SHOULDER TYPE
	WI 3	Yes	Alternative Dowel Bar Materials Alternative Dowel Bar Spacings Trapezoidal Cross Section	Conventional thickness (12 in.) Variable thickness (8-11 in.) Fiber composite dowels, stainless steel dowels, epoxy-coated steel dowels Dowel placement (2 layouts)	WI 29, Hatley	6	1997		2	11 in. JPCP 8-11 in. JPCP	6 in. AGG	18 ft	1.5 in. Dowels (various types)	None	Widened Slab + HMA
	WI 4	Yes	Alternative Dowel Bar Materials PCC Mix Design	High performance PCC mix (Class C fly ash 0.4 water to cement ratio) Stainless steel dowels (1.5) 24-36 in.. dense graded base 12-24 in.. select granular back fill 4 in.ches of open graded base	I-90, approx. 8 mi W of split of I-90 and I-94, Monroe County	2 test sections and 1 control section	2002	2500 ft.	2	13.5 in.. research slab / 12 in.. control slab	Std. 4" OGDL / 6" dense graded – Mod. 4" open graded drained / 10" dense graded	15 ft. joint spacing	18 in.. long, 1.5 in.. dia. Stainless steel	None	concrete
Federal Highway Administration	FHWA 1	No	Ultrathin Whitetopping	Validate the design equations and performance prediction models used in the ACPA UTW design procedure	FHWA's Pavement Test Facility, McLean, VA	8	1998–1999	400 ft	1	2.5 in. UTW 3.5 in. UTW (fibers varied)	HMA	3 x 3 ft 4 x 4 ft 6 x 6 ft			
Various States		No	MIT Scan-2	Conducted laboratory and field evaluation of MIT Scan-2	Various	Ongoing	Ongoing								

KEY:

AGG = aggregate
BAM = bituminous aggregate mixture
CAM = cement aggregate mixture
CRCP = continuously reinforced concrete pavement
FRCP = fiber-reinforced concrete pavement
GFRP = glass fiber reinforced polymer
GGBFS = ground granulated blast furnace slag
HMA = hot mix asphalt
HP = hot-poured sealant
JPCP = jointed plain concrete pavement
JRCP = jointed reinforced concrete pavement
LCB = lean concrete base
LSS = lime stabilized subgrade
N/A = not applicable
OGDL = open-graded drainage layer
PCC = portland cement concrete
RMP = resin-modified pavement
UTW = ultrathin whitetopping

APPENDIX B

BIBLIOGRAPHIC LISTING OF PROJECT REFERENCES BY STATE

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