REPORT S2-R21-RR-3

Composite Pavement Systems

Volume 2

PCC/PCC Composite Pavements

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STRATEGIC HIGHWAY RESEARCH PROGRAM
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STRATEGIC HIGHWAY RESEARCH PROGRAM

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Composite Pavement Systems

Volume 2: PCC/PCC Composite Pavements

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FOREWORD

James W. Bryant, Jr., PhD, PE, *SHRP 2 Senior Program Officer, Renewal*

Volumes 1 and 2 of the R21 project present the state of the practice and guidelines for designing and constructing new composite pavements. Volume 1 provides the tools needed to design and construct new hot-mix asphalt (HMA) concrete over a portland cement concrete (PCC) composite pavement that takes full advantage of using differing materials. Volume 2 provides guidance on the design and construction of two-layer, wet-on-wet PCC pavements where the upper layer is a thin high-quality layer (hard nonpolishing aggregate, higher cement content, higher quality binder) and excellent surface characteristics with the lower layer containing a higher percentage of local aggregates and recycled materials. Both volumes detail performance data on existing composite pavement systems and provide step-by-step guidance on the design of composite pavements using mechanistic-empirical design methods for both types of new composite pavements.

Composite pavements have proved in Europe and the United States to have long service life with excellent surface characteristics, structural capacity, and rapid renewal when needed. Based on statistics compiled in 2000, approximately 30% of the urban interstate system and just over 20% of the rural interstate system is classified as "composite" pavement. In most cases the composite pavements are the result of maintenance and rehabilitation activities and not intentionally designed new composite pavement systems.

This project developed the guidance needed to design and construct new composite pavement systems. The research determined the behavior, properties, and performance for both HMA/PCC and the PCC/PCC composite pavements under many climate and traffic conditions. Experimental composite pavements were constructed at MnROAD in Minnesota and the University of California Pavement Research Center at Davis, where the pavements were instrumented and monitored under climate and heavy traffic loadings. A composite pavement consisting of HMA over jointed plain concrete also was constructed in the field by the Illinois Tollway north of Chicago. At the Tollway, extensive field surveys were performed on 64 sections of the two types of composite pavements.

This project also evaluated, improved, and further validated applicable structural, climatic, material, and performance prediction models, and design algorithms that are included in the AASHTO *MEPDG* and DARWin-ME, CalME, NCHRP 1-41 reflection cracking, NCHRP 9-30A rutting, and the Lattice bonding model. The current DARWin-ME overlay design procedure for HMA/PCC and a special R21 version of the *Mechanistic-Empirical Pavement Design Guide* (*MEPDG* [v. 1.3000:R21]) can be used for new PCC/PCC composite pavements.

The key to the sustainable features of new composite pavements is the ability to use higher levels of recycled materials in the lower concrete layer. Additionally, the thickness of the lower concrete layer can be reduced when considering the insulating effect of the top pavement surface. Intentionally designed and constructed composite pavements will help highway agencies meet the goal of building economical, sustainable pavement structures that use higher levels of recycled materials and locally available materials.

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Executive Summary

Types of Composite Pavement Systems

Two composite pavement design strategies were determined to provide both excellent surface characteristics (low noise, very smooth, nonpolishing aggregates, and durability) that can be rapidly renewed, and long-lasting structural capacity for any level of truck traffic. These two composite pavement design strategies were determined to reflect the Strategic Highway Research Program 2 (SHRP 2) Renewal philosophy of “get in, get out, stay out.”

- High-quality, relatively thin, hot mixed asphalt (HMA) surfacing—such as dense HMA, stone matrix asphalt (SMA), porous HMA, asphalt rubber friction course (ARFC), or Novachip gap-graded asphalt rubber hot mix—over a new portland cement concrete (PCC) structural layer—such as jointed plain concrete (JPC), continuously reinforced concrete (CRC), joined roller compacted concrete (RCC), or a lean concrete base/cement-treated base (LCB/CTB).
- High-quality, relatively thin PCC surfacing atop a thicker, structural PCC layer.

Both types of composite pavements have strong technical, economical, and sustainable merit in fulfilling the key goals of the SHRP 2 program, including long lived pavements, rapid renewal, and sustainable pavements. A survey of U.S. and international highway agencies conducted under the SHRP 2 R21 project revealed considerable interest in both HMA/PCC and PCC/PCC composite pavements.

Research Objectives

The objectives of this research were to investigate the design and construction of new composite pavement systems. The previous technology for the design and construction of new composite pavements was limited. The structural and functional performances of these composite pavements were not well understood or documented. There were no existing mechanistic-empirical (M-E) performance models of these pavement systems, and they need to be developed or improved for use in design, pavement management, and life-cycle cost analysis (LCCA). In addition, the current construction techniques, guidelines, and specifications were insufficient to construct composite pavements properly.

These types of composite pavements give significant flexibility to the designer to optimize the pavement design in terms of life-cycle costs, reduction in future lane closures, and improved sustainability. They essentially exhibit the advantages of conventional HMA and PCC pavements while minimizing their disadvantages. The research under this study, which was conducted from 2007 to 2011, accomplished the following key goals.

- Objective 1. Determine the behavior, material properties, design factors, and performance parameters for each type of composite pavement.
- Objective 2. Develop and validate M-E–based performance prediction models and design procedures that are consistent with the *Mechanistic-Empirical Pavement Design Guide* (*MEPDG*).
- Objective 3. Develop recommendations for construction specifications, techniques, and quality management procedures for adoption by the transportation community.

Constructed and Field Survey Sections

Experimental composite pavements were constructed at two major research sites (MnROAD, Minnesota and the University of California Pavement Research Center [UCPRC] at Davis) and were instrumented and monitored under actual climate conditions and heavy traffic loadings. An HMA/JPC composite pavement also was constructed by the Illinois Tollway north of Chicago. Extensive field surveys were performed in the United States, Canada, and Europe of 64 sections of the two types of composite pavements and used in the analysis and validation.

MnROAD/Minnesota Department of Transportation

One of the major research sites was set up by MnROAD in Minnesota.

- **Design and materials:** Three sections were constructed. The top layer PCC mix contained increased cement content and a high-quality, very durable aggregate (granite). The aggregate in the top lift was gap-graded and had a maximum size of 0.5 in. (12.7 mm). All basic components of the lower-layer PCC were selected to reduce costs, investigate methods of sustainability, and investigate the reuse of materials into structural components. Higher traffic in the outside lane and lower traffic in the inside lane provided two levels of traffic. JPC was the basic type of pavement with transverse joints at 15 ft and dowels at all PCC/JPC joints and in the travel lane only for HMA/JPC joints.
  - **Cell 70:** This section was 3 in. of HMA over 6 in. of JPC (50% recycled concrete aggregate [RCA]; 40% fly ash replacement) over an unbound aggregate base course. The inner lane transverse joints included no dowels, but the outer lane included dowels. Transverse joints across both lanes were sawed and sealed for reflection crack control.
  - **Cell 71:** This section consisted of a 3-in. high-quality PCC layer over a 6-in. low-cost PCC layer (50% RCA; 40% fly ash replacement).
  - **Cell 72:** This section consisted of a 3-in. high-quality PCC layer over a 6-in. PCC layer with 60% fly ash replacement and inexpensive coarse aggregates.
  - **Texturing of Cells 71 and 72:** (1) Exposed aggregate concrete (EAC) achieved by brushing the surface, (2) conventional diamond grinding, and (3) ultradiamond grinding.
- **Specification development:** Full specifications for bidding were developed for each type of composite pavement.
- **Instrumentation and data acquisition:** Instrumentation installed in the pavements included thermocouples for measuring temperature throughout the pavement structure and humidity sensors to measure concrete moisture (relative humidity) levels within the slab. Static strain for static loads generated was measured with vibrating wire (VW) strain gauges to provide several critical pieces of information related to the performance of the pavement layers, responses to temperature and moisture changes, slab curvature, and in-place drying shrinkage. Dynamic strain sensors to measure the slab response to loads applied by truck traffic and the falling weight deflectometer (FWD) were also installed. All data were stored at the MnROAD facility.

- **Construction:** An initial 200-ft test section for PCC/PCC was built and the EAC surfaced prepared. The lessons learned were invaluable for building the main line, which was constructed in May 2010. Construction went well with no serious problems.
- **Loading and monitoring:** Pavements were opened to I-94 traffic in July 2010 and have been loaded ever since except for short closures for monitoring. A full year of heavy traffic has been achieved and the findings included in this report.

University of California at Davis Pavement Research Center

The other major research site was set up by the Pavement Research Center at the University of California at Davis.

- **Design:** The composite HMA/JPC pavement has four 12-ft-wide lanes to accommodate two HMA mixtures, with two HMA thicknesses, two PCC thicknesses, and PCC with and without dowels for load transfer. Each lane has three sections, each consisting of three slabs of 15-ft length. Each pass of the Heavy Vehicle Simulator (HVS) covered two transverse joints and one 15-ft slab in each section.
- **Specification development:** California State specifications were used for construction with some additional requirements.
- **Instrumentation:** Joint deflection measurement devices were installed to measure absolute vertical movement of PCC slab joints, from which the relative movement of the two slabs on each side of the joint can also be measured. Horizontal joint deflection measurement devices were used to measure relative horizontal joint movement caused by the opening and closing of PCC slab joints. Thermocouples and moisture sensors were installed to measure PCC and HMA temperature and relative humidity at various depths. Dynamic strain gauges were placed at slab corners and centers and between HMA lifts in the thicker HMA layers to measure strains occurring under the moving HVS wheel. Static strain gauges were installed to measure slowly changing PCC strains at the top and bottom of the slab caused by creep, shrinkage, warping, and curling.
- **Construction:** PCC was placed in August 2009, and the HMA was placed shortly thereafter. The PCC and two types of HMA both met their respective California Department of Transportation (Caltrans) paving specifications. An anionic SS-1h emulsion tack coat was applied. On Lanes A and B, the mix placed was a ¾-in. (19-mm) maximum aggregate size, dense graded mix with polymer modified PG 64-28 binder (PG64-28PM). On Lanes C and D, the mix placed was a ½-in. (12.5-mm) maximum aggregate size mix with gap-graded aggregate and an asphalt rubber binder produced using the "wet process" (RHMA-G).
- **Loading and monitoring:** The HVS was used to load and evaluate the pavement for HMA rutting, joint reflection cracking, and PCC slab fatigue cracking. The slab cracking loadings required 200,000 and 320,000 heavy wheel repetitions to be applied on two 5-in.-thick non-doweled slabs with thin and thick HMA, respectively. Additional cracking tests may be performed after the R21 project using other funding.

Illinois Tollway

There was also a research site set up in Illinois.

- HMA/JPC composite sections were constructed near Gurnee, Illinois, on the ramps from I-94 to Milwaukee Avenue (off-ramp in the eastbound direction and on-ramp in the westbound direction). The ramps were constructed in October and November 2010 to emulate best practices of constructing HMA/JPC composite pavements using recycled aggregate in the PCC slab.
- The project consisted of using stockpiled recycled asphalt pavement (RAP) coarse aggregate in the PCC mix with a warm mix asphalt (WMA) surface layer. The relatively thin (2-in. [50-mm]),

high-quality dense-graded WMA layer was placed and bonded to the newly placed 9-in. (225-mm), low-cost PCC lower lift after the PCC had hardened sufficiently.

- The PCC slab included a partial replacement of cement with fly ash (~20 to 25%). The use of RAP and fly ash offers environmental advantages by diverting the material from the waste stream, reducing the energy investment in processing virgin materials, conserving virgin materials, and minimizing pollution.
- For WMA, the mix is heated to a lower temperature than for conventional HMA (~60°F to 90°F reduction). Lower temperatures mean less fuel consumption, lower stack emissions, and less fume and odor generation at the plant and job site.
- Coarse aggregate fractionated from the RAP made up 30% of the total coarse aggregate in the PCC mix. Aggregate fines less than 4.75 mm (No. 4) used in the PCC mix were specified to come from virgin aggregate sources. RAP was fractionated, cleaned, and washed. As much as 15% of the total recycled coarse aggregate could consist of agglomerated sand/asphalt particles.
- The PCC surface was cured and textured after placement to ensure adequate bond with the HMA layer. A tack coat was sprayed on to ensure bond. The transverse joints were sawed and sealed in the HMA layer over the joints in the JPC.

Field Surveys of In-Place Composite Pavement Sections

Data were gathered from field surveys of in-place composite pavement sections. A variety of HMA/PCC composite pavement structures were identified:

- Thin asphaltic surfaces, including dense HMA, porous HMA, SMA, ARFC, Novachip, and WMA; and
- Concrete lower layers including JPC, CRC, jointed RCC, jointed LCB, and jointed CTB.

A variety of PCC/PCC composite pavement structures were identified:

- High-quality thin concrete surfaces, including EAC, higher strength PCC, and diamond-ground PCC; and
- Concrete lower layers, including JPC (some with recycled concrete, regular concrete, and lower cost concrete) and CRC.

European countries have been constructing HMA/PCC and PCC/PCC composite pavements for several decades and have substantial experience. HMA/PCC composite pavement was evaluated in the Netherlands using porous 2- to 3-in. HMA/CRC on more than a dozen major heavily trafficked projects, all of which exhibit low noise levels, no rutting, and no reflection cracking. Germany has built SMA surfaces on JPC and most recently over CRC. One SMA/JPC section was 15 years old under heavy traffic with sawed and sealed joints that had performed very well. Austria, Germany, and the Netherlands have all constructed many projects with 2- to 3-in. EAC PCC/JPC since the late 1980s. The entire 200 miles of the A1 freeway across Austria is of this design, with the lower layer PCC containing recycled concrete and about 10% RAP. This highway lies in the harsh climate of the Alps with lots of snow and ice. None of these sections exhibited significant problems and have performed very well over 20 years.

In reviewing these case studies and discussing the composite pavements with the host engineers and practitioners, numerous benefits to importing and implementing European techniques were identified. Dutch, German, and Austrian researchers say that composite pavements provide similar structural performance as an equivalently thick single layer at the same price in Europe, yet the road surface has higher quality and longer life and friction and noise reduction because of the high-quality top layer. Furthermore, composite pavements allow for the optimization of costs and materials throughout the pavement cross section:

- High-quality materials can be used in lesser quantities in the upper layer, where they will be of the most benefit to the system; and

- Less expensive materials can be used in greater quantities in the lower layer, where they will contribute structurally without detracting from the quality and performance of the overall pavement.

Studies in Spain provided valuable information on reflection cracking for HMA/RCC and HMA/CTB and the forming of joints in the RCC and CTB. Since 1991, Spain has used the wet-forming process to form joints. Long-term results show the effectiveness of wet-formed joints every 8 to 13 ft in terms of a reduction in joint deflections and high values of joint load transfer efficiency. The studies also showed that short joint spacing led to fewer reflection cracks, tighter cracks, and improved performance.

Composite Pavement Design

The design procedures in DARWin-ME for HMA overlay of jointed plain concrete pavement (JPCP) and continuously reinforced concrete pavement (CRCP) and in the *MEPDG* for bonded PCC overlay of JPCP and CRCP were found to be the most comprehensive and applicable for design of new composite pavements. Through use of appropriate inputs, the overlay procedure could be used for new composite pavement construction. Extensive testing and evaluations were performed, and many bugs related to composite pavements, as well as significant improvements, were identified and fixed in the *MEPDG*. A new version of the *MEPDG* (v. 1.3000:R21) was developed to use the Bonded-PCC-over-JPCP project to simulate newly constructed PCC/PCC and address limitations of the existing structural and environmental models for PCC/PCC.

CalME

The UCPRC has been developing an M-E pavement design method for Caltrans. The associated software is called CalME. CalME rutting and reflection cracking models were evaluated for the SHRP 2 R21 project. The rutting models were calibrated using the results of the HVS and MnROAD test sections, whereas the reflection cracking model was tested using the results of some of the HVS test sections. Although the number of test cells used in the calibration was small, the results show that the CalME models can predict measured performance effectively using average calibration coefficient values. A sensitivity analysis was performed to evaluate the effects of climate, traffic, HMA mix type, aggregate base stiffness, crack spacing, and HMA thickness. The sensitivity analysis showed that HMA mix type is the primary factor that affects both rutting and reflection cracking.

NCHRP Report 669 Reflection Cracking

In National Cooperative Highway Research Program (NCHRP) Report 669, a reflection cracking model was developed specifically to be implemented in the *MEPDG* and DARWin-ME. The procedure was reviewed, tested, and recommended for implementation in DARWin-ME. It appears that this approach and model will reasonably predict transverse joint reflection cracking for HMA/JPC composite pavements. The existing empirical reflection cracking model was intended as a placeholder and does not predict well.

NCHRP 9-30A Permanent Deformation of HMA Surface

The objective of NCHRP Project 9-30A was to recommend revisions to the HMA rut depth transfer function in the *MEPDG* software developed under NCHRP Project 1-37A. The recommended revisions were based on the calibration and validation of multiple rut depth transfer functions with measured material properties and performance data from roadways and other full-scale pavement sections that incorporate modified or other specialty mixtures, as well as unmodified asphalt binders. The NCHRP 9-30A rutting models for HMA/PCC composite pavements were evaluated and recommendations made for additional research. In summary, all three transfer functions did a fair job of predicting the measured rutting values using mixture properties and other pavement layer properties extracted from project files. Thus, the three rut depth transfer functions described and included in NCHRP 9-30A are believed to be reasonable for composite pavements.

Lattice Model for PCC/PCC Bonding

Extensive work was performed to more fully develop and use lattice models for composite slab simulations for debonding of the top PCC layer from the bottom PCC layer. Completed models coupled the lattice models with finite element models to provide a comprehensive model of the PCC/PCC interface bonding. For model simulations of realistic paving conditions in which newly constructed PCC/PCC pavements are placed in a reasonable time frame, debonding of the layers did not occur. Furthermore, additional simulations of layer behavior took into account unrealistic extreme thermal gradients and highly reduced shear strengths at the interface, and these simulations found failure at the interface in only the most extreme of cases, which would not be encountered in the field. This conclusion is supported by observations from the European PCC/PCC experience, as consultants to the R21 project were unable to cite an instance of PCC/PCC debonding. Based on these observations and model simulations, it was the assessment of the research team that debonding is only a concern in PCC overlays of existing PCC pavements, which was out of the scope of the SHRP 2 R21 project.

Recommendations for Composite Pavement Design

Based in part on these models and improvements made to the *MEPDG*/DARWin-ME software, the following can now be used in the design of new composite pavements:

- New HMA/JPC, HMA/RCC or LCB, and HMA/CRC can be designed using the overlay design feature in DARWin-ME.
- PCC/JPC and PCC/CRC can be designed using *MEPDG* (v. 1.3000:R21), which includes modifications to the allowable PCC layer thicknesses, representative PCC layer properties, slab and base interaction properties (full versus zero friction), PCC/PCC subgrade response modeling, and the distribution of the temperature nodes representing a thermal gradient through the composite pavement system.

Research Products

The products from this research can be classified into five broad categories: (1) design, (2) construction and materials, (3) training, (4) informational, and (5) other.

Design Products

MEPDG (v. 1.3000:R21) developed under this study includes modifications to the allowable PCC layer thicknesses, representative PCC layer properties, slab and base interaction properties (full versus zero friction), PCC/PCC subgrade response modeling, and the distribution of temperature nodes through the composite pavement system. Many of these revisions specifically targeted the Enhanced Integrated Climatic Model (EICM) used by the *MEPDG.* This new program will be submitted to the American Association of State Highway and Transportation Officials (AASHTO) for consideration to incorporate the improvements into the DARWin-ME software. In addition, bug fixes and improvements related to both types of composite pavements were made to the *MEPDG* software throughout the R21 contract (e.g., crack opening error in HMA/CRC), and all of these modifications have been already incorporated into the DARWin-ME software.

The structural fatigue damage and cracking models for both types of composite pavement were validated using all available data: MnROAD test sections, UCPRC test sections, and the existing 64 sections located in the United States, Canada, the Netherlands, Germany, and Austria. The existing global calibration factors were determined to be adequate. However, this does not mean that slab thickness will be the same for conventional or two-layer composite pavements.

- Various other structural and performance models for key distresses (rutting, joint faulting, smoothness) in new composite pavements were validated.
- Several detailed *MEPDG* design examples for composite pavements were prepared for guidance purposes. Comparisons of several examples with conventional JPCP or CRCP indicated a 1- to 3-in. reduction in required thickness for composite pavement. This reduction for HMA/JPC or HMA/CRC was attributable to a reduction in temperature gradients.
- Detailed recommended revisions were made to incorporate composite pavements into the *MEPDG*/DARWin-ME *Manual of Practice* (*MOP*).
- LCCA guidelines and examples were prepared. The life-cycle costs for composite pavement can be lower than those for conventional HMA or PCC pavements:
  - Use of the *MEPDG* (v. 1.3000:R21) and DARWin-ME to design HMA/JPC (including jointed RCC or LCB) or HMA/CRC. The HMA surface insulates the PCC slab from both temperature and moisture gradients. This has major implications regarding the reduction of stresses at the top and bottom of the slab and the resulting reduced fatigue damage, especially at the top of the slab. Comparative designs show a significant reduction in composite slab thickness.
  - In urban areas with high congestion and high costs of lane closures, rapid renewal is paramount. HMA/PCC can be designed for the PCC to structurally last to have a long life (if durable materials are used). The thin HMA can be milled and replaced rapidly with minimal disruption to traffic. PCC/PCC has much longer surface life, but when needed, the surface can be diamond ground to rapidly restore smoothness and friction and reduce pavement/tire noise.
  - Where high-quality aggregates for PCC are not available (or expensive because of long haul distances), local PCC aggregates may be susceptible to polishing and other durability-related distresses. In these situations, HMA or PCC surfaces can protect the structural integrity of the PCC and can be milled and diamond ground and rapidly renewed as needed.
  - Many urban areas and some rural areas exist with old PCC pavements that can be removed and processed and recycled directly back into lower layer PCC. This provides excellent improved sustainability opportunities for composite pavements.
  - Where low pavement noise is required, such as in urban areas with large populations in close proximity to the pavements, porous HMA surfacing of PCC provided the lowest level of noise measured. An alternative was discovered at the MnROAD site, where the next generation diamond grinding was performed on the EAC surfacing, and measurements showed the lowest noise concrete surface measured. These surfaces can be renewed rapidly into the future as needed.
  - Arizona has built many miles of major freeways with porous rubberized asphalt surface over new JPC and CRC to minimize noise. Arizona has had success with this type of pavement, but performance data on this type of pavement in other parts of the country are limited. Low noise is a major reason porous HMA/PCC and EAC PCC/PCC composite pavements are constructed in European countries.
  - Where conventional HMA pavements exhibit transverse cracks and deterioration of transverse cracks is a problem, HMA/CRC is a good alternative to eliminate reflection of transverse cracks. No low-temperature transverse cracks were observed in HMA/JPC or HMA/CRC, and no longitudinal wheelpath cracks have been observed in HMA/PCC pavements, either.
  - Composite pavements can be an economical choice when widening existing PCC or HMA/PCC pavement such that the widened section is compatible structurally with the existing pavement. Both the new and the existing lanes typically are covered with one or more lifts of HMA.

Construction and Materials Products

Construction specifications and guidelines were developed as part of construction at MnROAD and UCPRC for use by agencies considering constructing new HMA/PCC and PCC/PCC composite pavements. These include two-lift wet-on-wet construction of PCC/PCC pavements, timing and sequencing of operations, texturing procedures and related guidelines, guidelines for paving the stiffer lower lift PCC and the thin upper lift, saw cutting of joints, and the challenging exposed aggregate brushing technique. The MnROAD construction also involved the use of ultrasonic tomography to assess PCC/PCC layer thicknesses and bond quality at the PCC/PCC and slab/base interfaces. The PCC upper layer was diamond ground using a next-generation grind that produces a smoother and quieter surface.

Material specifications include those for recycled aggregate, cementitious materials such as cement and fly ash, aggregate type and gradation for EAC, and retarding/curing compound. Procedural specifications include those related to wet-on-wet construction, timing of paving operations, texturing, saw cutting, sealing of sawed and sealed joints, tack coat application for HMA/PCC.

Concrete freeze–thaw durability is a major concern for pavements in many parts of the United States and Canada. The upper layer PCC mixture will experience the most freeze–thaw cycles, but the lower layer mixtures will experience freeze–thaw cycles as well. The International Union of Testing and Research Laboratories for Materials and Structures (Paris) (RILEM) CIF concrete freeze–thaw standard was adopted based on European PCC/PCC experience, and the equipment was imported from Germany for use in the SHRP 2 R21 project. The CIF test evaluates the capillary suction, surface scaling resistance, and internal damage of concrete samples exposed to a 3% by volume sodium chloride solution and freeze–thaw cycles, whereas AASHTO T161 evaluates the internal freeze–thaw damage of concrete submerged in water and AASHTO T277 evaluates the freeze–thaw scaling resistance of concrete exposed to a 3% sodium chloride solution. RILEM CIF freeze–thaw testing and evaluations were conducted on all the concrete mixtures used at MnROAD.

All of these concrete mixes adequately resisted surface scaling and internal damage (modulus) caused by frost action. Compared with the decrease in relative modulus of other concrete samples studied with the RILEM CIF procedure, the loss of scaled material and the decrease in relative moduli of all of the samples were relatively small. The lack of scaling and internal damage in both lower PCC mixes after 56 freeze–thaw cycles indicated that these mixtures are suitable for use in long-life concrete pavements, despite containing recycled concrete aggregates or having a 60% cement replacement with fly ash, respectively. It was expected that the upper lift PCC samples would experience minimal scaling and internal damage caused by frost action because of: the high cement content and low water-to-cement ratio of the mix, as well as the use of high-quality granite aggregates.

Training Products

Materials were prepared to promote the use and accelerate the adoption of new composite pavements. The training materials include both design and construction materials. Design examples for both major types of composite pavements are included.

Informational Products

Includes the R21 final reports (Volumes 1 and 2) and detailed appendices and a database of test sections. Readers may also refer to the previously published report on the European Survey of Composite Pavements by Tompkins, Khazanovich, and Darter in 2010. The database contains material properties, performance, traffic, structure, and location, which are all inputs required for use with the *MEPDG*/DARWin-ME.

Other Products

Three test sections (two PCC/PCC and one HMA/PCC) were constructed at MnROAD with various surface textures (exposed aggregate, conventional grind, next-generation grind, HMA) and design features (doweled/nondoweled and with/without sawed and sealed joints for HMA/PCC) with two different PCC mixes in the lower lift. These are the only instrumented in-service composite pavement test sections in existence. The instrumentation includes static and dynamic gauges, moisture gauges, and temperature gauges, all of which are wired into a data acquisition unit for continuously collecting data. These sections were constructed in April through June 2010 and were opened to traffic in July 2010.

Instrumented UCPRC HVS test sections were constructed in May 2010 and loaded with the HVS equipment. The instrumented test cells can be used for future testing. Data were collected from rutting and reflection cracking tests at UCPRC (including laboratory testing). HMA/JPC full-scale fatigue cracking tests using the HVS were conducted to validate the *MEPDG* transverse cracking models, and the results provided validation. Additional testing may continue with other funding sources.

Overall SHRP 2 R21 Products Use

All of these products are available for use by federal, state, local, and other agencies for design, construction, materials, and management of new HMA/PCC and PCC/PCC composite pavements.

Examples of Composite Pavements

In-service composite pavements have been shown in to provide long lives with excellent surface characteristics, long-life structural capacity, and rapid renewal when needed. Composite pavements seem to reflect the current direction of many highway agencies to build more economical yet sustainable pavement structures that use recycled materials and locally available materials. The availability of DARWin-ME and the validation accomplished under R21 have made it possible to design these composite pavements with confidence. Table ES.1 provides examples

Table ES.1. Examples of HMA/PCC Composite Pavements in First Performance Period

Composite Pavement; Age and No. of Trucks	HMA Layer	PCC Layer	Performance and Maintenance	Design, Sustainability, and LCCA
ARFC/JPC I-10, Arizona; 17 years and 20 million trucks	1-in. ARFC	14-in. JPC 15-ft joints Dowels	Excellent performance; transverse joint reflection low severity; smooth; ARFC has lasted 20 years; no PCC cracks or repairs	DARWin-ME requires thinner slab design; low life-cycle cost over many years; no lane closures
SMA/JPC A93, Germany; 13 years and 47 million trucks	1.2-in. SMA with saw and seal joints	10.3-in. JPC 16-ft joints Dowels	Good performance; transverse joint saw and seal; smooth; no PCC cracks; SMA spall repair	DARWin-ME gives same slab design; low life-cycle cost; few lane closures
HMA/CRC I-10, San Antonio, Texas; 25 years and 24 million trucks	4-in. HMA	12-in. CRC HMA base	Excellent performance; no reflection cracks; smooth; no punchouts; no maintenance	DARWin-ME gives thinner slab design; low life-cycle cost over many years; no lane closures
HMA/RCC White Road, Columbus, Ohio; 7 years and 70,000 trucks	3-in. HMA with sealed cracks after cracking	8-in. RCC 45-ft joints No dowels	Excellent performance; reflection cracks sealed just after cracked; smooth; no maintenance	DARWin-ME gives thinner slab design; short joint space; low life-cycle cost; no lane closures
HMA/JPC I-94, Minnesota; 1 year and 600,000 trucks	3-in. HMA with sawed and sealed joints	6-in. JPC 15-ft joints Dowels	Excellent performance; sawed and sealed transverse joints good condition; no PCC cracks, smooth; no maintenance	DARWin-ME gives same design; PCC contains 50% RCA and 60% fly ash

Note: Trucks given for heaviest lane, one direction only.

of HMA/JPC and HMA/CRC composite pavements for a wide range of heavy truck traffic in their first performance period. The following is a brief summary of the field performance of HMA/PCC type of composite pavements:

- Relatively thin asphaltic surfaces that have performed well include a wide variety of types and thicknesses under heavy traffic: 1- to 2-in. SMA directly on PCC or on HMA on PCC, 2- to 4-in. dense graded HMA over PCC, 1-in. porous HMA over dense HMA/PCC, 1-in. ARFC over PCC projects, and 0.625-in. Novachip over HMA/PCC. There are several successful thin asphaltic surface courses that perform very well over 10 to 15 years. They do not rut significantly. Transverse joint reflection cracks occurred on all JPC and RCC pavements, with most of low to medium severity. Projects in Spain showed that shorter joint spacings (e.g., 10 ft) result in much less reflection cracking and severity. Dowel bars greatly reduced severity of joint reflection cracks on comparative sections in Minnesota. Sawed and sealed joint projects were all in excellent condition and are highly recommended for thin asphaltic surfaces over jointed PCC.
- The JPC, RCC, and LCB concrete layers had a wide range of thicknesses from 5 to 14.5-in. with the thicker sections being very overdesigned. The RCC ranged from 6 to 15 in. thick (way overdesigned). The LCB/CTB ranged from 6 to 11-in. None of the JPC, RCC, LCB/CTB, or CRC showed any transverse fatigue cracking, except the 5-in. JPC in Minnesota under heavy traffic.
- The CRC layers show a wide range of thicknesses, from 8 to 13 in., with percent reinforcement ranging from 0.55% to 0.70%. The only section with punchouts was a section in Arizona with low steel of 0.55% and 0.5-in. ARFC under very heavy traffic over 16 years.
- Joint spacing for JPC typically ranged from 15 to 30 ft. Joints usually were cut in RCC at 15- to 45-ft intervals. Based on other experimental sections in Spain, the shorter joint spacings (e.g., 10 ft) were greatly beneficial in reducing the severity and amount of transverse reflection/shrinkage cracking through the HMA. Sawing and sealing of joints was also greatly beneficial in controlling the severity of the cracks in thin asphaltic surfaces.
- Dowels were used on many heavily trafficked JPC sections but many other sections had none. No dowels were used with RCC or LCB/CTB. Reflection cracks dramatically showed the benefits of dowel bars in controlling joint load efficiency and thus a reduction in HMA deterioration over the joints.
- Truck traffic ranged from low to very heavy. Typically the following ranges existed in the heaviest travel lane:
  - Interstates and freeways: 1.4 million trucks/year (range: 0.5 to 3.6);
  - Highways: 0.2 million trucks/year (range: 0.1 to 0.3); and
  - Local streets: 0.05 million trucks/year (range: 0.004 to 0.08).
- Total trucks in the design lane ranged to 47 million and the age ranged to 45 years.
- One section had a total life of 45 years, during which the asphaltic surface was replaced three times but the PCC did not require any repair. This and another similar HMA/JPC are expected to carry traffic continually into the future with no fatigue cracking, thus no slab replacements, and more rapid renewal. In fact, fatigue cracks developed only on the exceptionally thin PCC layers on some experimental sections. None of the typical thickness JPC developed any slab fatigue cracking.

Table ES.2 shows examples of HMA/JPC sections that have been through two and three HMA surface replacement cycles that were done rapidly because none of the underlying JPC slabs were cracked and needed replacement. These and other HMA/PCC composite pavements have performed well over many years with only the rapid replacement of the HMA type surface course required. They have performed as “long-life” pavements.

Table ES.3 provides examples of PCC/JPC composite pavements for freeways with heavy truck traffic. These and other PCC/JPC composite pavements have performed well over many years with only the eventual renewal of the surface course required through diamond grinding.

Table ES.2. Examples of "Long-Life" HMA/PCC Composite Pavements Over Several Performance Periods

Composite Pavement; Age and No. of Trucks	Surface and Rehabilitation	Base Slab Characteristics	Performance and Maintenance	Design, Sustainability, and LCCA
HMA/JPC I-5, Seattle, Washington; 45 years and 35 million trucks	4-in. HMA original; 2-in. at 13 years; 2-in. at 16 years; 2-in. at 11 years; (some milling at times of resurfacing)	6-in. PCC No joints No dowels	Excellent performance; transverse cracks at 70 ft reflected medium severity after 8 years; smooth; replaced HMA at 11- to 16-year intervals; no additional transverse cracks; no PCC repairs	DARWin-ME would design thicker slab, add doweled transverse joints at 10 to 15 ft; saw and seal would extend life; low life-cycle cost over many years; few lane closures for rehabilitation
HMA/JPC I-294, Chicago, Illinois; 19 years and 30 million trucks	1992: 3.5-in. HMA original; 2001: Milled off and added 3-in. HMA; no additional rehabilitation after 10 more years	12.5-in. JPC; 20-ft joint spacing Dowels	Excellent performance; transverse joints reflected medium severity; smooth; replace HMA at 9- to 10-year intervals; no transverse fatigue cracks in JPC; no PCC repairs	DARWin-ME gives thinner slab design; shorter joint spacing; saw and seal joints would extend life; low life-cycle cost over many years

Note: Trucks given for heaviest lane, one direction only.

Table ES.3. Examples of PCC/PCC Composite Pavement Characteristics, Applications, and Performance

Composite Pavement; Age and No. of Trucks	Upper PCC Layer	Lower PCC Layer	Performance and Maintenance	Design, Sustainability, and LCCA
PCC/JPC I-75, Detroit, Michigan; 18 years and 72 million trucks	2.5-in. EAC	7.5-in. JPC 6-in. LCB 15-ft joint space Dowels	Fair performance; no transverse fatigue cracking; no joint faulting; smooth; only distress is joint spalling or debonding	Designed for very heavy traffic; low expected life-cycle cost; few lane closures
PCC/JPC FL-45, Florida; 30 years and 5 million trucks	3-in. PCC	9-in. JPC Lower PCC strength A, B, and C; 15- and 20-ft joint spacing Doweled and nondoweled	Excellent performance; low transverse fatigue cracking; low joint faulting	Pavement somewhat overdesigned; low life-cycle cost; no lane closures over 30 years; savings of cement; good sustainability
PCC/JPC A93, Germany; 13 years and 53 million trucks	2.8-in. EAC	7.5-in. JPC 16.4-ft joint space Dowels Tied PCC shoulders	Excellent performance; no transverse fatigue cracking; no joint faulting; smooth; low noise; pavement should last many more years	Designed for very heavy traffic; low life-cycle cost; no lane closures good sustainability
PCC/JPC A1, Austria; 14 years and 47 million trucks	2-in. EAC	7.9-in. JPC (RCA materials) 18-ft joint space Dowels ATB	Excellent performance; no transverse fatigue cracking; no joint faulting; smooth; low noise pavement should last many more years	Designed for very heavy traffic; low life-cycle cost; no lane closures; good sustainability
PCC/JPC K-96, Kansas; 14 years and 2.1 million trucks	3-in. PCC	7-in. JPC 15-ft joint space Dowels PCC shoulders	Excellent performance (new pavement); no distress; smooth	Pavement overdesigned; low expected life-cycle cost; no lane closures
PCC/JPC N279, the Netherlands; 8 years and 11.9 million trucks	3.5-in. EAC	7-in. JPC 15-ft joint spacing Dowels	Excellent performance; no transverse fatigue cracks; smooth; low noise; no other distress	Well-designed; low expected life-cycle cost; no lane closures
PCC/JPC I-70, Kansas; 4 years and 3 million trucks	1.5-in. PCC 8 different surface textures	11.8-in. PCC 15-ft joint space Dowels PCC shoulders	Excellent performance (new pavement); no distress; smooth; low noise; long life expected	Designed for very heavy traffic; low life-cycle cost expected
PCC/JPC I-94, Minnesota; 1 year and 600,000 trucks	3-in. EAC and diamond grinding	6-in. JPC 15-ft joint spacing Dowels	Excellent performance; no transverse fatigue cracks; smooth; no maintenance	DARWin-ME gave this design for 15-year life, PCC 50% RCA, 60% fly ash, good sustainability

Note: Trucks given for heaviest lane, one direction only.

A brief summary of the field performance of PCC/PCC type of composite pavements is as follows:

- Relatively thin high-quality concrete surfaces include a variety of types and thicknesses:
  - 2- to 3-in. PCC over JPC performed well for more than 18 years under very heavy traffic. No debonding of PCC from lower layer PCC was observed, with the exception of some cracking at the transverse joints of the I-75 Michigan project after 18 years.
  - 3-in. higher strength PCC over JPC performed well for more than 30 years in Florida. No debonding of the PCC has occurred.
- The JPC concrete lower layers had a range of thicknesses from 6 to 9 in. None of the JPC showed any transverse fatigue cracking.
  - Joint spacing for JPC ranged from 15 to 20 ft.
  - Dowels were used on all of these sections because most were heavily trafficked. As a result, joint faulting was not significant.
- Truck traffic ranged from medium to very heavy. Typically the following ranges existed in units of trucks per year in the heaviest travel lane:
  - Interstates and freeways: 3.3 million trucks/year (range: 1.8 to 4); and
  - Highways: 0.3 million trucks/year (range: 0.1 to 0.7).

  Practically none of the PCC/JPC slabs showed any transverse fatigue cracks.
- Total trucks in the design lane ranged to 72 million, and the age ranged to 30 years.

Implementation Road Map

The road to implementation includes continued monitoring of constructed composite test sections at MnROAD. Additional analysis of the instrumentation data and the performance data will be extremely useful for convincing highway agencies of the validity of the concepts, the design procedures, and the construction guidelines and specifications. The MnROAD test sections can be used to hold national and regional open houses or workshops to disseminate information regarding both types of composite pavements.

The products developed as part of the SHRP 2 R21 project will result in improved design and life-cycle cost procedures for composite pavements. The guidelines, techniques, and specifications developed in R21 will greatly advance the state of the practice of constructing composite pavements. Composite pavements are congruent with the SHRP 2 Renewal philosophy because they are designed to be long-lasting pavements that can be renewed rapidly. For highway engineers, designers, and agency decision makers, composite pavements provide a cost-effective alternative to conventional concrete and asphalt pavements over the life cycle of the pavement. Together, the R21 reports, software, and guidelines provide information for these technologies to become widely adopted by the transportation community.

Based on the comprehensive results achieved from this study, the key characteristics of composite pavements were determined to be:

- There are excellent surface characteristics from the thin, high-quality asphaltic or concrete top layers. These include low noise (especially for permeable mixtures), high friction, very good initial smoothness, minimal rutting, and reasonable durability over a 10- to 15-year period.
- There is an ability to rapidly renew a thin surface course as it wears under traffic and weather (removal and replacement of asphaltic materials, diamond grinding, or retexturing of concrete materials).
- There is long life structural design of the lower PCC layer (designed for minimal fatigue damage over a 40-year period or more).
- There is avoidance of certain distress types that occur regularly in conventional pavements but are rare or nonexistent in composite pavements. For example, HMA/JPC or HMA/CRC rarely show top-down HMA or PCC longitudinal cracking in the wheelpaths (thermal gradients are

reduced that lowers top-down fatigue damage in PCC); these composites rarely show any low temperature transverse cracking (they are bonded to the PCC); and they show only minimal amounts of rutting. Transverse reflection from JPC joints can be controlled by the saw and seal procedure. Transverse reflection of CRC cracks rarely occurred in the HMA/CRC included in the database. PCC/JPC composite pavement has shown no longitudinal top-down cracking and only small amounts of fatigue transverse cracking. The durability of this surface has led to very little polishing in the wheelpaths.

- There are improved life-cycle costs attributable to both lower construction costs and lower maintenance and rehabilitation costs over time.
- There are improved sustainability practices through structural and materials design of the lower PCC layer in both types of pavements. Increased use of recycled or alternative materials (RCA, RAP), increased use of more local and less expensive aggregates, and higher substitution rates for cementitious materials (higher contents of fly ash or other supplementary cementitious materials).

CHAPTER 1

Introduction and Background

This R21 project, "Composite Pavement Systems," fits under the Renewal area, the goal of which is to develop a consistent, systematic approach for performing highway renewal that is rapid, causes minimal disruption, and produces long-lived facilities. The Renewal scope applies to all classes of roads.

Two strategies that have shown great promise for providing strong, durable, safe, smooth, and quiet pavements needing minimal maintenance are: (1) surfacing a new portland cement concrete (PCC) layer with a high-quality hot mix asphalt (HMA) layers, and (2) placing a relatively thin, high-quality PCC surface atop a thicker PCC layer. However, the structural and functional performances of these two types of composite pavements were not well understood or documented. Models for predicting the performance of these pavement systems needed to be developed and/or confirmed for use in design, pavement management, and life-cycle cost analysis (LCCA). In addition, guidance on the development of specifications, construction techniques, and quality management procedures was needed for these technologies to become widely adopted.

Research Objectives and Overview

The objectives of this research were to investigate the design and construction of new composite pavement systems, and specifically not those resulting from the rehabilitation of existing pavements. The goal was to

1. Determine the behavior of new composite pavement systems and identify critical material and performance parameters.
2. Develop and validate mechanistic-empirical (M-E) based performance prediction models and design procedures that are consistent with the *Mechanistic-Empirical Pavement Design Guide* (*MEPDG*).
3. Develop recommendations for construction specifications, techniques, and quality management procedures for adoption by the transportation community.

This project consisted of the following three phases:

- Phase 1 consisted of a literature search, survey of various national and international highway agencies, field survey of composite pavements in three European countries, an evaluation of existing design procedures, development of database for full-scale applications, populating the database with information from available projects, and an initial evaluation of existing data. Phase 1 was completed and the Phase 1 interim report prepared and submitted to SHRP 2 in May 2008.
- Phase 2 consisted of further completion of the databases, analyzing the databases, identifying failure mechanisms and other distresses relevant to new composite pavements, performance modeling, and conducting parametric evaluations of the performance models. Phase 2 also included the development of the detailed research plan for Phase 3. Phase 2 was completed and the Phase 2 interim report prepared and submitted to SHRP 2 in May 2009.
- Phase 3 consisted of implementing the research plan developed in Phase 2. Full-scale roadway sections were constructed and tested at MnROAD. Field composite pavement sites with long-term performance were surveyed, and detailed information was collected in the United States, Canada, and three European countries. The results of these investigations were used to refine and validate the performance models and develop the final design guidelines and procedures. Phase 3 also included the development of construction specifications, design guidelines, and a plan for long-term evaluation and validation of the design models, development of training materials, and delivery of the final report for this research.

Overview of Report

The purpose of this report is to present the work performed throughout the course of this project. Included are the executive summary and two volumes. Volume 1 covers HMA/PCC composite pavements, and Volume 2 covers PCC/PCC composite pavements. Each volume includes six chapters, with Chapter 1 being this introduction and background. Chapter 2 includes details of test sections, and Chapter 3 covers the various aspects of the research relevant to analysis and modeling. Design and construction guidelines are included in Chapters 4 and 5, respectively. Chapter 6 includes product summary, conclusions, and recommendations for future research.

Definitions

PCC/PCC composite pavement systems for the purposes of this research are defined as a relatively thin, high-quality concrete surface placed immediately on top of a plastic concrete layer (Figure 1.1). The lower concrete layer may include increased amounts of recycled materials, including RCA, RAP, and others; increased use of local and less expensive aggregates; and higher substitution rates for cementitious materials (fly ash or other supplementary cementitious materials [SCMs]) that may be less suitable for use in a surface layer at the higher substitution amounts. Construction is accelerated by placing the concrete surface layer on top of the lower concrete layer before the latter has set to facilitate a total bond (no slippage) between the two layers of concrete; this construction technique is commonly called "wet-on-wet" paving. The PCC can be constructed as jointed plain concrete (JPC) or continuously reinforced concrete (CRC). Both PCC layers provide structural capacity, but the lower PCC layer is the primary load-carrying layer (because of the greater thickness) and is expected to provide a durable and strong base that is economical to construct and promotes the ideals of sustainability and energy efficiency. The upper PCC layer is expected to provide excellent surface characteristics over a long time period and to be rapidly renewable (through diamond grinding or other texturing methods).

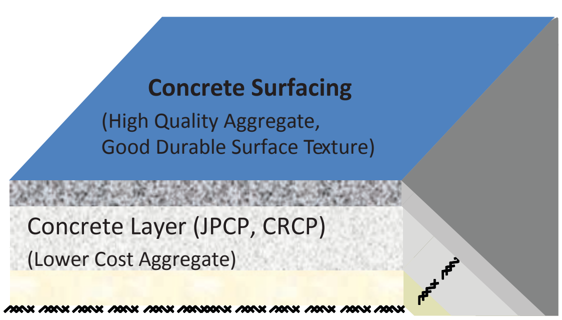


Figure 1.1. Typical cross section for PCC/PCC composite pavements.

History

The history of PCC/PCC in the United States dates to the first concrete pavement constructed in the country, located in Bellefontaine, Ohio, in 1891 (Snell and Snell 2002). This experimental pavement section featured a 2-in. surface and 4-in. structural layer with water-to-cement ratio (w/c) of 0.60 and a durable wearing course with w/c of 0.45. From roughly 1950 to the mid-1970s, two-lift paving for concrete pavements was very common in many U.S. states for the construction of jointed reinforced concrete pavements (JRCP), which began to disappear in the 1970s as agencies began to move toward jointed plain concrete pavement (JPCP) designs. Appendix B provides a review of the history and background of PCC/PCC composite pavements.

In Europe, two-lift paving in the sense of constructing two layers with different properties for the sake of reducing noise, increasing skid resistance, lowering costs, and so forth, has been much more common than in the United States. Austria in particular has been active in regular two-lift paving for concrete pavements, and the standard concrete pavement in Austria is constructed according to two-lift specification (FHWA 1992; Hall et al. 2007). Two-lift paving has been used for special projects in countries such as Switzerland, Belgium, the Netherlands, France, and Germany with regularity since the 1930s and is becoming more common as the techniques are refined. Germany has also used two-lift paving in airport pavements as a way of reclaiming recycled materials (FHWA 1992). Overall, the desire for quieter, more economical and especially more sustainable roadways is motivating many countries to increase the frequency with which concrete pavements are constructed in two unique lifts.

Much like their European counterparts, American pavement engineers have put a great deal of research, design, and construction effort into developing PCC/PCC. There were a limited number of experimental PCC/PCC projects in Iowa, Florida, and North Dakota during the late 1970s and 1980s. The High Performance Concrete Pavement (HPCP) project (FHWA 2006; Larson 2006; and Wojakowski 1998) in particular was responsible for the development of experimental two-lift sections in the 1990s in Michigan and Kansas. These new two-lift experiments had as their larger research goals a desire to increase the service life of concrete pavements, lower life-cycle costs, use innovative designs and materials, and improve construction practices.

Florida Test Sections

Thirty-three composite PCC/PCC test sections on SR-45 near Fort Myers, constructed in 1978, were designed with 3-in. standard PCC in the top lift and 9-in. lean concrete in the lower lift. The sections were designed to observe performance and make comparisons between the different pavement constituents and design properties. The materials under investigation were three types of lean concrete and two subbases. The lean concrete lower lifts differed in terms of the amount of cement (8.5%, 7.3%, and 5.5% by weight), whereas the subbases were either 6-in. cement-treated subgrade (A-3) or 6-in. shell-stabilized subgrade (A-3). The main sections, consisting of PCC surfacing, three levels of PCC lower layers, and two joint spacings, performed very well over a 30-year period (ERES Consultants 1998; Greene et al. 2010). The two-layer composite performed better than the one-layer conventional section. The project conclusions stated that "this experimental project has also demonstrated that a two-layer concrete system consisting of a relatively thin higher quality PCC surface over a lower quality econocrete layer and a granular subbase can be a sustainable and long-lasting pavement design alternative" (Greene et al. 2010).

Kansas Test Sections

The desire to use more innovative materials, such as recycled aggregates, led to the creation of three two-lift test sections on K-96 in Kansas. The K-96 test sections look at three different factors of interest to the Kansas Department of Transportation (DOT):

1. The use of RAP in the lower lift;
2. The use of a durable igneous rock with high alkali silica reactivity as aggregate in the upper lift, instead of an abundant limestone in Kansas that has a tendency to polish and reduce skid resistance; and
3. The use of a lower w/c in the upper lift to investigate if the differential volume changes between the two lifts would lead to debonding.

Researchers at the Kansas DOT found that the replacement of 15% of total aggregates with RAP in a concrete mix did not affect the workability of the mix and resulted in a durable lower lift for a PCC/PCC. In addition, the researchers were able to counteract the alkali-silica reactivity of the hard igneous rock in the second two-lift section by the replacement of cement with a locally available pozzolanic product. The innovative use of the materials was a success, as tests for expansion indicated volume changes far below what would have been expected had alkali-silica reactivity (ASR) occurred.

Finally, the low w/c ratio section showed no shrinkage cracking or evidence of debonding, despite expectations of being difficult. It should be noted that all sections performed well in the long term, although a large number of transverse cracks were observed on the sections with the igneous rock in the upper lift by Kansas DOT in a 2002 annual report. The K-96 project is one of a growing field of research projects that examines PCC/PCC as a potential cost savings and performance increased opportunity through the use of innovative materials (Wojakowski 1998).

Michigan Test Section

One of the first results of the 1992 U.S. TECH scanning tour of European concrete highways (FHWA 1992) was the later development of the 1993 PCC/PCC new construction on I-75 near Detroit, Michigan. The overall goal of the project was to compare the performance of a standard Michigan DOT concrete pavement with its structural PCC/PCC equivalent of European design.

Although the research project had this comparison of design performance as its goal, the project also was a testing ground for two-lift paving techniques that had not been attempted in the United States. The I-75 European PCC/PCC sections were placed without serious problems, but the placement went slowly because of the new techniques required for this composite pavement. In constructing these sections, Michigan DOT and researchers from Michigan State University developed numerous recommendations for future two-lift paving. These recommendations include observations on appropriate sawing depths when forming joints, dowel bar spacing to save costs, minimal thicknesses of surface lifts, and improved techniques for brushing away cement in creating surface texture (Weinfurter et al. 1994; Smiley 1995; Smiley 1996; Buch et al. 2000).

Kansas Test Sections, 2008

The most significant two-lift concrete pavement project constructed in the United States was done in 2008 on I-70 near Abilene, Kansas (Fick 2008). The construction of this several-mile-long project is part of an innovative technology demonstration of two-lift concrete paving. Both conventional and innovative textures are included. The conventional textures include longitudinal tining, burlap drag, longitudinal grooving, and diamond grinding surfaces. The innovative textures include the "next-generation" diamond grinding, along with EAC texture.

Agency Survey

The research team conducted a survey of U.S. and international highway agencies to assess the state of practice and knowledge regarding composite pavement systems. The goals of this survey included the following:

- Assessment of the interest of various highway agencies in designing/building composite pavements within their jurisdiction.
- Identification of agency contacts and projects that can be used in the R21 database for development of the performance models.

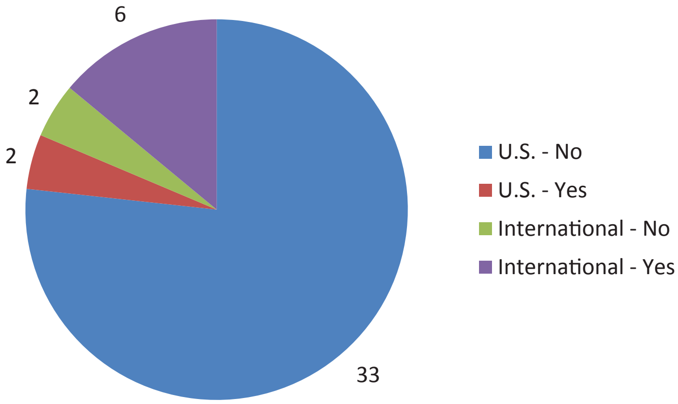


Figure 1.2. Pie chart depicting agency response to the question "Has your agency constructed new PCC/PCC composite pavements in the past 20 years?"

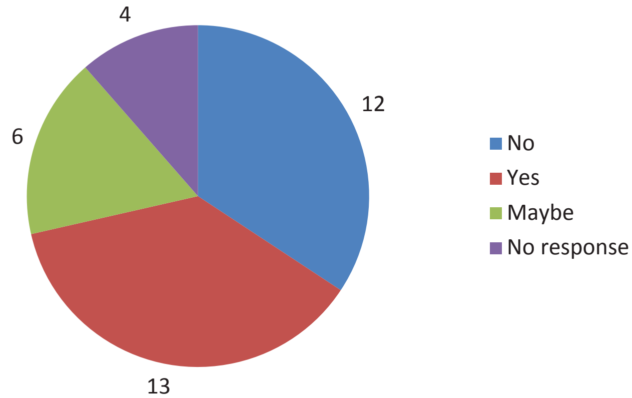


Figure 1.3. Pie chart depicting agency response to the question "Is the design and construction of new PCC/PCC composite pavements of interest to your agency?"

- Gathering information on individual agencies' experiences with composite pavements and identifying the appropriate contacts for development of guidelines and construction specifications.

A list of key agencies to be contacted was developed. These agencies included all 50 states of the United States, the District of Columbia, the provinces of Ontario and Quebec in Canada, and Austria, Belgium, the Czech Republic, Germany, United Kingdom, the Netherlands, Italy, France, Spain, Sweden, South Africa, and Australia. The initial request consisted of a few questions, and agencies that responded positively were contacted for additional information on specific field sections. Responses were received from 35 of 51 (69%) of the U.S. agencies and 7 of 14 (50%) of the international agencies contacted. The results of the survey are summarized in Figures 1.2, 1.3, and 1.4 and detailed in Appendix C.

Summary of European Practices

Many European countries have been constructing PCC/PCC composite pavements for several decades and have substantial experience with the design and construction of composite pavements. Members of the SHRP 2 R21 research team conducted a trip to some of these European countries to better understand and document their experiences with the construction of composite pavements. Tompkins, Khazanovich, and Darter (2010) described case studies visited in the Netherlands, Germany, and Austria.

Austria has been very active in regular two-layer PCC paving for concrete pavements, and the standard concrete pavement in Austria is constructed according to two-layer PCC specification. Austria also has a great deal of experience with the recycling of concrete pavements. In the late 1980s, Austria undertook the long process of recycling PCC pavements that

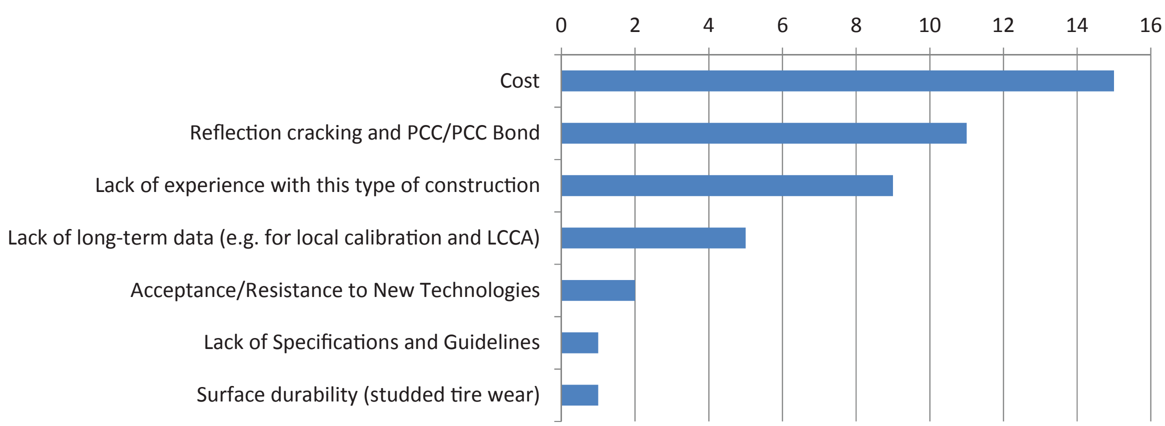


Figure 1.4. Key concerns of agencies regarding construction of PCC/PCC composite pavements in their jurisdictions.

were 30 years old or more (some of which already were overlaid with HMA) along the A1 motorway into new PCC/PCC composite pavements. This experience has led Austrian researchers to claim that the recycling concept is "an important innovation that is both economically and environmentally advantageous." Two-layer construction requires a consistent, quality effort. Construction techniques were one of two main areas of emphasis in Germany and Austria (the other emphasis being the quality of the aggregate in the upper layer). As part of the research trip to Europe, the research team studied construction guidelines, specifications, and practices for constructing PCC/PCC composite pavements and surveyed pavements constructed more than 15 years ago, as detailed in Tompkins, Khazanovich, and Darter (2010).

Two-layer PCC paving has been used in countries such as Austria, Switzerland, Belgium, the Netherlands, France, and Germany with regularity since the 1980s, and some much earlier. PCC/PCC composite pavements are becoming more common as the techniques are refined further. European research related to PCC/PCC composite pavements includes construction techniques and the use of recycled materials in the lower layer.

Dutch, German, and Austrian researchers report that composite pavements provide similar structural performance as an equivalently thick single layer at the same price in Europe, with the added benefits of higher quality, longer life, and friction and noise reduction because of the high-quality top layer. Furthermore, composite pavements allow for the optimization of costs and materials throughout the pavement cross section because:

- High-quality materials can be used in lesser quantities in the upper layer, where they will be of the most benefit to the system.
- Lower quality (cheaper) materials can be used in greater quantities and in the lower layer, where they will contribute structurally without detracting from the quality and performance of the overall pavement.

Although there are obstacles to the adoption of composite paving in the United States, it is clear from the European experience that overcoming these obstacles will result in high-quality, durable, and sustainable composite pavements.

Distress Mechanisms

Key failures in typical PCC pavements should also be considered for PCC/PCC composite pavements. These failure mechanisms include

- Bottom-up fatigue cracking for JPC;
- Top-down fatigue cracking for JPC;
- Longitudinal fatigue cracking for JPC;
- Punchouts for CRC; and
- Joint faulting for JPC.

These individual failure mechanisms are not expected to be a greater concern in PCC/PCC composite pavements than in conventional PCC pavements. PCC/PCC composite pavement may also experience some debonding between the layers. Depending on the materials chosen for the lower PCC layer, durability problems may arise in that layer. Details of these distress mechanisms and how they relate to the design of composite pavements are discussed in Appendix E.

Longitudinal Fatigue Cracking

Longitudinal cracking can be a concern for PCC pavements provided that a PCC pavement is very thin and nondoweled or a significant shrinkage and built-in curling occurs in that pavement. Given that PCC/PCC pavements are sufficiently thick as a result of their layered structure, it is not anticipated that longitudinal cracking caused by insufficient thickness will be a concern for PCC/PCC. Experts on composite pavements in Europe concurred that for a slab consisting of heterogeneous layers, shrinkage and built-in curl and the resulting threat of longitudinal cracking is no more a threat to PCC/PCC than to a structurally similar, single-layer PCC pavement. In addition, this failure was not observed during field surveys of PCC/PCC pavements or was during the 1992 European tour of PCC/PCC pavements.

Bottom-Up and Top-Down Fatigue Cracking

Both bottom-up and top-down cracking are important modes of failure in single-layer concrete pavements. The main instigator of these modes of cracking is a combination of traffic loading and the "curled," deformed slab that results from built-in curling and warping and any combination of either temperature or moisture gradients through the slab. Once the slab is in a deformed convex or concave shape, traffic loading creates a cantilever effect that can result in bottom-up or top-down transverse cracking in the pavement slab.

Curling is as prevalent in two-layered slabs as it is in single-layered slabs, and for this reason transverse cracking is an important mode of failure for two-layered composite PCC/PCC pavements. However, after discussions with experts in Europe and from simulations using Lattice3D, it was evident that the built-in stresses in a given two-layered PCC/PCC slab are not exacerbated simply by the layers being heterogeneous. Provided that a two-layered system is constructed using wet-on-wet methods, the slab's performance in transverse cracking is not significantly different than that of a structurally equivalent single-layered conventional PCC pavement.

Joint Faulting

Although joint faulting is a concern for single-layer JPCPs, it is not more of a concern in PCC/PCC composite pavements than in JPCPs. As in the case of the heterogeneous-layered slab response to transverse cracking, the presence of heterogeneous layers in a slab does not exacerbate faulting relative to that of a structurally equivalent JPCP. Joint faulting is no more or less likely to occur in PCC/PCC than it is in single-layered PCC due to the classical causes of joint faulting: base, subbase, subgrade erodability; subpar load transfer efficiency (LTE); or oversaturation of the base near the joint, and the same models for JPC pavement faulting can be used for PCC/PCC composite pavements.

Debonding

Debonding is a particularly challenging issue that is aggravated by shrinkage and thermal gradients through the heterogeneous layers at early ages. With proper wet-on-wet construction techniques, debonding of the two PCC layers is not expected to be an issue. However, as the time between placing the two PCC lifts increases (greater than 90 minutes), the lower PCC lift starts hydrating, and the surface of the lower lift may no longer be "wet." In such situations, the bond between the two lifts potentially can be compromised. To account for debonding in PCC/PCC, the SHRP 2 R21 research considered nonuniform shrinkage, nonuniform thermal expansion/contraction (especially at early ages), nonlinear thermal gradients, nonuniform heat of hydration, and crack formation and propagation.

Freeze–Thaw Durability and Performance Complications of RCA Use

The use of RCA or local materials of lower quality can be expected in the lower layer PCC mix. Therefore, durability is an important consideration in the performance of PCC/PCC composite pavements. This is especially important given that a key difference between RCA and natural aggregate is the variability in the absorption capacity of different RCAs, attributable in part to the existing mortar surrounding the original aggregate.

Construction Defects

Construction defects that occur during placement of PCC/PCC composite pavements are the same type as those found in typical JPCP or CRCP. Construction defects include vibration issues, such as inadequate consolidation of the PCC mixes around dowel bars or reinforcing steel, overmixing of the two PCC lifts, improper dowel bar placement, PCC mix issues (such as slump, gradation, temperature, and so forth), improper texturing or curing, and mechanical issues related to the paver. Construction defects can be reduced only with an adequate quality control/quality assurance (QC/QA) and inspection program.

Use of PCC/PCC Composite Pavements

Key questions often asked with regard to PCC/PCC composite pavements are: Where will composite pavements be used, and what will be the demand? PCC/PCC composite pavements allow the pavement designer to design pavements using the best qualities of two different PCC mixes to produce a more functional and economical structure that generally is cost-effective in terms of service throughout its life. Specifically, PCC/PCC composite pavements are optimal solutions for the following situations:

- When PCC/PCC is the less expensive alternative. In some design situations, based on materials, climate, traffic, and support conditions, the life-cycle costs for PCC/PCC composite pavements can be lower than those of conventional PCC pavements. This is particularly true when quality local aggregates are not available or aggregates are expensive because they need to be hauled long distances to the project location. In these situations, a high-quality PCC surface can protect the structural integrity or avoid the polish potential of the lower PCC layer made for lower quality materials.
- Where low maintenance pavement is desired. In urban areas with high costs of lane closures, rapid renewal is paramount. PCC/PCC pavements can be designed for the pavement to have a long life, structurally speaking (if durable materials are used in both layers). The high-quality PCC surface can be retextured rapidly through diamond grinding (or other methods) with minimal disruption to traffic over time. The retextured surface can also be expected to have high durability because of the hard aggregate and PCC mix quality and strength.
- When recycling is an option. Many urban areas and some rural areas exist with old PCC pavements that can be removed and processed and recycled directly back into lower layer PCC for use in PCC/PCC composite pavements. This provides excellent improved sustainability opportunities for pavements.
- When low pavement noise is needed, such as in urban areas with large populations in close proximity to the pavement. The high quality of the surface PCC layer makes any surface texture durable. As such, low-noise textures, such as those achieved with conventional diamond grinding, next-generation diamond grinding, EAC, and others, can be expected to last longer and, when needed, can be redone with another durable surface texture.
- Where avoidance of certain distress types related to the PCC surface is needed. A higher-strength PCC surface layer with high-quality aggregates may be beneficial in

reducing or eliminating top-down cracking, surface wear down (wheelpath rutting from studded tires), and polishing of the surface.

Differences between Conventional JPCP or CRCP and PCC/PCC Composite Pavement

There are several key differences that should provide for superior performance of a new PCC/PCC composite pavement compared with that of conventional JPCP or CRCP.

- Excellent surface characteristics from the thin high-quality concrete surface layer. These include low noise, high friction, very good initial smoothness, minimal wear over time, and high durability over a long time period (beyond 20 years) even under harsh weather conditions.
- Long-life structural design of the lower PCC layer (e.g., designed for minimal fatigue damage over a period of 40 years or more, which may require a thicker layer), where lower cost materials can be used.
- Higher-strength PCC surface layer may be beneficial to reduce or eliminate top-down cracking (depending on thickness, strength, climate, traffic, and other factors), CRCP punchouts, and surface wear down (wheelpath rutting from studded tires).

CHAPTER 2

PCC/PCC Test Sections

Introduction

The field composite pavement sections used in the structural modeling included a combination of special research sections in the United States and Canada and regularly constructed projects in Europe.

1. Regularly constructed projects in Europe:
   a. Austria is one of the most experienced countries with regard to construction of two-lift PCC/PCC composite pavements, which is their standard design for PCC pavements. Two projects on A1 in Austria were surveyed as part of the R21 European trip. These sections were constructed in the mid-1990s.
   b. Germany is also very experienced in the construction of composite pavements. As part of the R21 European trip, a section of PCC/PCC composite pavement, constructed in 1995, was surveyed on the A93.
   c. The Netherlands also routinely constructs PCC/PCC composite pavements. A section on N279, constructed in 2000, was surveyed as part of the R21 European trip.
2. Specially constructed research sections:
   a. MnROAD included two PCC/PCC composite sections on I-94 under heavy truck traffic and severe weather conditions:
      - Cell 71 was constructed in May 2010 under SHRP 2 R21. This section was 3-in. PCC over 6-in. PCC (using RCA). Joints were spaced at 15 ft and 1.25-in. diameter dowels were used at the transverse joints. This section exhibited no distress after a full year of 1 million heavy trucks.
      - Cell 72 constructed in May 2010 under SHRP 2 R21. This section was 3-in. PCC over 6-in. PCC (using low-cost high fly ash content PCC). Joints were spaced at 15 ft, and 1.25-in. diameter dowels were used at the transverse joints. This section exhibited no distress after a full year of 1 million heavy trucks.
   b. Experimental PCC/PCC sections have been constructed on State Route 45 in Fort Myers, Florida; US-75 in Rock Rapids, Iowa; K-96 in Haven, Kansas; I-70 in Salina, Kansas; I-75 in Detroit, Michigan; and A15 in Montreal, Quebec, Canada. These sections constructed in the last four decades differ with respect to mix designs for the PCC layers and surface textures. These sections generally were constructed as relatively short experimental sections (except I-70 in Kansas, where the total project length with seven different surface textures is several miles long) with special design features and not as part of routine construction in these states.

Figure 2.1 shows the geographic locations of the PCC/PCC composite sections. The sections can be seen to offer a reasonable spread across different geographic and climatic conditions in the United States and Canada. The sections in Germany, Austria, and the Netherlands are also shown.

These PCC/PCC composite pavement sections include several different types of surface textures, including EAC, conventional diamond grinding, next-generation diamond grinding, longitudinal tining, longitudinal grooving, Astroturf drag, and Astroturf drag and longitudinal grooving.

These sections show a range of designs, including the following:

- The thin PCC surface layers range in thickness from 1.5 to 3.5 in. The top layer PCC varied considerably from one section to another with respect to aggregates (types, hardness, gradation, and so forth), cement content, and use of SCMs such as fly ash and pozzolan, use of admixtures, and other mix properties.
- The thicker PCC lower layers range in thickness from 6 to 11.8 in. The lower layer PCC varied considerably with respect to mix designs and included conventional PCC, PCC with RCA, PCC with high fly ash content, PCC with low cement content (econocrete), and inclusion of ASR susceptible aggregates.

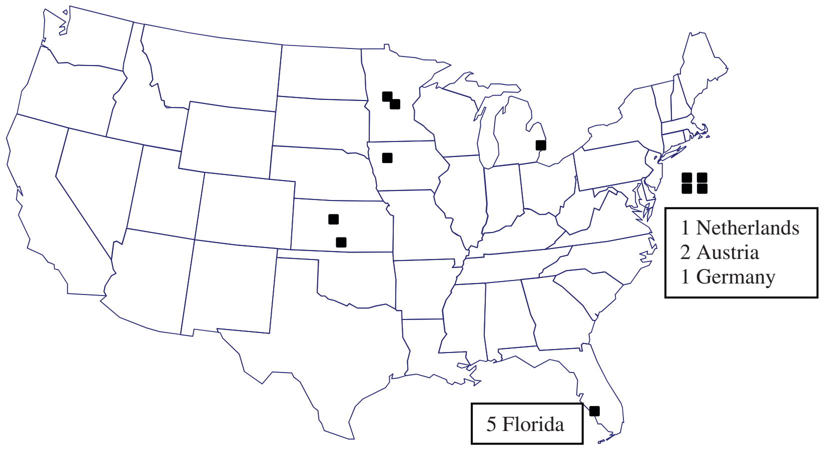


Figure 2.1. Map showing geographic dispersion of PCC/PCC sections.

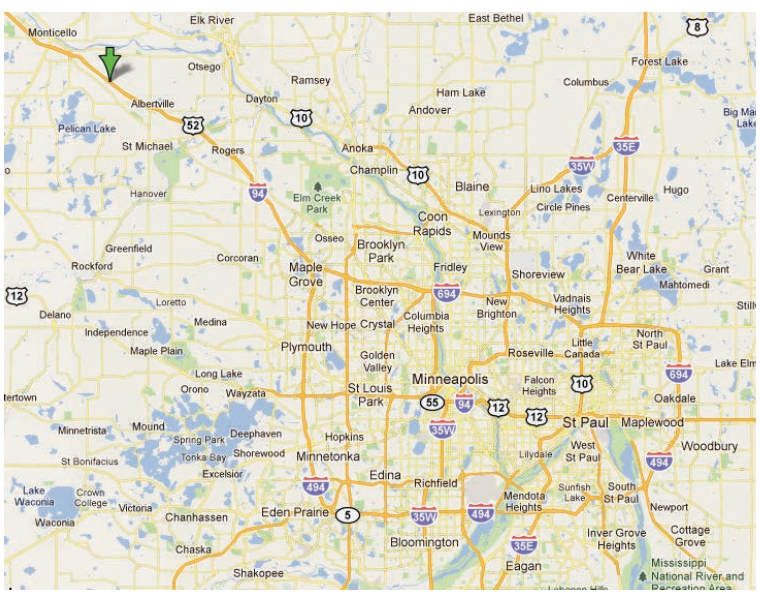
Test Sections at MnROAD

Introduction

In May 2010, two full-scale PCC/PCC test sections were constructed on I-94 at MnROAD to emulate best practices of constructing PCC/PCC composite pavements. Before the construction of the mainline test sections, a 200-ft two-lane test strip was constructed at the MnROAD facilities. A summary of the construction of the test section from initial site grading and aggregate base compaction to PCC placement and instrumentation installation is presented in this section. Details of each of these topics are included in Appendix F. Figure 2.2 shows the location of the MnROAD test section relative to Minneapolis. An aerial view of a portion of the MnROAD facility is shown in Figure 2.3.

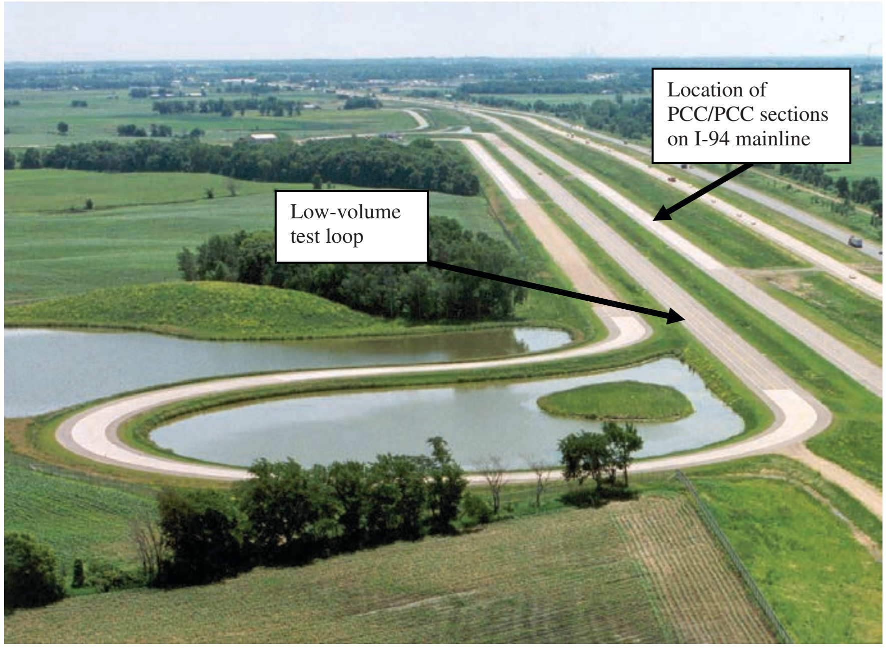
Design and Specifications

The project consisted of recycling an existing concrete pavement; the coarse aggregate (RCA) from the recycled pavement was used to construct the lower PCC layer for one of the PCC/PCC test sections. PCC/PCC sections constructed at MnROAD were designed to feature a 3-in. high-quality PCC layer over a 6-in. low-cost or RCA PCC lower layer. The



Source: © 2012 Google.

Figure 2.2. Location of the MnROAD PCC/PCC on I-94 near Albertville, approximately 40 miles northwest of Minneapolis.



Note: The mainline I-94 traffic is diverted to the center lanes during construction and testing.

Figure 2.3. Aerial view of MnROAD facility and location of PCC/PCC test sections.

term "low-cost" signifies that the PCC design was such that the lowest possible amount of cement and most inexpensive coarse aggregates were used by the contractor.

In addition, various textures were considered for the surface PCC layer. Because of its potential with respect to durability, texture, and noise characteristics, an EAC was chosen. The EAC was constructed for both PCC/PCC test sections. However, the first 475 ft of the EAC was considered the "learning area," and the EAC texture was later diamond ground using conventional grinding in the passing lane and next-generation grinding in the driving lane. The designs are summarized in Table 2.1 and Figure 2.4.

Table 2.1. PCC/PCC Designs for MnROAD Sections

Section		PCC over RCA PCC (Cell 71)	PCC over Low-cost PCC (Cell 72)
Upper PCC	Thickness	3 in.	3 in.
	Mix	High-quality portland cement (~616 lb/yd³) plus 109 lb/yd³ (15%) fly ash Class C (FAC)	High-quality portland cement (~616 lb/yd³) plus 109 lb/yd³ (15%) FAC
	Coarse aggregate	Crushed granite (maximum size ⅜ in.)	Crushed granite (maximum size ⅜ in.)
Lower PCC	Thickness	6 in.	6 in.
	Mix	Low-quality portland cement (~360 lb/yd³) plus 240 lb/yd³ (40%) FAC	Low-quality portland cement (~240 lb/yd³) plus 360 lb/yd³ (60%) FAC
	Coarse aggregate	50% RCA, 50% Minnesota DOT Class A, maximum aggregate size 1.25 in.	100% Minnesota DOT Class A, maximum aggregate size 1.25 in.
Base		8 in., Class 5 unbound	8 in., Class 5 unbound
Subgrade		Clay	Clay
Joint spacing		15 ft	15 ft
Doweling		1.25 in. placed on baskets at total PCC middepth	1.25 in. placed on baskets at total PCC middepth
Surface texture		Conventional diamond grinding, next-generation diamond grinding	EAC, conventional diamond grinding, next-generation diamond grinding

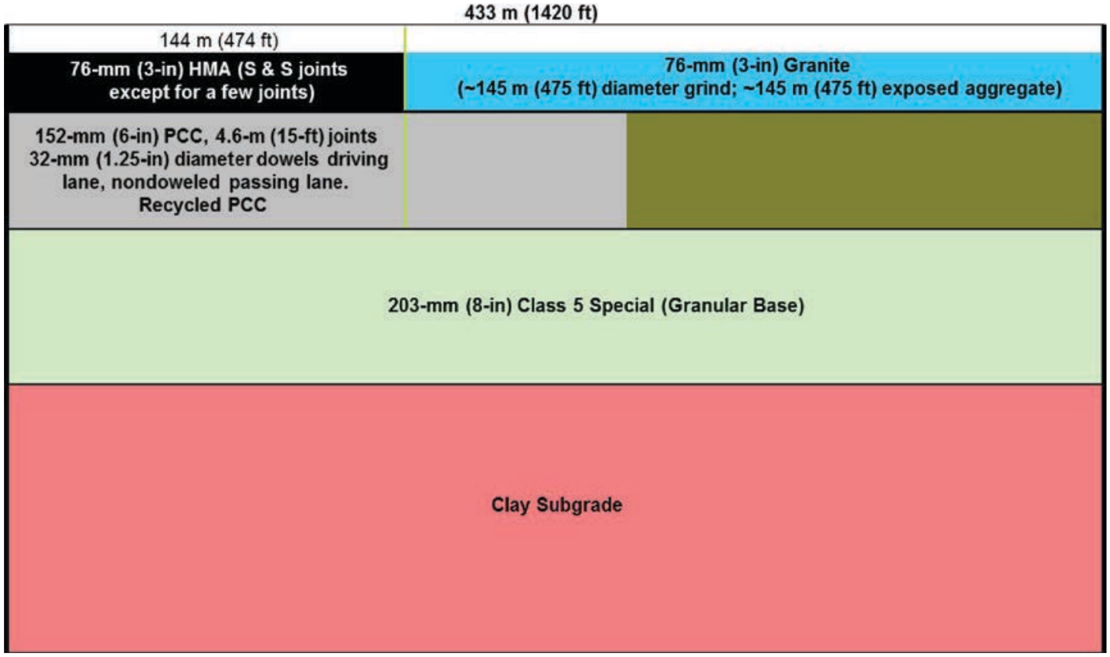


Figure 2.4. Layout of test sections at MnROAD (Cell 70, HMA/PCC; Cell 71, PCC/RCA PCC; Cell 72, PCC/low-cost PCC).

Construction of the Test Sections

The construction project was awarded to C. S. McCrossan of Maple Grove, Minnesota. WSB and Associates, Inc. was responsible for the administration of the construction contract and the inspections. Table 2.2 shows a timeline of the major steps involved in the construction process.

Because of the unique nature of this project, special provisions were used as part of the bid package to modify the DOT's existing specifications. The special provisions included

1. Salvage concrete pavement: Specifications for the salvage operation to recycle and reuse coarse aggregate from existing on-site concrete pavement.
2. Structural concrete: Specifications for the concrete mix design and aggregate gradations for both PCC layers and concrete design details.
3. Concrete curing and texturing: Specifications for the curing and texturing of the PCC surface particularly to obtain the EAC surface texture (application of curing/retarding compound, brushing, and so forth).
4. Concrete pavement joints: Specifications covering details of saw cutting the joints in the surface PCC layer.
5. PCC/PCC composite pavement operation: Sequence of paving activities for the construction of PCC/PCC composite pavements.

Table 2.2. Construction Timeline for Major Tasks

Major Task	Date
Salvage and recycling operations	April 12–16, 2010
Trimming and grading of subgrade	April 19–22, 2010
Aggregate base placement	April 23, 2010
Trimming base and preparing for PCC instrumentation placement	April 26–30, 2010
PCC placement and instrumentation	May 6 and 10, 2010
HMA shoulders	May 20, 2010
Open to traffic	June 7, 2010

Recycling Operations

The recycling operations consisted of breaking, removing, transporting, crushing, washing, screening, and stockpiling the concrete pavement material from an existing MnROAD cell to be used as coarse aggregate in the recycled concrete mix. The concrete portions of the existing cells were broken with a guillotine crusher (Figure 2.5), removed (Figure 2.6), and transported to a crushing location.

The crushing method and system determines some of the qualities of the RCA, such as mortar content and the gradation. An increase in the number of crushing processes reduces the mortar content (Sanchez de Juan and Gutierrez 2009). As specified, all joint material, reinforcing members, and other inert materials (such as wood) were separated from

Figure 2.5. Guillotine crusher breaking existing concrete for recycling into the lower PCC layer of the PCC/PCC composite pavement.

the concrete sections before the existing concrete was crushed into coarse aggregate. For this project, the contractor used an industrial crushing operation that included a primary jaw crusher (Figure 2.7) operating at less than full capacity and a secondary cone crusher (Figure 2.8), then washed, screened, and stockpiled. The jaw crusher jaws were distanced to adjust the maximum aggregate size produced. The cone crusher was used as secondary crusher to further remove the mortar from the natural aggregates. A cone crusher squeezes material between an eccentrically gyrating spindle and a bowl below. As the pieces are broken, they fall to the lower, more closely spaced part of the crusher and are further crushed until small enough to fall through the bottom opening.

Laboratory tests on the recycled aggregate (AASHTO T84 and T85) revealed that the RCA absorption was 2.93%.

Figure 2.6. Removal of existing concrete pavement for recycling.

Figure 2.7. The primary crusher was the jaw crusher operating at less than full capacity.

Subgrade Soil Grading and Compaction

A string line was set for trimming of the subgrade and the base. The subgrade was cut with a trimming machine (Figure 2.9) and compacted with a steel drum roller (Figure 2.10). Hand holes and conduits were set for the instrumentation cables (Figure 2.11). Testing was performed on the compacted subgrade using a dynamic cone penetrometer (DCP), lightweight deflectometer (LWD), and falling weight deflectometer (FWD). The Class 5 aggregate base was constructed in two 4-in. lifts.

PCC Mix Design

Numerous options for PCC mixes to be used in the top and lower lifts of the PCC/PCC pavement were explored, which

Figure 2.8. A cone crusher was used as a secondary crusher to further remove the mortar from the natural aggregates.

Figure 2.9. Trimming the subgrade using a string line and trimmer.

Figure 2.11. Installing conduits to carry and protect the instrumentation cables.

involved a series of iterations on mix design, followed by laboratory testing of the mixes (see Appendix Q).

For this SHRP 2 R21 project, a "high-quality" PCC mix for the upper layer of PCC/PCC composite pavements is defined as a PCC mix containing increased cement content (relative to the American PCC paving standard of roughly 500 to 600 lb/yd^3 [297 to 357 kg/m^3]) and a high-quality, very durable aggregate (i.e., granite). The aggregate in the upper lift must be gap-graded and of a maximum size no larger than 0.3 in. (8 mm). Although German and Austrian mix designs do not typically contain fly ash, it was used in mixes at MnROAD. In addition, the research team aimed for a PCC mix for the lower PCC that would contain reduced cement (relative to the standard described above), locally available natural fine aggregates, and a coarse RCA as a low-cost alternative coarse aggregate. All basic components of the lower-layer PCC were selected in light of a desire to reduce costs, investigate methods of sustainability, and investigate the reuse of materials into structural components. Each of the PCC mixes used is summarized below and in Table 2.3.

Figure 2.10. Compacting the subbase using a steel drum roller.

RCA PCC Mix Design

Per the special provisions, the RCA comprised 50% of the total coarse aggregate in the PCC mix. In addition, aggregate fines less than 4.75 mm (No. 4) and coarse aggregates greater than 25.4 mm (1 in.) used in the PCC mix were specified to come from virgin aggregate sources. The special provisions also required the contractor to clean and wash the RCA. As much as 10% of the total recycled coarse aggregate could consist of bituminous particles. The cementitious fraction was specified to consist of as much as 60% SCMs, including but not limited to fly ash. Fly ash replacement of 40% was approved and used in the final mix design. The main concern with regard to this mix had to do with the use of coarse RCA. As a result of these concerns, an extensive investigation into the use of RCA for structural PCC was conducted. This included laboratory work investigating aggregate absorption, gradation, freeze–thaw durability, aggregate washing/preparation, and methods of crushing, as detailed in Appendix F.

Low-Cost PCC Mix Design

Per the special provisions, the cementitious fraction was specified to consist of as much as 60% supplementary

Table 2.3. PCC Mix Design for PCC/PCC Construction at MnROAD

Materials	Weight per Cubic Yard (lb/yd³)		
	RCA PCC	Low-cost PCC	Upper Layer PCC
Water	234	173	283
Cement	360	240	616
Fly ash	240	360	109
Sand	1,200	1,263	843
California No. 1 (virgin aggregate, 1½-in. maximum aggregate size)	825	787	na
California No. 2 (recycled aggregate)	920	na	na
California No. 3 (virgin aggregate, ¾-in. maximum aggregate size)	na	1,102	na
⅜-in. Washed granite chips	na	na	843
½-in. Washed granite chips	na	na	1,133
Air entrainer	2 to 15 oz	2 to 15 oz	2 to 15 oz
Hydration stabilizer	na	na	0 to 5 oz
Water reducer	1 to 5 oz	1 to 5 oz	1 to 5 oz
Accelerator	0 to 30 oz	0 to 30 oz	na
Properties			
Water-to-cement ratio	0.39	0.29	0.39
Maximum slump	3 in.	3 in.	3 in.
Entrained air content	7%	7%	7%

Note: na = not applicable.

cementitious materials including, but not limited to, fly ash. Fly ash replacement of 60% was approved and used in the final mix design for the low-cost PCC mix. The main concern with regard to this mix had to do with setting time and early strength because of the high fly ash replacement percentage. However, it should be noted that although 60% is atypical, this level of cement replacement is possible and has been accomplished for other transportation concretes. Furthermore, a high percentage of cement has successfully been replaced using slag as the SCM in many construction projects throughout Europe.

Upper Layer PCC Mix Design

The upper layer PCC mix included high cement content (616 lb/yd³) in addition to 15% fly ash substitution. The mix incorporated polish-resistant, granite aggregates with a high cement content that would allow for an exposed aggregate surface texture. The maximum aggregate size of the coarse aggregate was specified as 9.5 mm. The mix included as-needed hydration stabilizer for slump retention.

The portland cement used was a Holcim, St. Genevieve Type ½ cement. The fly ash was a Class F, Headwaters Coal Creek fly ash. The fine aggregate was an Elk River Concrete Sand. The coarse aggregate comprised No. 67, ¾-in., and No. 4, 1½-in. Elk River gravel. The coarse aggregate for the top layer PCC comprised ⅜- and ½-in. washed granite chips from Martin Marietta. The water reducer, accelerator, and air entrainer were Sika products. The hydration stabilizer was a BASF product. The gravel aggregates and gradation of those aggregates were similar to those of conventional PCC pavements used by Minnesota DOT for the MnROAD facility.

PCC Mix Gradation

The research team elected to use an EAC surfacing for the demonstration slab and mainline sections. Although EAC is used successfully in Europe, challenges were faced by the Kansas and Michigan DOTs in applying EAC techniques in the United States. A key issue with regard to attaining a low-noise, high-durability EAC texture is the aggregate gradation of the PCC surface mix. Gradations for the PCC surface mix and the lower lift PCC mixes were chosen based on a combination of laboratory testing, communication with contractors and engineers and review of research reports from construction in Kansas and Michigan, and communication with engineers and contractors in Europe. Ideally, to obtain a high-quality EAC texture, a gap-graded mix (small percentage between No. 4 and No. 16 sieve sizes) with maximum aggregate size less than 8 mm is desirable for the PCC surface mix. This results in closely spaced aggregates with a negative surface texture. However, limitations of construction funds and sources of aggregates close to MnROAD necessitated modifications to the original specifications. The modifications included use of maximum aggregate size of 9.5 mm and a denser gradation. The gradation ranges specified in the original and updated specifications along with the final approved gradation are shown in Table 2.4.

Instrumentation Plan

To determine the overall response of the pavement to environmental loads, the physical response of the pavement and the climatic conditions within the structure were monitored. Environmental sensors were installed to document the temperature and moisture gradients that developed throughout the depth

Table 2.4. Aggregate Gradation for PCC Mixes

Designation	37.5 mm (1½ in.)	31.5 mm (1¼ in.)	25.0 mm (1 in.)	19.0 mm (¾ in.)	16.0 mm (⅝ in.)	12.7 mm (½ in.)	9.5 mm (⅜ in.)	6.3 mm (¼ in.)	4.75 mm (No. 4)	2.36 mm (No. 8)	1.18 mm (No. 16)	.030 mm (No. 50)	.015 mm (No. 100)	.0075 mm (No. 200)
Working range limits	±5	±5	±5	±5	±5	±5	±5	±5	±5	±4	±4	±3	±2	1.6% maximum
RCA PCC Mix														
As-written specifications	100	97–87	87–77	76–66	70–60	63–53	55–45	na	41–31	30–20	23–13	14–4	10–1	7–1
Updated specifications	100	100	95–80	85–70	na	70–55	60–45	55–40	50–35	45–30	35–25	10–2	10–0	5–0
Final approved PCC blend	100	100	88	73	na	54	46	na	41	37	30	6	1	0.2
Low-cost PCC Mix														
As-written specifications	100	97–87	87–77	76–66	70–60	63–53	55–45	na	41–31	30–20	23–13	14–4	10–1	7–1
Updated specifications	100	100	95–80	85–70	na	70–55	60–45	55–40	50–35	45–30	35–25	10–2	10–0	5–0
Final approved PCC blend	100	100	89	76	na	64	56	49	42	37	30	6	1	0.2
Upper Layer High-quality PCC Mix														
As-written specifications	100	100	100	100	100	100	100	75–65	48–38	48–38	48–38	13–7	7–1	5–1
Updated specifications	100	100	100	100	100	100	100–95	75–65	55–45	40–30	35–25	13–7	7–1	5–0
Final approved PCC blend	100	100	100	100	100	100	98	69	48	33	29	11	2	0.4

Note: na = not applicable.

of the slab. Temperature sensors were located in each of the different pavement structures so that the seasonal, daily, and construction temperature profiles that developed could be documented. Moisture sensors were installed in the concrete to study the effects of the surface layers on the moisture distribution through the depth of the slab. Static strain gauges were used to monitor the effects of uniform moisture and temperature changes, as well as moisture and temperature gradients on the slab shape. Figure 2.12 shows the elevation and plan view of the instrumentation layout.

The response of the structures to applied vehicle loads was measured using dynamic strain sensors installed within the pavement structure. An on-site weather station recorded air temperature, relative humidity, and wind speed every 15 minutes. The various sensors installed at the MnROAD test section are described here:

- Temperature sensors: Thermocouples were used for measuring temperature throughout the pavement structure. Critical locations for monitoring temperature included the midslab, the slab corner, and midslab adjacent to the longitudinal joint.
- Concrete moisture: To measure moisture levels within the concrete, 24 Sensirion SHT75 relative humidity and temperature sensors were installed. The SHT75 sensor is a relatively small (approximately 0.75 × 0.25 × 0.125 in.) and cost-effective means of measuring relative humidity in concrete.
- Static strain: The PCC response to static loads generated was measured with vibrating wire (VW) strain gauges. The VW gauges were used to provide several critical pieces of information related to the performance of the PCC layers, including:
  - Degree of bonding between the PCC layers;
  - Slab curvature; and
  - In-place drying shrinkage and thermal coefficient of expansion.

  Geokon Model 4200 VW concrete embedment strain gauges were used. The gauges operate on the VW principle. A steel cable is tensioned between two metal end blocks. When the gauge is embedded in concrete and concrete deformations occur, these end blocks move relative to one another. The movement of these end blocks influences the degree of tension in the steel cable. The tension in the cable is quantified by an electromagnetic coil, which measures the cable's resonant frequency of vibration on being plucked. The sensor is also equipped with a thermistor so corrections for temperature can be made.
- Dynamic strain: Dynamic strain sensors were installed to measure the pavement response to loads applied by truck traffic and the FWD. The dynamic sensors used in the concrete were Tokyo Sokki PML-60-2L strain gauges. The Tokyo Sokki PML-60-2L consists of a copper/nickel alloy resistance foil gauge attached to two lead wires. This foil is attached to an electrically insulated backing and, with the use of a special adhesive, is attached to one of two thin acrylic plates. The two plates are sealed together to protect the gauge from contamination when installed in the concrete. These acrylic plates are coated with a fine, granular material to improve bonding to the surrounding concrete. The insulated backing expands and contracts with the concrete, causing the resistance in the foil gauge to change.
- Data acquisition: Automated static and dynamic data (or "online" data) were entered into the MnROAD database through the MEGADAC acquisition system. This system of dynamic cabinets, computers, fiber-optic cables, and copper-wire sensors automatically retrieved data from instruments at the MnROAD facility in Albertville and returned this information to the MnROAD database in Maplewood.

Instrumentation Installation

Figure 2.13 shows dynamic strain gauges, static strain gauges, humidity sensors, and thermocouples affixed to the aggregate base prior to PCC placement. The lead wires are buried in the sublayers and carry the signal from the gauges to the data acquisition unit. The gauges were packed in the concrete, and the concrete was vibrated with a hand vibrator to ensure consolidation of concrete around the gauges (Figure 2.14).

Paving Operations

Paving operations for the R21 test sections at MnROAD began on April 28, 2010, with the construction of a 200-ft demonstration slab, and concluded on May 10, 2010, with the completion of 950 ft of test sections along the mainline (I-94) test area. The two-lift paving used two GOMACO model GHP2800 pavers and a belt placer between the two pavers to place fresh mix for the upper lift (Figure 2.15).

Many aspects of the paving were similar to those of a normal single-layer PCC pavement. As detailed in Table 2.1, the pavement design included 1.25-in. dowels, placed at the middepth of the full PCC slab using dowel baskets. Furthermore, the design included 30-in., no. 4 tie bars spaced at 30 in. to reinforce longitudinal joints; the bars were inserted using a tie-bar inserter attachment on the first paver. One difference in the use of two pavers in PCC/PCC versus single-layer PCC is that the upper lift paver was adjusted to "crown" the lower lift slab by 0.75 in. on each side; that is, the second paver paved a lift 1.5 in. wider than the first paver in the train.

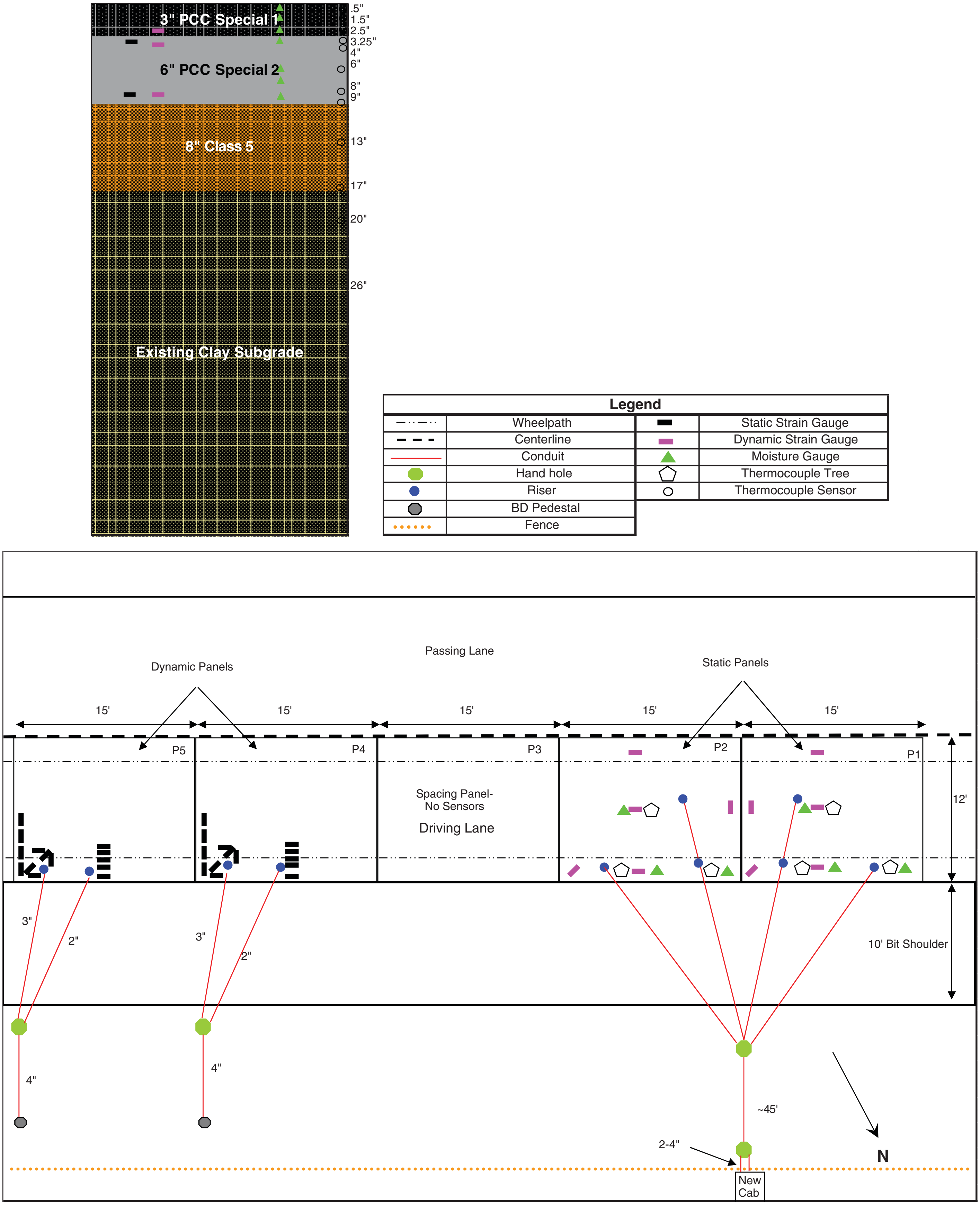


Figure 2.12. Elevation and plan view of instrumentation layout for PCC/PCC test sections at MnROAD.

Figure 2.13. Instrumentation installed prior to placement of the PCC to measure pavement responses to temperature and traffic loads (static strain gauge,* top left*; dynamic strain gauges,* bottom left*; humidity sensors,* top right*; temperature sensors [thermocouple tree],* bottom right*).

Figure 2.14. Overview of instrumentation packed in concrete before PCC paving.

Figure 2.15. Paving train constructing R21 test sections along I-94 at MnROAD (from left to right: the mixer truck, first paver, belt placer, and second paver).

Unlike the paving trains encountered in Europe, which use low-frequency vibration on the second paver only, the paving train used for the MnROAD construction used vibrators in both pavers (the first paver set to 8,000 vpm, the second paver set to 4,000 vpm). Methods differ on this point, in part because of the use of automated dowel bar inserters in Europe. Because of the use of dowel baskets for placing the dowel bars, vibrating the lower lift PCC was necessary to consolidate the PCC mix around the dowel bars. However, given the height of the dowel baskets (4.5 in.) and the small thickness of the lower PCC lift (6 in.), the vibrations were surficial. The vibration in the second paver was low and shallow to avoid overmixing the two PCC layers, particularly at the interface, and thus ensure the integrity of the individual layers.

The key complications with respect to the paving were those brought about by delays in the delivery of PCC for the two lifts. Although the construction specifications indicated that paving of the second lift was to occur no later than 90 minutes after the first lift (ideally no later than 60 minutes), on all three occasions of PCC/PCC paving (demonstration slab and two mainline sections), the paving was frequently stalled for more than 90 minutes as crews waited for batched upper lift PCC to arrive. During the construction of the demonstration slab, mix delivery delays led to 90- to 100-ft stretches of the placed lower lift being exposed to the environment for more than 120 minutes before the second lift was placed.

These delays resulted in a few problems that could be observed immediately on-site during paving. The most apparent was the setting up of concrete in the auger, the grout box, and on the profile pan of the paver. Frequent delays allowed the concrete to hydrate and attach to surfaces, normally assumed to be smooth, that physically form the slab. When paving resumed after long delays, concrete that had clung to these surfaces would "tear" at the freshly paved concrete, resulting in the need for additional finishing. Figure 2.16 illustrates the tearing.

The delays in the delivery of the upper PCC compromised the pavement, given that the weather during the demonstration slab paving was unseasonably warm, sunny, and windy. Temperatures were between 60°F and 69°F, the sun was strong with no clouds, and the wind was steady at 5 to 10 mph with occasional strong gusts. These conditions are especially critical when the slab in question was composed of the early batches of PCC that arrived for the demonstration slab, which were considerably dry (with measured slump on-site of 0.75 to 1 in. from batch to batch). This early dry PCC was used for the 90 to 100 ft of lower lift placed at the beginning of the demonstration slab, for which more than 120 minutes passed before an upper lift was placed. Figure 2.16 illustrates the most exaggerated of the shrinkage cracking encountered in these early slabs.

Figure 2.16. At left, "torn" edges and surface caused by concrete setting on various parts of the paver. At right, coring showed poor mix consolidation in the lower PCC at select locations.

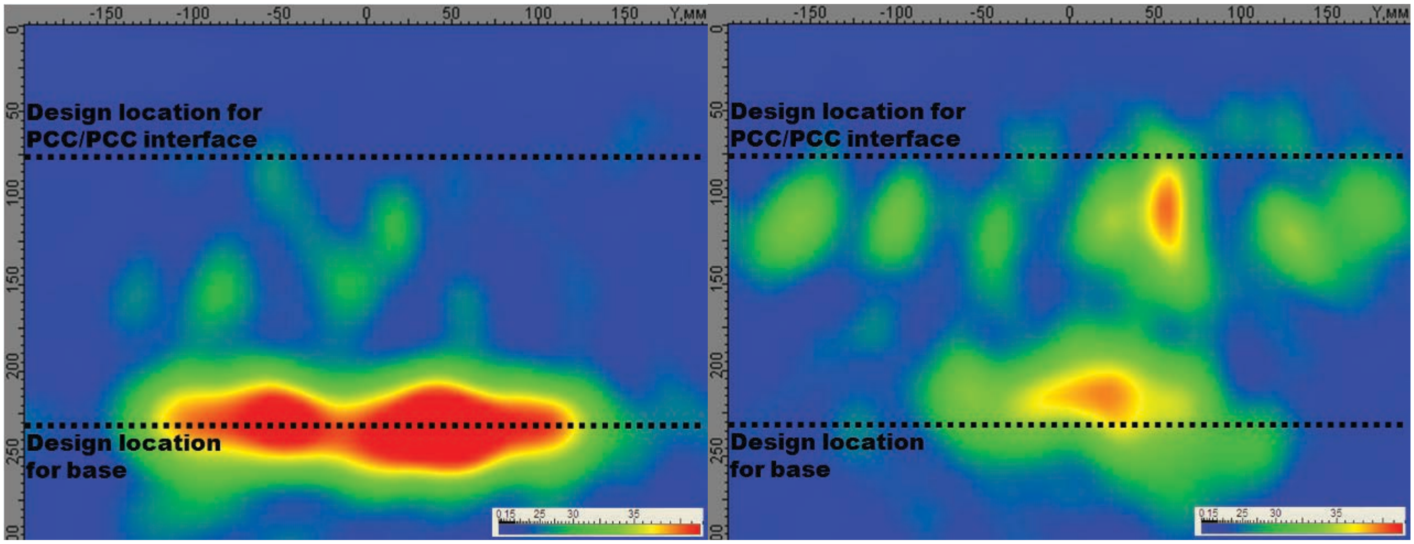


Figure 2.17. At left, a typical tomogram from the PCC/PCC demonstration slab at MnROAD; at right, tomogram with ultrasonic reflection near the depth of the PCC-PCC interface.

Another concern about delays included the integrity of the bond at the interface of the two lifts. An ultrasonic tomography testing device was used to assess the bond at the transverse joints and at midslab locations on the demonstration slab. Tomograms from two representative scans are illustrated in Figure 2.17.

Ultrasonic reflection occurs only noticeably at the start of the base layer (measured as approximately 8 in.) in the tomogram at left in Figure 2.17. In the tomogram at right, however, significant ultrasonic reflections are measured at a depth of approximately 4 in., near the interface of the two PCC layers. This reflection near the interface may be indicative of a poorly developed bond between the two layers of PCC, or it may be indicative of other problems (such as tearing and voids) in the pavement. The figure at left is further evidence that the composite layers, from the view of the tomogram, are a unified layer, whereas the reflections in the figure at right suggests the possibility of internal distress (these conclusions were confirmed by cores taken from the demonstration slab).

Many lessons (with regard to mix delivery, lower PCC slump, instrumentation installation, saw cutting, and so forth) were learned from the construction of the demonstration slab that was incorporated during the construction of the mainline test sections. However, despite the problems encountered during the construction of the demonstration slab, a large portion of the PCC/PCC constructed (particularly the second 100 ft) was found (based on coring and trenching evidence) to meet the requirements of the design with good integrity of the individual PCC layers (no intermixing) and good bond between the PCC layers (Figure 2.18).

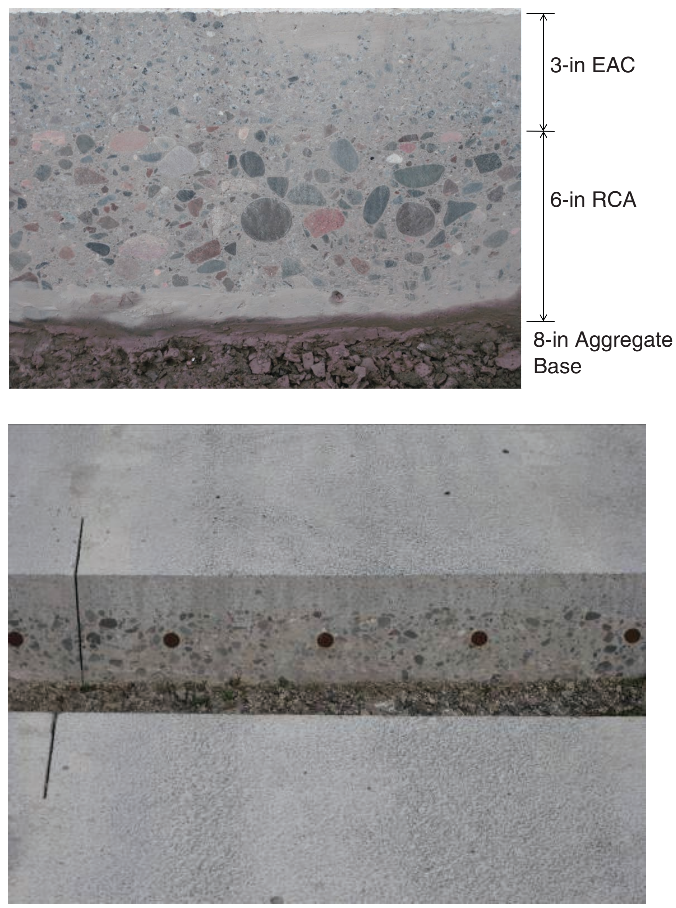


Figure 2.18. Cross section of the PCC/PCC demonstration slab constructed at MnROAD showing very good integrity and bond between the two PCC layers.

Paving the mainline sections progressed at a rate of between 1 and 4 ft per minute, and the project contractor was confident that this rate would be greatly increased with a larger project and a consistent supply of PCC for paving. While joint cuts sawed to a depth of 3 in. did not necessarily propagate well on the demonstration slab because of construction delays and dry mixes, for the mainline sections all saw cuts were found to propagate as anticipated for a single-lift equivalent slab. The specifications were changed to saw cut one-third of the total thickness or top layer thickness plus 0.5 in., whichever is greater. In this case, it was a minimum of 3.5 in.

Mix Design and Delivery

One of the more challenging aspects of the PCC/PCC sections constructed at MnROAD was the PCC itself. This challenge presented itself in: (1) the development of a mix design that uses alternative materials and/or meets low-cost specifications and (2) terms of the logistics behind batching and delivering concrete to meet the demands of the paving operations.

The characteristics of the three PCC mixes used are summarized in Tables 2.3 and 2.4. The most conventional of the three is the PCC mix used for the upper lift, whereas the PCC used for the lower lifts presented challenges in its use of high fractions of fly ash and/or RCA. The specification for as much as 60% fly ash in the lower lift PCC was inspired by the high-fraction of SCM replacement in the new St. Anthony Falls (I-35W) bridge in Minneapolis, which used as much as 81% SCM replacement in its mixes. The existence of a lower lift was also viewed as an opportunity to use lesser quality aggregates. To this extent, a thorough review of existing research on the use of RCA in PCC was performed. This review concluded that RCA was a viable coarse aggregate for the lower lift PCC provided the RCA came from a known source, fines were excluded, and the stockpile was properly maintained (i.e., kept saturated to eliminate variable absorption as a concern).

Both the use of high-fraction SCM replacement and RCA came with the challenges discussed above. These challenges were met with mixing preliminary batches of each mix in the laboratory (Figure 2.19). As a result of these tests, adjustments to the mixes were made.

Although this preliminary work addressed some challenges, the two-lift paving at MnROAD revealed a larger problem for the concrete in terms of consistency from batch to batch. The challenge of providing a consistent batch from truck to truck was thought to be overcome after the demonstration slab. However, paving on the mainline again suffered from the consistency problem, particularly in the case of the lower PCC mixes, whose as-delivered slump varied between 0.25 and 2.75 in (the target slump was 1 in.). Although the causes of this inconsistency are still uncertain, there are numerous possible causes:

- The use of RCA requires close attention. The contractor had secured RCA of a known source and had washed the RCA of fines; however, the preparation of the RCA for batching, most notably, its degree of saturation, was not consistent. One explanation of the inconsistency from batch to batch, as evident in the variable slump, is the inadequate maintenance of the RCA stockpile (Figure 2.20). It is possible that portions of the stockpile had been allowed to dry.
- Another concern to emerge from the use of RCA was the underestimate of unprocessed recycled concrete required

Figure 2.19. At left, PCC specimens cast in preparation for the paving and brushing of the PCC/PCC demonstration slab; at right, top-surface PCC mix being placed in front of the second paver on-site at MnROAD.

Figure 2.20. RCA stockpile being processed and saturated at a concrete recycling facility.

to achieve a coarse aggregate of a desired size. Early estimates missed the amount of recycled concrete required, which led to only 275 ft of PCC/PCC using the RCA mix being paved, instead of the originally planned 475 ft. Note that RCA was used in construction of Cell 70, the HMA/PCC test section, and the PCC/PCC demonstration slabs and several truckloads had to be rejected (for a variety of reasons, including slump and entrained air).

- The ready-mix supplier used by the contractor did not frequently design concretes using a large fraction of fly ash. As a result, the ready-mix supplier's inexperience in fly ash led to the mix designs being inadequately composed to handle such large amounts of this SCM (in terms of water demands, admixtures, and so forth).
- A final challenge in meeting the mix design for the PCC/PCC pavements was the use of a local ready-mix supplier for the PCC/PCC concrete. Because of the small size and scope of the project, the contractor used a local ready-mix plant, instead of a mobile batching plant, and the use of one plant instead of two (as observed in Europe).

Thus, the observed delays in mix deliveries may have been attributable to the use of a ready-mix supplier that was inexperienced in certain mix designs and in delivering those designs in sufficiently large volumes. During the post-construction review, the contractor maintained that one plant was enough to accommodate the three mixes for this project, but the contractor also stated that a ready-mix plant was not sufficient to provide consistency in mix design and delivery. The contractor was confident that for a larger project, using the company's own mobile batching plant and staff (rather than subcontracting this work to a local ready-mix supplier), mix consistency/delivery would not complicate PCC/PCC paving.

Figure 2.21 shows the placement of the lower PCC mix (note the dryness and low slump of the mix, which was specified at 1 in.). Figure 2.22 shows the placement of the upper PCC mix above the stiff lower PCC layer using a belt placer. The lower PCC layer was stiff enough to carry the impact and weight of the upper PCC layer while being placed (and the weight of an average size person as seen by the footprint impressions on the lower PCC). The footprints also show that the lower PCC layer was still "wet," which is necessary for a good bond between the two PCC layers.

Figure 2.21. Placement of the lower PCC mix.

Surface Texturing

The finishing platform used for the construction was a GOMACO model TC600 with Power Pavers Inc TC 2700T spray attachment. After paving, a curing/retarder compound (MBT Reveal from BASF Building Systems) was applied to the surface that both acted as a moisture barrier (curing agent) and as a retarder of hydration in the PCC surface. During construction of the demonstration slabs, early applications

Figure 2.22. Placement of the upper PCC mix on the lower PCC layer.

*Figure 2.23. Treated surface and finishing platform (*left*). Equipment for EAC brushing (*right*).*

of the surface treatment were delayed because of mechanical problems on the finishing platform, which provided insufficient pressure to the spray nozzles. The treatment (Figure 2.23) was intended to be applied almost immediately after finishing of the placed second lift; however, because of frequent delays, the treatment was applied anywhere between 60 and 90 minutes after the completion of paving a given segment. For the construction of the mainline sections, the nozzle heights were adjusted and wind guards were attached to the side of the curing cart to apply the compound more uniformly.

For the demonstration slab and the first day of mainline paving, brushing was initiated anywhere between 5 and 8 hours after paving of a given section had completed. The brush timing was based on limited laboratory tests, which did not mimic field conditions closely. To compensate for the lack of field experience, the surface was frequently tested at regular intervals, judging the brush readiness of the surface by the amount of cement and aggregate dislodged using a metal rod and/or handheld brush.

The brushing was accomplished using a small front-end loader with rotating wire brush attachment (Figure 2.23). The brushing was complicated by the inability of the operator to know the depth of texturing with any kind of precision. Thus, the brushing was done in multiple passes to gauge the level of cement removal between the aggregates, slowly revealing the EAC texture in pass after pass (Figure 2.24). The extent

*Figure 2.24. Surface after first pass with brush (*left*). Finished EAC surface (after wash) (*right*).*

Figure 2.25. Quality control tests for brushing: (**left*****) 25 cm^2 test to count aggregate peaks and (*****right*****) sand patch test to determine texture depth.***

of brushing was determined using a combination of a sand patch test and an aggregate peak counting test (Figure 2.25). More detail on these techniques can be found in Weinfurter et al. 1994. Although it was not specified, the aggregate peak counting test was an informal quality control for the brushing, adapted from Austrian methods. It aimed for a count of anywhere between 40 and 50 aggregate points per 25 cm^2 (3.88 in.^{2}), according to Haider et al (2006). The sand patch test was conducted according to ASTM E965 at intervals as a quality control measure during brushing to ensure that the mean texture depth (MTD) was between 0.8 and 1.2 mm (0.03 to 0.05 in.), as specified. This target was based in part on German and Austrian specifications for texture depth (0.6 to 0.8 mm and 0.8 to 1.0 mm, respectively). The completed average MTD for the EAC surface was 0.76 mm (0.030 in.). Two to five passes were needed to obtain the desired texture.

During the paving of the second PCC/PCC test section on the mainline, the construction encountered sudden onset of rain in the late afternoon. A vast portion of the finished, treated PCC/PCC paved was subjected to the rain before being covered with polyurethane sheeting. The delay in sheeting was caused by delays in paving and then in the application of the surface treatment. For these reasons, the brush timing was uncertain, and brushing was not initiated until the morning of the next day, 20 hours after the second lift had been placed and after the joints had been sawed. Although an EAC texture was still obtained, because of the various factors (localized washing of some of the curing/retarding compound by rain, low temperatures, variability in application of the curing compound, and so forth) the final texture was less uniform, with areas of good EAC texture and other areas of insufficient exposed aggregate. The situation was a reminder of the need to remain aware of the weather and sheet the PCC as soon after placement as possible should rain occur.

As shown in Figure 2.26 and described, the initial 475-ft portion of the PCC/PCC EAC texture (all of Cell 71 and a portion of Cell 72) was diamond ground, resulting in a total of three surface textures for the PCC/PCC sections:

- 475-ft passing and driving lane EAC;
- 475-ft driving lane next-generation diamond grind; and
- 475-ft passing lane conventional diamond grind.

As-constructed Properties

The FHWA Mobile Concrete Laboratory visited the R21 MnROAD construction site and collected PCC cores and material samples. The results, which are the average of two tests, are summarized in Table 2.5.

According to the Materials and Construction Optimization project (National Concrete Pavement Technology Center 2008), for adequate protection of concrete in freeze–thaw environment, spacing factor values less than 0.01 in. are desirable, although values less than 0.015 in. are commonly considered acceptable. The spacing factors of all the samples from this project were less than 0.015 in. In fact, the spacing factors of three of the four samples were less than 0.01 in. For specific surface, which indicates the size of the air bubbles, values greater than 600 in.$^{-1}$ are desirable. Three of the four samples that were tested had values higher than 600 in.^{1}. Based on the test results for specific surface and spacing factor, the mixtures used in the composite pavements project

Figure 2.26. The initial learning portion of the EAC texture was diamond ground using conventional grinding in the passing lane and next-generation grinding in the driving lane.

have good air void distribution for protection against freeze–thaw damage.

Noise Measurements

Construction of the EAC finish was attempted because of its durability and because it channels water away from the wheel path in multiple directions. However, the primary benefit of a properly constructed EAC surface is its noise mitigation potential. On-board sound intensity (OBSI) measurements of all of the finished composite pavement surfaces were collected to compare the sound intensity of the various surface finishes (Akkari and Izevbekhai 2011). Noise data from the EAC and diamond ground surfaces were compared with those from the HMA/PCC composite pavement.

The OBSI test setup consists of a sedan outfitted with four GRAS sound intensity meters, a Brüel & Kjær front-end four-channel frequency analyzer, and a standard reference test tire (SRTT). The microphones are suspended from the vehicle frame and positioned at 3-in. vertical displacement and at 2-in. lateral displacement from the leading and trailing end of the standard reference tire and pavement contact. The microphones are anchored to a free rotating ring mounted on the right wheel that allows the microphone assembly to be fixed in position and direction without inhibiting the rotation of the tire. The OBSI equipment is shown mounted to a sedan wheel in Figure 2.27.

PULSE noise-and-vibration software is installed in a connected computer. The computer receives and analyzes the data, categorizing the response into component third octave frequency output. Pavement noise response from the microphones is condensed into a third octave frequency sound intensity plot averaged for the leading edge and trailing edge. The OBSI parameter is the average of the logarithmic sum of the sound intensity at 12 frequencies (400, 500, 630, 800, 1,000, 1,250, 1,600, 2,000, 2,500, 3,150, 4,000, and 5,000 Hz). OBSI analysis is based on the AASHTO TP76-08 protocol. The results from OBSI testing done in 2010 are shown in Figure 2.28.

Figure 2.28 shows that the innovative diamond-ground finish (also called the next-generation) had the lowest OBSI throughout the 3 months tested. The traditional diamond grind had an OBSI similar to that of the hot-mix asphalt (HMA) surface. The EAC surface had the highest OBSI. There was not a considerable difference between the OBSI in the passing lane compared with that in the inside lane in either Cell 70 or 72. In a survey of exposed aggregate concrete pavements in Europe conducted by the National Concrete Pavement Technology Center, OBSI values were found to range from 101 to 106 dBa, which is similar to the results

Table 2.5. As-constructed PCC Mix Properties[a]

Property	Cell 71 RCA Mix	Cell 71 Surface PCC Mix	Cell 72 Low-cost Mix	Cell 72 Surface PCC Mix
Entrained air content, %	6.5	4.5	6.5	6.5
Unit weight, lb/ft^3	145.3	145.12	148.4	142.8
Flexural strength, psi (7 day)	527	606	468	790
Flexural strength, psi (14 day)	578	798	515	897
Flexural strength, psi (28 day)	665	891	548	816
Flexural strength, psi (90 day)	785	958	669	1011
Compressive strength, psi (7 day)	3,599	5,314	3,618	5,289
Compressive strength, psi (14 day)	3,890	5,628	4,071	5,260
Compressive strength, psi (28 day)	4,305	5,855	5,062	5,663
Compressive strength, psi (90 day)	5,663	7,598	6,695	7,388
Modulus of elasticity, psi (7 day)	4.30×10^6	4.76×10^6	4.73×10^6	4.45×10^6
Modulus of elasticity, psi (14 day)	4.87×10^6	4.82×10^6	NA	4.44×10^6
Modulus of elasticity, psi (28 day)	4.83×10^6	4.95×10^6	5.11×10^6	4.83×10^6
Modulus of elasticity, psi (90 day)	5.42×10^6	5.46×10^6	5.77×10^6	5.04×10^6
Poisson's ratio (7 day)	0.22	0.26	0.21	0.25
Poisson's ratio (14 day)	0.25	0.24	NA	0.22
Poisson's ratio (28 day)	0.25	0.26	0.23	0.21
Poisson's ratio (90 day)	0.30	0.28	0.23	0.23
Split tensile strength, psi (28 day)	337	392	343	390
°F	5.8×10^{-6}	5.6×10^{-6}	5.4×10^{-6}	5.6×10^{-6}
Air void analyzer (AVA) spacing factor, in.	0.0088	0.0125	0.0088	0.0082
AVA specific surface, in.$^{-1}$	580	639	628	710

Note: NA = not available.
[a] FHWA Mobile Concrete Laboratory.

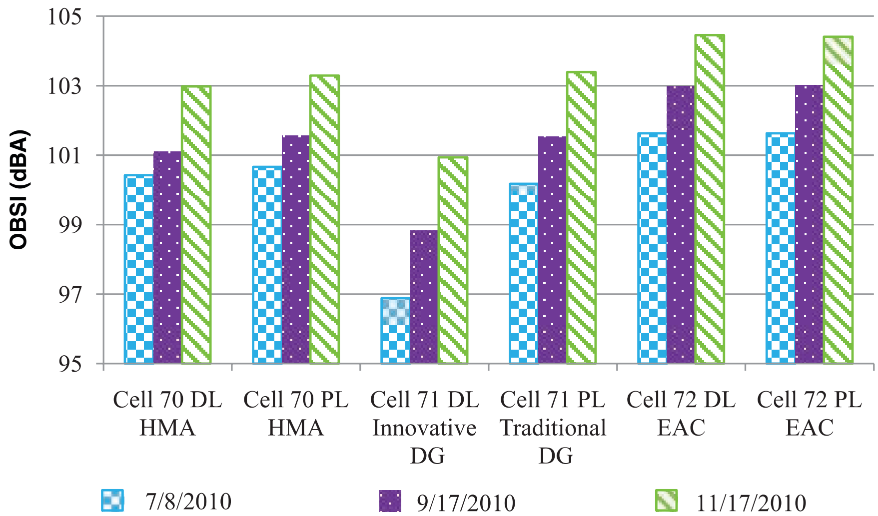
Figure 2.27. OBSI device.

obtained for Cell 72. The one-third octave sound-intensity spectrums used to calculate OBSI values are shown in Figure 2.29; great similarities exist in the spectrums of the various surfaces, except for that of the IG surface.

Field Survey Sections

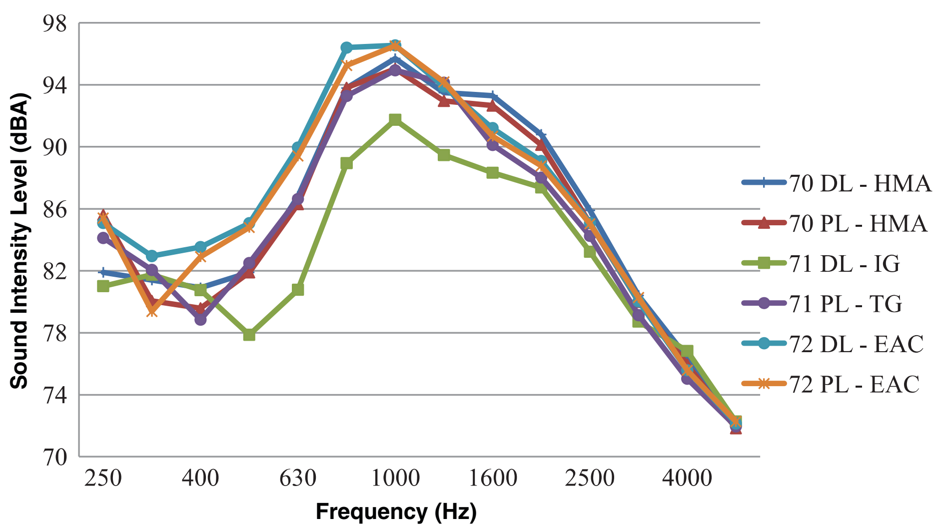
Introduction

In addition to the constructed test sites, additional field sites were identified to cover other types of PCC/PCC composite sections and to bring some long-term performance data into the analysis. An on-site condition survey was conducted, and detailed information regarding traffic, materials, and additional performance data was collected from the highway agencies. Table 2.6 provides a list of the PCC/PCC composite pavements that were included in the database, along with key design and construction information and photographs.



Source: Akkari and Izevbekhai 2011.

Figure 2.28. OBSI for the HMA, innovative diamond-ground, traditional diamond-ground, and EAC finishes.

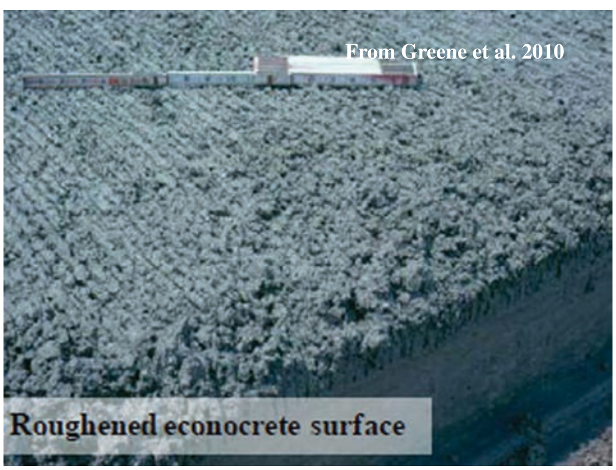


Source: Akkari and Izevbekhai 2011.

Figure 2.29. The one-third octave sound intensity spectrum for composite pavement surfaces on July 8, 2010.

Table 2.6. PCC/PCC Composite Pavement Field Sections

Composite Pavement Type	Location	Construction Year and Traffic	Comments
3-in. PCC over 9-in. econocrete (low-quality PCC with fc = 2,000, 1,250, or 750 psi) Nondowel, 15-ft skew joints	Fort Myers, Florida US-41 12 Test sections	1978 New construction 11,000 average daily traffic (ADT) (12% trucks)	1992: Rigid pavement performance report does not indicate performance problems. 2010: Greene et al. note good bond and performance over 30-year life.
3-in. PCC over 9-in. econocrete (low-quality PCC with fc = 2,000, 1,250, or 750 psi) Nondowel, 15-ft square joints	Fort Myers, Florida US-41 12 Test sections	1978 New construction 11,000 ADT (12% trucks)	1992: Rigid pavement performance report does not indicate performance problems. 2010: Greene et al. note good bond and performance over 30-year life.
3-in. PCC over 9-in. econocrete (low-quality PCC) with fc = 2,000, 1,250, or 750 psi 1-in. dowel, 20-ft square joints	Fort Myers, Florida US-41 12 Test sections	1978 New construction 11,000 ADT (12% trucks)	1992: Rigid pavement performance report does not indicate performance problems. 2010: Greene et al. note good bond and performance over 30-year life.



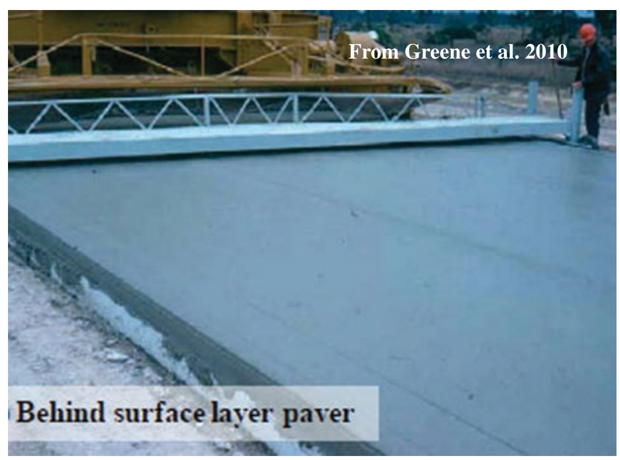


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Table 2.6. PCC/PCC Composite Pavement Field Sections (continued)

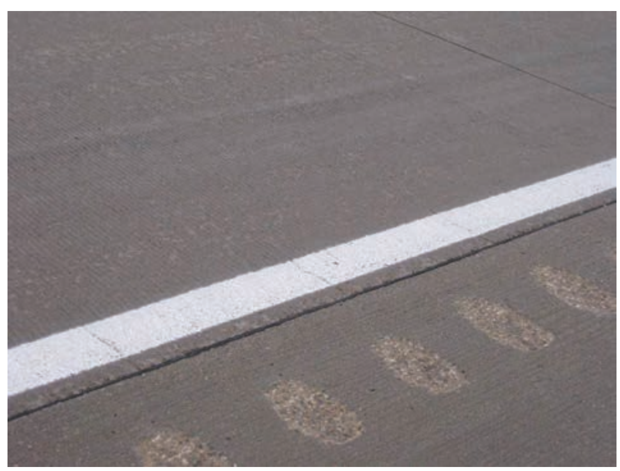
Composite Pavement Type	Location	Construction Year and Traffic	Comments
3-in. PCC over 7-in. PCC (including slightly reduced cement content) 20-ft joint spacing	Rock Rapids, Iowa US-75	1976 New construction Traffic unknown Estimated 6000 ADT (13% trucks)	2004: Cable and Frentress indicate pavement performing well to date.
3-in. alkali-silica reactivity (ASR)-susceptible PCC with lower water-cement ratio over 7-in. high absorption limestone PCC 15-ft joint spacing Section 12	Haven, Kansas K-96	1997 New construction 4,800 ADT (11% trucks)	1998: Wojakowski reports no initial performance concerns. 2011: Some evidence of ASR, but overall the pavement performing exceptionally well with no other distresses.
			
3-in. ASR-susceptible PCC with 20% pozzolan (Dura-Poz) replacement for ASR mitigation over 7-in. high-absorption limestone PCC 15-ft joint spacing Section 11	Haven, Kansas K-96	1997 New construction 4,800 ADT (11% trucks)	1998: Wojakowski reports no initial performance concerns. 2011: Some evidence of surface edge tearing/cracking (not fatigue) but overall the pavement performing exceptionally well with no other distresses. No evidence of ASR.
			

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Table 2.6. PCC/PCC Composite Pavement Field Sections (continued)

Composite Pavement Type	Location	Construction Year and Traffic	Comments
3-in. normal PCC over 7-in. PCC with 15% RAP 15-ft joint spacing Section 9	Haven, Kansas K-96	1997 New construction 4,800 ADT (11% trucks)	1998: Wojakowski reports no initial performance concerns. 2011: No major distresses. Only two locations of minor spalling that have been covered with HMA near the longitudinal joint.
			
2.5-in. PCC over 7.5-in. PCC 15-ft joint spacing	Detroit, Michigan I-75	1993 New construction Traffic unknown, estimated 120,000 ADT (13% trucks)	1996: Smiley details early noise problems and localized distress, otherwise performing well. 2010: No longitudinal or transverse fatigue cracking. Wheelpath exhibits severe spalling, some of which has been patched. Spalling appears to originate from deterioration of the lower PCC at the joints. EAC texture looks good.
			

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Table 2.6. PCC/PCC Composite Pavement Field Sections (continued)

Composite Pavement Type	Location	Construction Year and Traffic	Comments
1.5-in. PCC over 11.8-in. PCC 15-ft joint spacing Eight different surface textures	Salina, Kansas I-70	2008 New construction Opened December 2008 13,000 ADT (31% trucks)	2011: Surveyed. No structural problems observed. Pavements performing well. Some popouts of longitudinal groove texture and minor surface edge tearing/cracking (not fatigue) observed at some locations. Minor ASR observed.






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Table 2.6. PCC/PCC Composite Pavement Field Sections (continued)

Composite Pavement Type	Location	Construction Year and Traffic	Comments
3-in. PCC over 6-in. PCC (including RCA) 15-ft joint spacing	MnROAD, Albertville, Minnesota I-94	2010 New construction 25,000 ADT (14% trucks)	2011: No performance issues. Section performing well with no distresses. Excellent bond between PCC layers.
3-in. PCC over 6-in. PCC (low-cost) 15-ft joint spacing	MnROAD, Albertville, Minnesota I-94	2010 New construction 25,000 ADT (14% trucks)	2011: No performance issues. Section performing well with no distresses. Excellent bond between PCC layers.
			
2-in. PCC over 7.9-in. PCC (including RCA) 18-ft joint spacing	Traun, Austria A1	1994 New construction 110,000 ADT (13% trucks)	2010: R21 tour noted impressive structural performance and little wear in wheelpath. No distress existed after 16 years.
			
1.6-in. PCC over 8.3-in. PCC (including RCA) 18-ft joint spacing	Eugendorf, Austria A1	1993 New construction 56,000 ADT (13% trucks)	2010: R21 tour noted impressive structural performance (no cracks), yet signs of wear in wheelpath resulting from tire chains after 17 years.
			

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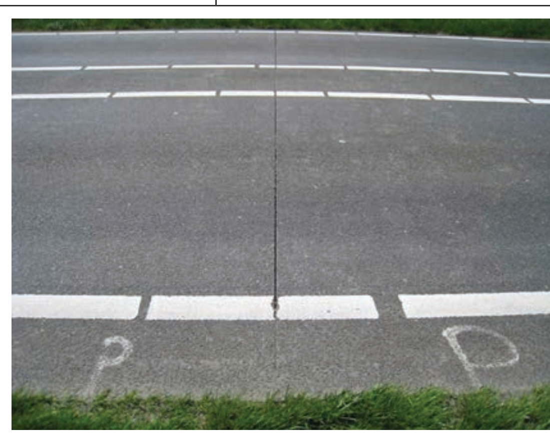
Table 2.6. PCC/PCC Composite Pavement Field Sections (continued)

Composite Pavement Type	Location	Construction Year and Traffic	Comments
3-in. PCC over 7.5-in. PCC 15-ft joint spacing	Montreal, Quebec, Canada A15	2009 New construction Traffic unknown, estimated 150,000 ADT (13% trucks)	2011: Ministry of Transportation of Quebec indicates sections performing well with no major distresses.
			
2.8-in. PCC over 7.5-in. PCC 18-ft joint spacing	Bavaria, Germany A93	1995 New construction 70,000 ADT (25% trucks)	2010: Test sections developed to investigate texturing (including EAC), overall impressive structural performance of sections given heavy traffic after 15 years.
			
3.5-in. PCC over 7-in. PCC 18-ft joint spacing	Veghel, the Netherlands N279	2000 New construction Traffic unknown, estimated 25,000 ADT (30% trucks)	2010: Test sections developed to investigate EAC texture depth. R21 tour noted good structural performance after 10 years.
			

Note: fc = 28-day compressive strength.

CHAPTER 3

PCC/PCC Analysis and Performance Modeling

Introduction

The analysis of laboratory and field data along with the development of models to better understand PCC/PCC design and behavior are detailed in this chapter. Although composite pavements generally are known to perform quite well, there has been little formal research into the performance of new PCC/PCC structures. The effort conducted through this SHRP 2 R21 project involved the review and modification of existing models in the *Mechanistic-Empirical Pavement Design Guide (MEPDG)* for bonded-PCC-over-JPCP projects and the use of lattice models for fracture to better understand the risk of debonding at the PCC/PCC interface. Table 3.1 introduces anticipated modes of failure for PCC/PCC, prediction models associated with these distresses (including *MEPDG*), changes made to the associated models if applicable, and overall comments on the model modifications and development in the project work.

Overall, the structural models in place for the *MEPDG* structural analysis of bonded PCC overlays were found to be sufficient for PCC/PCC composite pavements. The main difficulty in *MEPDG* modeling for PCC/PCC was in the EICM, used by versions of *MEPDG* before *MEPDG* v. 1.3000:R21, to determine thermal gradients through the pavement system and assign *k*-value to the subgrade. For this reason, the bulk of changes to *MEPDG* models were in the EICM, as detailed in the following sections and in Appendix R.

Analysis of Test Section Laboratory Data

As detailed in Chapter 2, three concrete mixes were used at MnROAD. These include concrete with RCA aggregate and 40% fly ash replacement (denoted as RCA concrete), concrete with Class A aggregate with 60% fly ash replacement (denoted as low-cost concrete or LC), and concrete with high-quality granite aggregate with specified gradation for the exposed aggregate texture, and 15% fly ash replacement (denoted as EAC concrete). The notations RCA, LC, and EAC are used in the following sections to represent these three concrete mixes.

Hardened PCC Properties

Compressive Strength

Compressive strength tests were performed on 4-in. and 6-in. EAC, RCA, and LC cylinders between 1 and 28 days. The concrete for these cylinders was taken from the trucks delivering the concrete to the construction site. Samples were cured on-site for 24 hours and then transported to laboratories. Figures 3.1, 3.2, and 3.3 show the compressive strength values of EAC, RCA, and LC samples, respectively.

The figures show there was significant variability in the compressive strengths reported for the EAC, RCA, and LC. The variability in compressive strengths of the EAC samples ranged between 1,000 and 2,000 psi. This strength variance was the largest of the three concrete types, although this concrete had the most consistent plastic properties during placement. The compressive strengths of the RCA and LC ranged within a few hundred psi until approximately 7 days and increased to approximately 2,000 psi by 28 days. Although the Minnesota DOT data points appear more variable than the FHWA data points, it should be noted that all of the FHWA data points are average compressive strengths of two samples. All samples exceeded the 4,000 psi requirement by 28 days.

A comparison of the average daily compressive strengths of all EAC, RCA, and LC samples is shown in Figure 3.4. The EAC samples had the highest average compressive strengths, whereas the RCA and LC compressive strengths were similar. The high compressive strengths for the EAC samples were thought to be attributable to the high quantity of cementitious material in the mixture compared with that in the RCA and LC mixes. The average 28-day compressive strengths for EAC, RCA, and LC field samples are within a few hundred

Table 3.1. Modes of Failure, MEPDG *Models Available, R21 Project Modifications for PCC/PCC Design*

Failure Mechanism	*MEPDG* Model Available	SHRP 2 R21 Model Modification	Comment
Bottom-up transverse cracking (fatigue)	Available, Limited calibration	Limited	Existing JPCP model in *MEPDG* found sufficient for PCC/PCC with limited modification (thickness and slab/base friction). Additional work included modification to Enhanced Integrated Climatic Model (EICM).
Top-down transverse cracking (fatigue)	Available, Limited calibration	Limited	Existing JPCP model in *MEPDG* found sufficient for PCC/PCC with limited modification (thickness and slab/base friction). Additional work included modification to EICM.
Longitudinal cracking (fatigue)	Not available	Not available	Models for longitudinal cracking in PCC/PCC are not needed because they are not typically observed in the field.
Joint faulting	Available, Limited calibration	None	Existing JPCP model in *MEPDG* found sufficient for PCC/PCC composite pavement.
Debonding between PCC layers	Not available	Not available	Used lattice models for fracture to investigate debonding and determined that debonding in newly constructed PCC/PCC is not a concern.

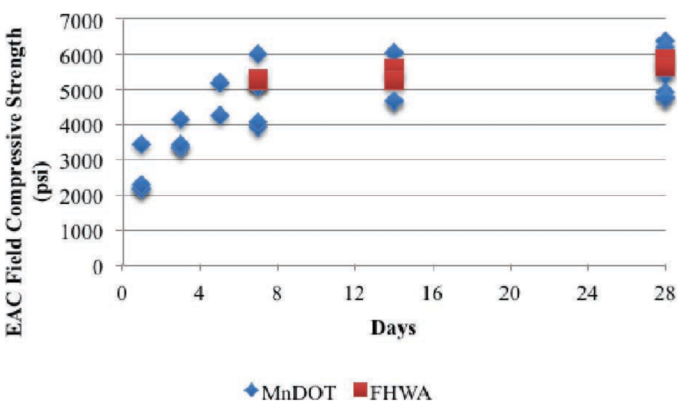


Figure 3.1. EAC compressive strength values for test sections constructed at MnROAD.

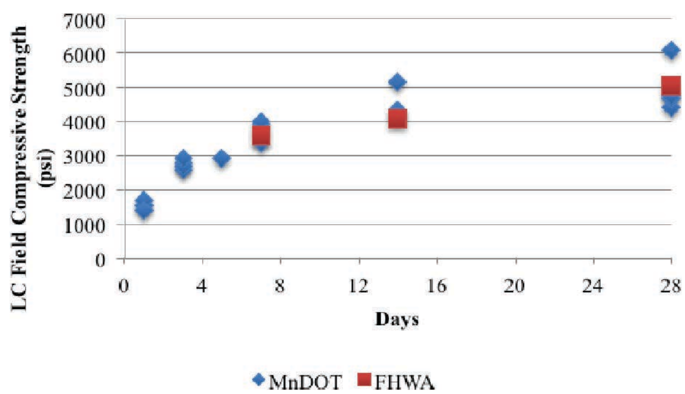


Figure 3.3. LC compressive strength values for test sections constructed at MnROAD.

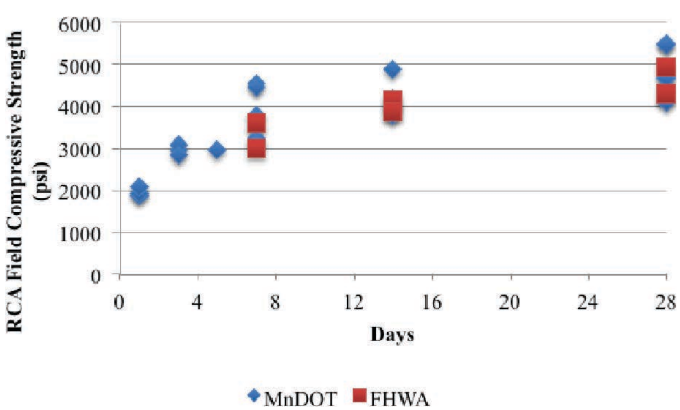


Figure 3.2. RCA compressive strength values for test sections constructed at MnROAD.

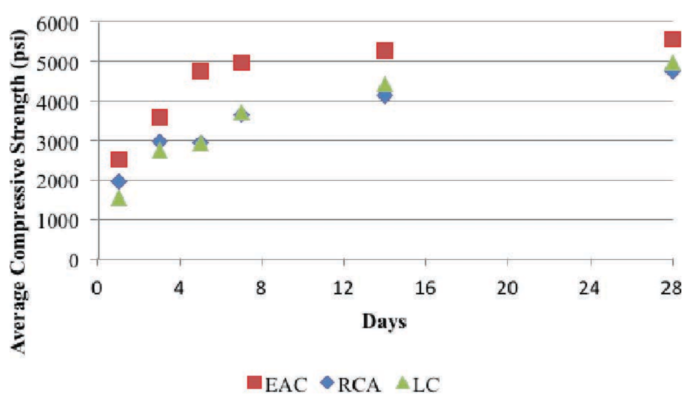


Figure 3.4. Average EAC, RCA, and LC compressive strengths for all cylinder samples for test sections constructed at MnROAD.

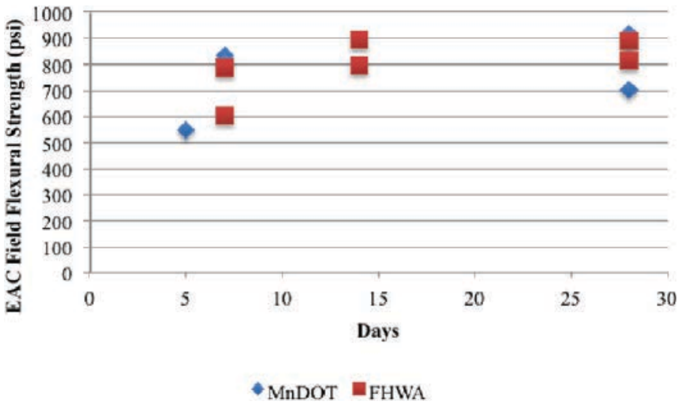


Figure 3.5. EAC flexural strength values for test sections constructed at MnROAD.

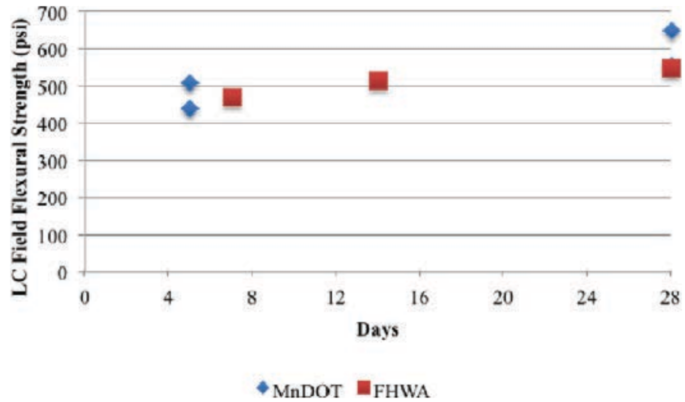


Figure 3.7. LC flexural strength values for test sections constructed at MnROAD.

psi of those determined for the laboratory mixed versions of these concretes by an independent, third-party testing agency.

Flexural Strength

Flexural strength tests were performed on 6- × 6- × 24-in. concrete beams at the ages of 5, 7, 14, and 28 days. The concrete for these beams was taken from the trucks delivering the concrete to the construction site. Samples were cured on-site for 24 hours and then transported to laboratories. Figures 3.5, 3.6, and 3.7 show the measured flexural strength values of EAC, RCA, and LC samples, respectively.

The figures indicate that a 75 to 200 psi variation in flexural strength was common for each concrete mixture between 5 and 28 days. It should be noted that all of the FHWA data points are the average of two beam specimens. Figure 3.8 shows the average daily flexural strengths of all EAC, RCA, and LC beam samples. The EAC samples had the highest average flexural strengths, whereas the RCA had a slightly higher (50 to 100 psi) flexural strength than did the LC. The average 28-day flexural strength for the field EAC sample was approximately 200 psi less than that of the laboratory-mixed sample, as determined by an independent testing agency. The RCA field sample flexural strength was almost 300 psi less than that of the RCA laboratory flexural strength. The difference in the flexural strengths of the LC field and laboratory samples was around 50 psi.

Poisson's Ratio, CTE, and Split Tensile Strength

In addition to compressive and flexural strengths, the FHWA Mobile Concrete Laboratory measured the modulus of elasticity, Poisson's ratio, split tensile strength, and coefficient of thermal expansion (CTE) properties of the EAC, RCA, and LC samples, as summarized in Table 2.5 in Chapter 2. The concrete for these samples was taken from the trucks delivering the concrete to the construction site. Samples were cured on-site for 24 hours and then transported to the FHWA laboratory.

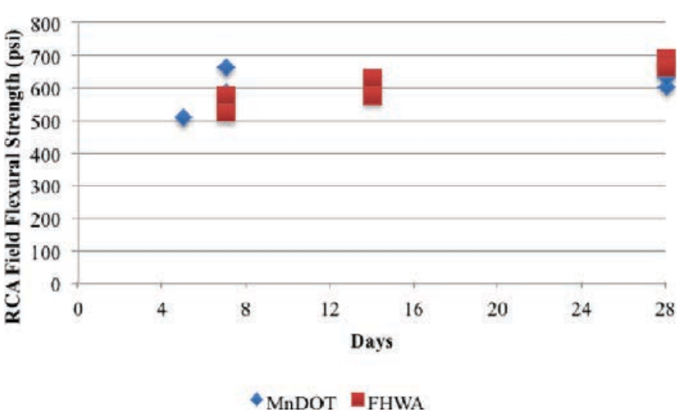


Figure 3.6. RCA flexural strength values for test sections constructed at MnROAD.

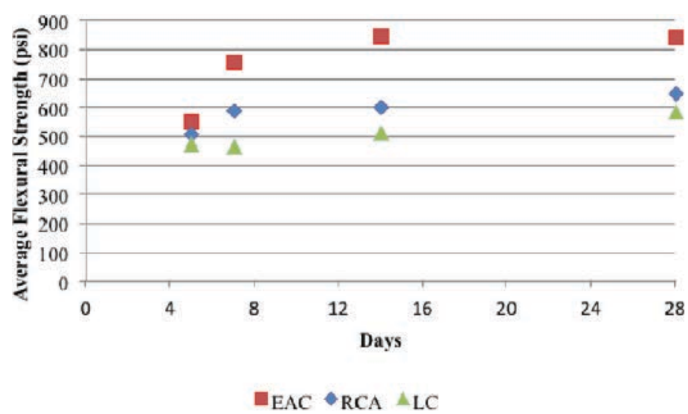


Figure 3.8. Average EAC, RCA, and LC flexural strengths for all beam samples for test sections constructed at MnROAD.

Table 2.5 showed that LC had the highest modulus of elasticity of the three concretes, which confirms that it is not a low-quality concrete. The aggregates could have been used for conventional concrete, but it is considered as low cost because fly ash was substituted for 60% of its cement. Generally, because the lower layer of composite pavements does not receive tire wear, lower quality aggregates that are less polish-resistant could be used in the layer. For the construction at MnROAD, the contractor did not have access to lower quality aggregates that were not polish resistant and instead had to use normal or higher quality aggregates.

Slant Shear Bond Strength

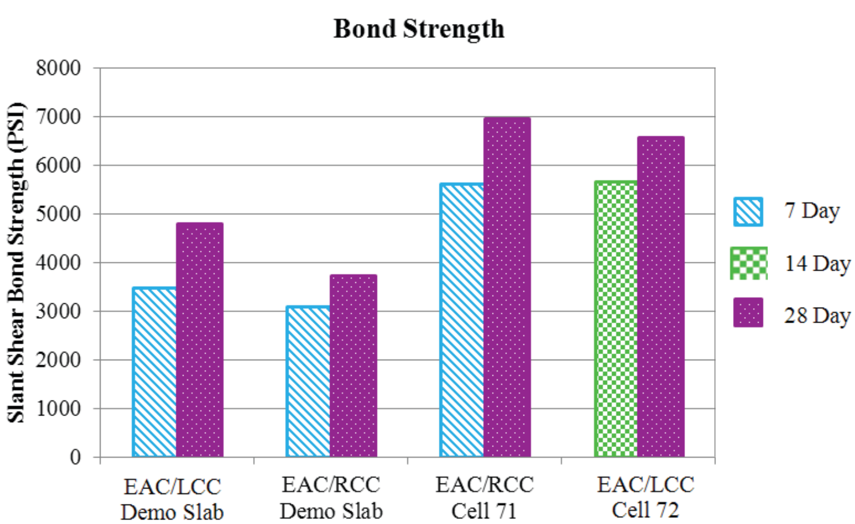
Minnesota DOT's Department of Materials and Road Research made concrete samples for ASTM C882: Standard Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete by Slant Shear. The specimens were made at the construction site by bonding two layers of concrete—EAC over LC or EAC over RCA—at an angled plane in a cylinder. An example of a slant shear specimen is shown in Figure 3.9. Figure 3.10 shows the measured bond strengths of either EAC over LC or EAC over RCA. Notice that LCC = LC and RCC = RCA (Akkari and Izevbekhai 2011).

Source: Akkari and Izevbekhai 2011.

Figure 3.9. Example of a slant shear specimen.

Freeze–Thaw Durability of MnROAD PCC/PCC Concretes

Despite a relaxation in constituent standards, it is still important for the lower layer pavement concrete to be freeze–thaw durable because, although the surface concrete layer will act as an insulator in reducing temperature extremes, it will not prevent the lower layer from experiencing regular freeze–thaw cycles. To test the freeze–thaw durability of the proposed concrete mixtures, the International Union of Testing and Research Laboratories for Materials and Structures (Paris) (RILEM) CIF concrete freeze–thaw standard (Setzer 1997; Setzer 2009) was implemented, instead of the AASHTO T161 and AASHTO T277 standards. RILEM is an international union of laboratories



Source: Akkari and Izevbekhai 2011.

Figure 3.10. Measured bond strengths of EAC/LC or EAC/RCA concretes.

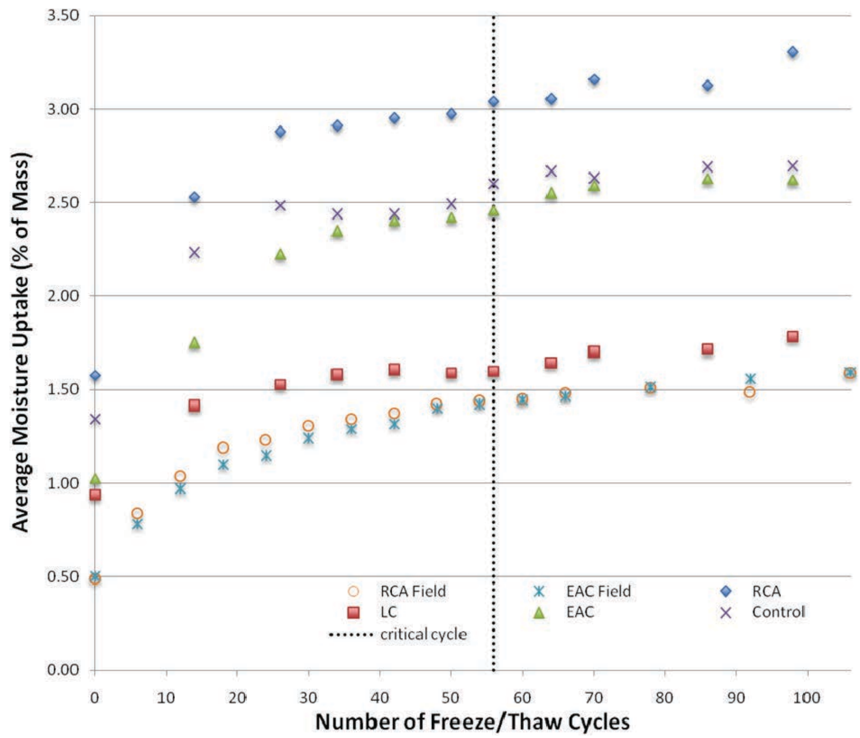


Figure 3.11. Field and laboratory samples' average moisture uptake as a percentage of sample mass during freeze–thaw cycles.

and experts in construction materials, systems, and structures. The CIF test evaluates the capillary suction, surface scaling resistance, and internal damage of concrete samples exposed to 3% by volume sodium chloride solution, whereas AASHTO T161 evaluates the internal damage of concrete submerged in water from rapid freeze–thaw cycles and AASHTO T277 evaluates the scaling resistance of concrete exposed to 3% sodium chloride solution and freeze–thaw cycles.

Capillary Suction

Figure 3.11 shows the mass of water or solution absorbed per unit sample area during the capillary suction period (cycle 0) and during freeze–thaw cycles. The test surfaces of the field RCA and EAC samples were submerged in pure water, and the test surfaces of the laboratory RCA, LC, EAC, and control samples were submerged in 3% sodium chloride solution. The dotted line on Figure 3.11 indicates the moisture uptake of the concrete samples at 56 freeze–thaw cycles.

Tables 3.2 and 3.3 show the mean and standard deviations (SD) of mass increase after the capillary suction phase and before freeze–thaw cycles began (cycle 0), at the critical freeze–thaw cycle (56 cycles), and at the end of the test, which was 106 cycles for the field samples and 98 cycles for the laboratory samples.

During the 7-day preconditioning period, the field RCA and EAC samples averaged a 0.49% mass gain because of capillary suction of pure water. Comparatively, the laboratory RCA and EAC samples averaged mass gains of 1.58% and 1.03%, respectively, attributable to capillary suction of 3% sodium chloride solution. After the preconditioning period and during the freeze–thaw cycles, the rate of mass increase was approximately equal for all field and laboratory samples. At approximately 30 freeze–thaw cycles, the rate of mass increase for all field and laboratory samples decreased to almost zero.

At first glance, it seems that the disparity in the initial uptake of test liquid between the field and laboratory samples is a function of the test liquid: pure water for the field samples and 3% sodium chloride solution for the lab samples. However, consideration of the compressive strengths (Table 3.4) and delineation of the moisture uptakes between the field and the laboratory samples (Figure 3.11) suggest that the disparity in moisture uptake has more to do with the samples' capillary porosity than with the test liquid. In other words, despite being made with identical mix designs and concrete constituents, the laboratory samples had a greater volume

Table 3.2. Field Sample Relative Increase in Mass Mean and Standard Deviation

Cycle	Field RCA Mean Mass Increase (%)	Field RCA SD Mass Increase (%)	Field EAC Mean Mass Increase (%)	Field EAC SD Mass Increase (%)
0	0.49	0.0012	0.50	0.0010
66	1.48	0.0016	1.47	0.0016
106	1.59	0.0024	1.60	0.0022

Table 3.3. Laboratory Sample Relative Increase in Mass Mean and Standard Deviation

Cycle	Laboratory RCA Mean Mass Increase (%)	RCA SD Mass Increase (%)	Laboratory Mean LC Mass Increase (%)	LC SD Mass Increase (%)	Laboratory Mean EAC Mass Increase (%)	EAC SD Mass Increase (%)	Laboratory Mean Control Mass Increase (%)	Control SD Mass Increase (%)
0	1.58	0.17	0.94	0.07	1.03	0.16	1.34	NA
64	3.06	0.20	1.65	0.11	2.55	0.06	2.67	NA
98	3.31	0.18	1.78	0.06	2.63	0.07	2.70	NA

Note: NA = not available.

Table 3.4. Plastic and Hardened Properties of Field and Laboratory Samples

	Field RCA	Field EAC	Laboratory RCA	Laboratory LC	Laboratory EAC	Laboratory Control
Plastic Properties						
Slump (in.)	1.75	2.25	4.5	2	1	4.75
Air (%)	6.5	4.5	7.5	6.2	4.7	8
Unit weight (lb/ft^3)	145	145	140	140	151	141
Hardened Properties						
7-day compressive strength (psi)	3,599	5,314	1,738	2,950	3,836	1,769
28-day compressive strength (psi)	4,305	5,855	2,365	4,185	4,404	2,294

and average capillary pore size than did the field samples, as evidenced by the lower 7- and 28-day strengths. The average RCA laboratory sample's 28-day compressive strength was 50% that of the average RCA field sample. Similarly, the average EAC laboratory sample's 28-day compressive strength was 75% that of the average EAC field sample.

The reason for the strength, and thus capillary suction, disparity between the field and laboratory samples remains unknown. Two production disparities between the field and laboratory samples may have contributed to the disparity in moisture uptake. One production disparity was that the concrete used to make the field samples was batched at a ready-mix concrete plant, and the concrete used to make the laboratory samples was batched in a 1-yd^3 mixer. A second production disparity was that the field samples were consolidated on a vibrating table and the laboratory samples were consolidated by rodding. Note that the laboratory samples were prepared twice because the first samples failed to gain the expected strength. The compressive strength of the second batch of laboratory samples also did not gain the expected strength.

In a RILEM CIF test study of high-performance concrete mixtures with water-to-cement ratios of approximately 0.30 and average cement contents of 742 lb/yd^3, the moisture uptake was between 1.5% and 3.0% of the samples' mass (Setzer 1997). In another study that used the CIF test on concrete samples with 0.5 w/c ratios and with between 50% and 100% coarse RCA substitution, the moisture uptake was between 3% and 4.5% (Setzer 1997). For the MnROAD samples, as the moisture uptake ranged from 0.5% to 3%, it is reasonable to conclude that the water uptake by all of the field and laboratory samples was within an acceptable range.

Deicing Salt Scaling

Figure 3.12 shows the scaled test surfaces of an EAC sample after 56 freeze–thaw cycle.

Figure 3.12. Scaled test surfaces of an EAC sample after 56 freeze–thaw cycles.

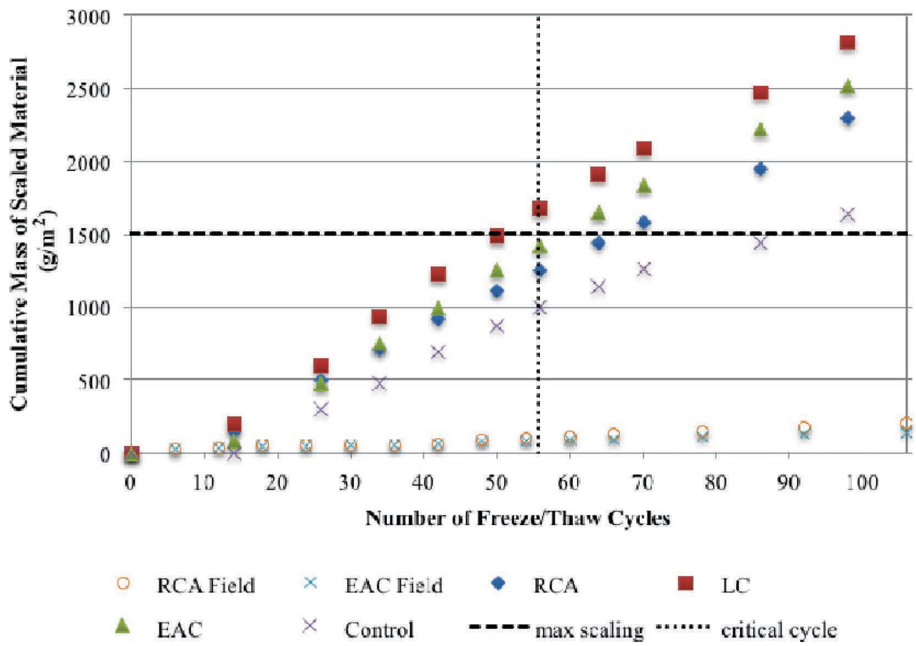


Figure 3.13. Cumulative mass of scaled material per unit area subjected to freeze–thaw cycles.

Figure 3.13 shows the cumulative mass of scaled material per unit area for both field and laboratory concrete samples as the number of freeze–thaw cycles increases. It is important to reiterate here that the test surfaces of the field RCA and EAC samples were submerged in pure water and the test surfaces of the laboratory RCA, LC, EAC, and Control samples were submerged in 3% sodium chloride solution. The horizontal dashed line on Figure 3.13 indicates the maximum recommended scaling mass after 56 freeze–thaw cycles. The vertical dotted line indicates mass of scaled material at the 56th freeze–thaw cycle. Tables 3.5 and 3.6 show the mean value and the standard deviation of scaled material at the 56th freeze–thaw cycle and after termination of the CIF test for the field and laboratory concrete samples, respectively.

As shown by Figure 3.13, all field and laboratory samples, except the laboratory LC sample, averaged below 1,500 g/m^2 of scaled material at 56 freeze–thaw cycles. The laboratory LC sample average was 1,913 g/m^2, which is within 2 SD of the limit and thus can be cautiously accepted as adequately able to resist surface scaling.

The field RCA and EAC samples scaled significantly less than the laboratory RCA and EAC samples. This disparity was expected and is primarily attributable to the difference in test liquid (pure water for field samples and 3% sodium chloride solution for laboratory samples). The disparity was not caused by capillary suction (Setzer 2009), nor was it likely caused by a lack of entrained air in the surface concrete. The entrained air measurements of the plastic concretes were sufficient (between 4.5% and 8%) for all field and laboratory

Table 3.5. Mean and Standard Deviation of Field Sample Scaled Material per Surface Area

Cycle	Field RCA Mean Scaled Material (g/m²)	Field RCA SD (g/m²)	Field EAC Mean Scaled Material (g/m²)	Field EAC SD (g/m²)
66	126	19	95	19
106	204	30	143	25

Table 3.6. Mean and Standard Deviation of Laboratory Sample Scaled Material per Surface Area

Cycle	Laboratory RCA Mean Scaled Material (g/m²)	RCA SD (g/m²)	Laboratory Mean LC Scaled Material (g/m²)	LC SD (g/m²)	Laboratory Mean EAC Scaled Material (g/m²)	EAC SD (g/m²)	Laboratory Mean Control Scaled Material (g/m²)	Control SD (g/m²)
64	1,438	71	1,913	225	1,651	342	1,139	NA
98	2,296	251	2,816	270	2,520	348	1,642	NA

Note: NA = not available.

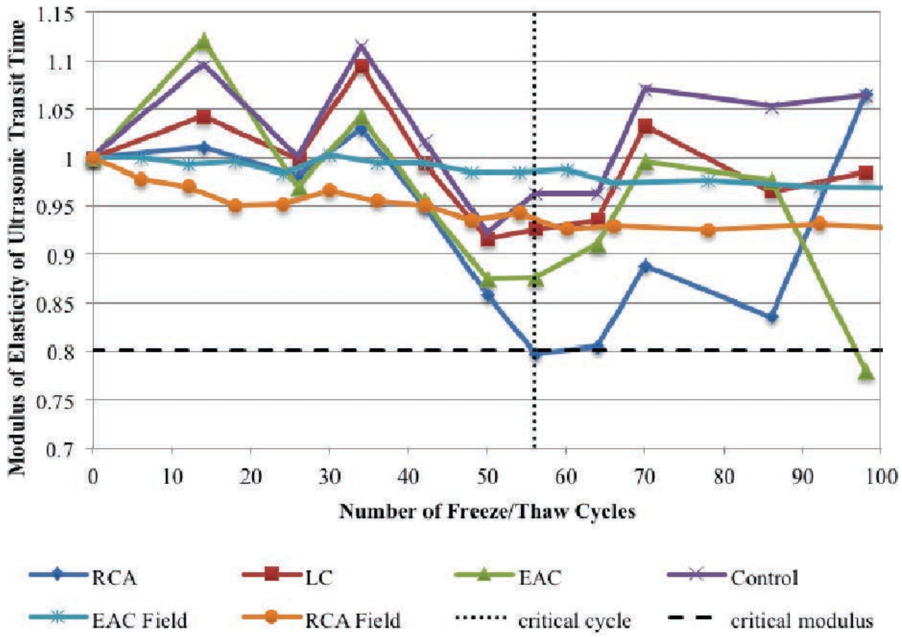


Figure 3.14. Modulus of elasticity of ultrasonic pulse velocity.

samples. For the PCC/PCC composite pavements constructed at MnROAD, the EAC was the only concrete that would have been subjected to deicing salt, and it proved to be an adequate concrete mixture for resisting deicing salt scaling.

Internal Damage

Figure 3.14 shows the relative modulus of elasticity of ultrasonic transit time (relative modulus) for field and laboratory samples. The vertical dotted line indicates the relative modulus at 56 freeze–thaw cycles. The horizontal dashed line indicates the modulus below which the sample is considered distressed. Tables 3.7 and 3.8 show the mean and standard deviation relative modulus values at 54 or 56, and either 106 or 98 freeze–thaw cycles for the field and laboratory CIF test samples, respectively.

After 56 freeze–thaw cycles, the relative moduli of the field RCA and EAC samples were approximately 94% and 98%, respectively, indicating very little internal damage. As confirmed by the low uptake of moisture, the capillary porosity of the field samples was low and the capillaries likely were disconnected, which are indicators of a concrete paste that is resistant to freeze–thaw damage. In addition, these samples were adequately air entrained. CIF experiments by others have shown that internal damage is not a function of test liquid (sodium chloride solution), surface texture, or carbonation but rather depends mostly on the sample's water-to-cement

Table 3.7. Average Relative Modulus of Elasticity of Ultrasonic Transit Time for Field RCA and EAC Samples

Cycle	Field RCA Average $R_{u,n}$ (%)	Field RCA SD (%)	Field EAC Average $R_{u,n}$ (%)	Field EAC SD (%)
0	100	0	100	0
54	94	4	98	2
106	93	3	97	2

Note: The values shown represent an average from five samples.
$R_{u,n}$ = relative modulus of elasticity of ultrasonic transit time.

Table 3.8. Average Relative Modulus of Elasticity of Ultrasonic Transit Time for Laboratory RCA, LC, EAC, and Control Samples

Cycle	Laboratory Average RCA $R_{u,n}$	RCA SD	Laboratory Average LC $R_{u,n}$	LC SD	Laboratory Average EAC $R_{u,n}$	EAC SD	Laboratory Average Control $R_{u,n}$	Control SD
0	100	0	100	0	100	0	100	NA
56	80	17	93	2	88	3	96	NA
98	84	3	98	12	78	26	106	NA

Note: $R_{u,n}$ = relative modulus of elasticity of ultrasonic transit time; NA = not available.

ratio and the quantity and spacing of entrained air bubbles (Setzer 1997).

As indicated by the erratic peaks and valleys of the laboratory sample modulus measurements, the data did not follow a neat downward line, and sometimes the modulus values exceeded 100%. Despite the fluctuations, the data trended toward a decreasing modulus as the number of freeze–thaw cycles increased. The relative moduli of the samples also remained at or above the 80% after 56 cycles, which indicates that the samples adequately resisted internal damage caused by frost action. As stated, it is the strength of the concrete matrix, rather than the test liquid, that influences the resistance of a sample to internal damage.

Compared with the decrease in relative modulus of other concrete samples studied with the RILEM CIF procedure, the decreases in relative moduli of all of the field and laboratory samples were relatively small (Setzer 1997; Setzer 2009). The difference between the concretes used for the MnROAD construction and those in the referenced studies is that all of the MnROAD construction samples were air entrained. One study evaluated a 0.5 water-to-cement ratio concrete without entrained air, and the relative modulus values decreased below 80% after between 10 and 30 freeze–thaw cycles (Setzer 1997). Another study showed that even in a CIF experiment that evaluated 0.3 water-to-cement ratio and high cement content samples, only those concrete mixtures that contained entrained air were able to adequately resist internal cracking according to the RILEM CIF standard.

For the SHRP 2 R21 composite pavement project, the lack of internal damage in both the RCA and LC mixtures after 56 freeze–thaw cycles indicated that these mixtures are suitable for use in long-life concrete pavements, despite containing RCA or having a 60% cement replacement with fly ash, respectively. It was expected that the EAC samples would experience minimal internal damage caused by frost action because of its superior granite aggregates and high cement content paste.

Analysis of Field Data at MnROAD

The two PCC/PCC test sections constructed at the MnROAD facilities were labeled Cell 71 and Cell 72. In addition, as detailed in Volume 1, an HMA/PCC section was also constructed and labeled Cell 70. Cell 71 had the same lower lift PCC mix as Cell 70 with RCA in the PCC mix, whereas Cell 72 did not have RCA in PCC mix but had higher fly ash content (60% replacement versus 40% replacement). Results from Cell 70 are included in some of the figures and discussions in this section.

PCC Slab Temperature Profiles

The simplest way to characterize the temperature distribution in the slab is by assuming a linear distribution for the temperature throughout the depth of the slab. The linear temperature gradient (LTG) is calculated as the temperature difference between the top and bottom of the slab taken over the distance between the two. However, several field studies have shown that the distribution of temperature throughout the slab depth is primarily nonlinear (Armaghani et al. 1987; Yu et al. 1998). To account for the nonlinearity of the temperature distribution in the slab, the equivalent temperature gradient concept was developed (Thomlinson 1940; Choubane and Tia 1992; Mohamed and Hansen 1997). The equivalent linear temperature gradient (ELTG) is a linear gradient that would produce the same curvature in the slab as the original nonlinear temperature gradient. The ELTG concept was later generalized for nonuniform, multilayered slabs (Khazanovich 1994; Ioannides and Khazanovich 1998). The latter method, in which the ELTG is established for an effective slab, with a thickness and stiffness equivalent to that of a composite multilayer section, is used in this analysis.

Effect of Location on Slab (Midslab versus Edge versus Corner)

An assessment was made for the LTGs between locations within each slab. It is possible that the magnitude of temperature gradients in a slab can differ between locations because of the different boundary conditions at midslab, edge, and corner. To assess the variation in the temperature gradients that develop in the slab with location, the LTG in the composite of the upper and lower layers for Cells 71 and 72 was calculated over a typical day. Temperature data collected on July 19, 2010, was used for this analysis. The ambient temperature and solar radiation for this day is presented in Figure 3.15. The maximum ambient temperature of 80°F observed on July 19, 2010, is not representative of the extreme conditions observed at MnROAD during the summer. However, because the objective of this analysis is only to determine the variation in temperature gradients with

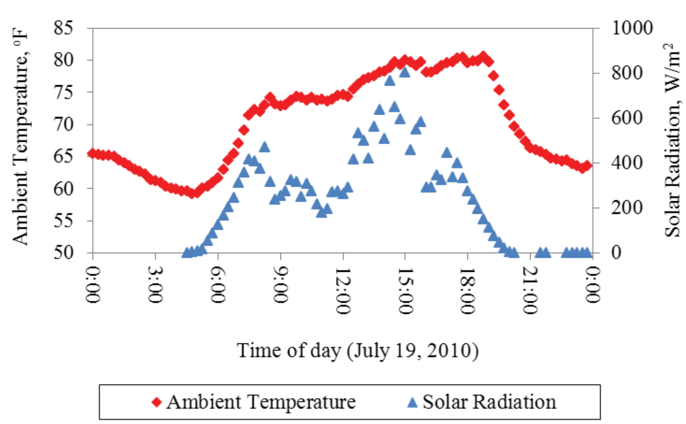


Figure 3.15. Ambient temperature and solar radiation for July 19, 2010, measured at the project site.

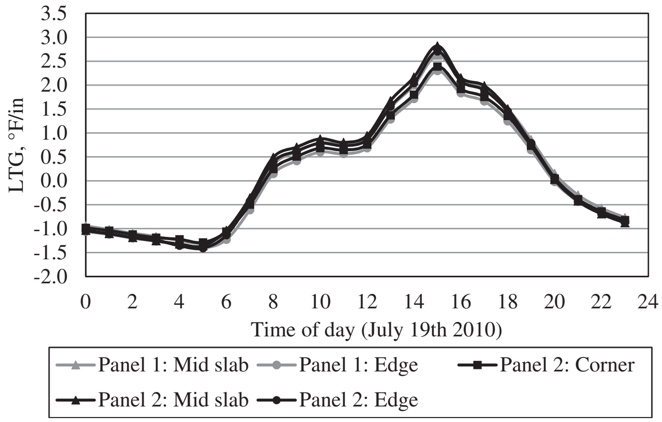


Figure 3.16. LTGs for various slab locations in Cell 71 on July 19, 2010.

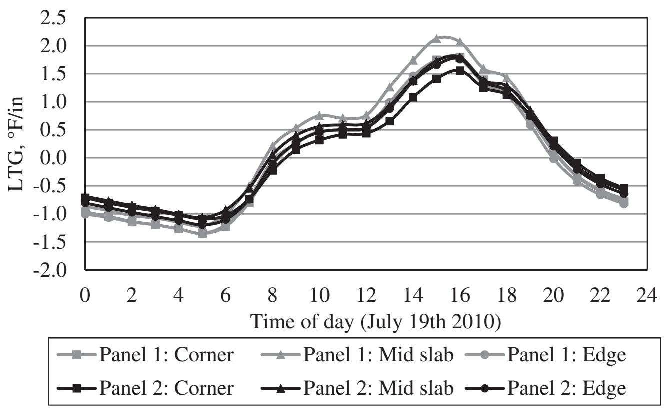


Figure 3.17. LTG at various locations in the slabs in Cell 72 on July 19, 2010.

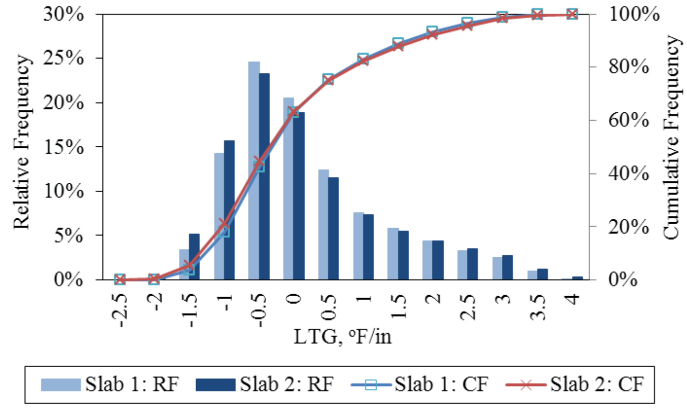


Figure 3.18. LTG distribution for Slabs 1 and 2 in Cell 71.

location over a typical day, the LTGs computed for this date from the temperature data at all the available thermocouple locations were used.

The LTGs calculated at different locations within Cells 71 and 72 are shown in Figures 3.16 and 3.17. The midslab locations in both Slabs 1 and 2 for both cells experience higher magnitude of LTG compared than do other locations. The thermocouple depths are summarized in Table 3.9 for both cells. The depths available for the sensors, shown in Table 3.9, indicate that the sensors were installed at similar depths for each of the locations.

Based on the comparative analysis of the variation in the LTG for each location within the slab, it can be concluded that the largest positive and negative LTGs occur at the midslab location. Therefore, the LTG corresponding to this location is used for subsequent analyses. The variation between LTGs for different locations is not significant and is not expected to affect the analysis.

Variation in LTG between Cell 71 and Cell 72

Figures 3.18 and 3.19 show histograms of the LTG distribution in the slabs for Cells 71 and 72, respectively. The figures show that the variation in LTG between the slabs within the same cell is negligible. To compare the LTGs between Cells 71 and 72, LTGs only for Slab 2 are compared in Table 3.10. The temperature data for Slab 1 in Cell 72 was not reliable for a considerable period, so the data for Slab 2 in both the cells is considered.

Based on the data in Table 3.10, the variation between the two cells is apparent. Cell 71 experiences larger temperature

Table 3.9. Approximate Depths of the Top and Bottom Thermocouples for Cell 71 and Cell 72

	Thermocouple Depths, in.					
	Cells 71 and 72: Slab 1			Cells 71 and 72: Slab 2		
Location	Corner	Midslab	Edge	Corner	Midslab	Edge
Top	0.75	0.75	0.75	0.75	0.75	0.75
Bottom	9.25	9.25	9.25	9.25	9.25	9.25

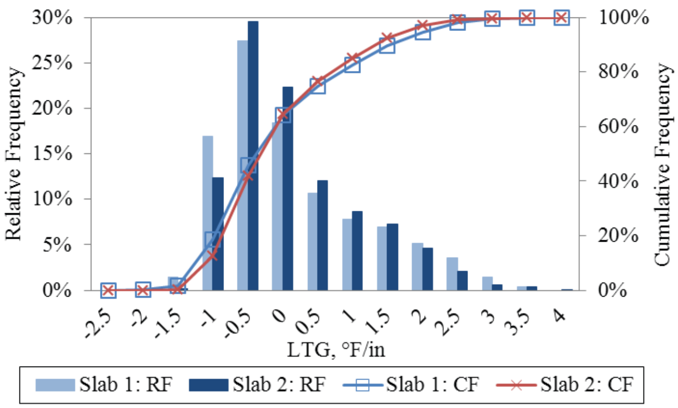


Figure 3.19. LTG distribution for Slabs 1 and 2 in Cell 72.

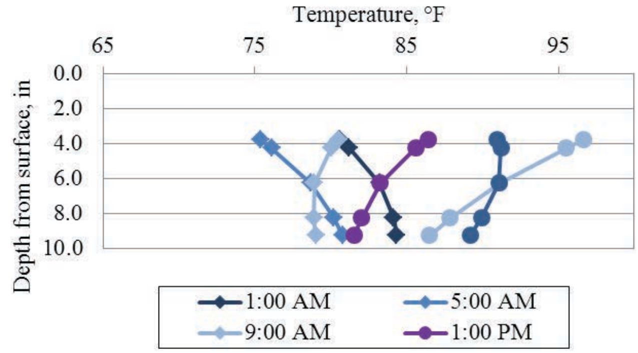


Figure 3.20. Temperature distribution in the PCC for Cell 70.

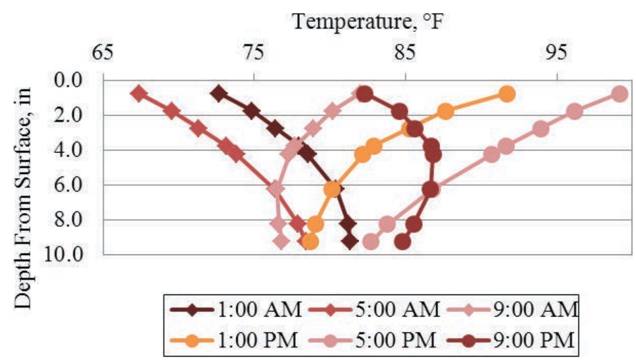


Figure 3.21. Temperature distribution in the composite section for Cell 71.

gradients more frequently than does Cell 72. As discussed in Appendix H, the weighted average temperature (WAT) for Cell 71 was higher than that for Cell 72. This finding could be the result of the limited data collected or the difference in thermal properties (such as heat capacity and thermal conductivity) between the two cells. This analysis should be performed again once a larger database of data is available.

Comparison of Exposed Concrete (Cell 71) to HMA-Covered Concrete (Cell 70)

Figures 3.20 and 3.21 illustrate the temperature distributions in the slabs throughout a single summer day for Cells 70 and 71, respectively. The figures show that the deviation of the temperature profiles from a linear gradient is more pronounced in Cell 71, which is not covered by the HMA, as is the case for Cell 70. The difference in nonlinearity between Cell 70 and Cell 71 suggests that LTG alone is not sufficient for characterizing the actual variation of the temperature throughout the depth. The nonlinearity of the temperature distribution should also be considered, which is done using ELTG.

Table 3.10. Statistics for LTG Over the Analysis Period, Comparing Cells 71 and 72

	LTG, °F/in.	
	Cell 71	Cell 72
Average	0.03	0.01
Maximum	4.14	2.93
Minimum	–2.22	–1.91
SD	1.26	0.93
Median	–0.37	–0.25
Number of observations per slab = 20,814		

The ELTGs were estimated for Cells 70 and 71, using Equations 3.1 to 3.3.

$$\Delta T_{\text{eff}} = \frac{12}{h_{\text{eff}}^2} \times \left\{ \int_{-x}^{h_{\text{top}}-x} \left(T(z) - T_0\right) z\,dz + \frac{\alpha_{\text{bot}} E_{\text{bot}}}{\alpha_{\text{top}} E_{\text{top}}} \int_{h_{\text{top}}-x}^{h_{\text{top}}+h_{\text{bot}}-x} \left(T(z) - T_0\right) z\,dz \right\} \quad (3.1)$$

where

- ΔT_{eff} = difference between temperatures at the top and bottom surfaces of the effective slab,
- $T(z)$ = temperature distributions through the PCC concrete,
- T_0 = zero-stress temperature,
- z = vertical coordinate measured downward from the neutral axis of the composite pavement,
- h_{top} = thickness of the upper layer,
- h_{bot} = thickness of the lower layer,
- E_{top} = elastic modulus of the upper layer,

E_{bot} = elastic modulus of the lower layer,
α_{top} = coefficient of thermal expansion of the upper layer,
α_{bot} = coefficient of thermal expansion of the lower layer, and
h_{eff} = effective thickness of the pavement, which can be determined from Equation 3.2:

$$h_{eff} = \sqrt[3]{h_{top}^3 + \frac{E_{bot}}{E_{top}} h_{bot}^3 + 12\left(h_{top}\left(x - \frac{h_{top}}{2}\right)^2 + \frac{E_{bot}}{E_{top}}\left(h_{top} + \frac{h_{bot}}{2} - x\right)^2 h_{bot} \right)} \qquad (3.2)$$

x = distance between the neutral plane and the top surface of the upper layer, which can be determined from Equation 3.3:

$$x = \frac{\frac{h_{top}^2}{2} + \frac{E_{bot}}{E_{top}} h_{bot}\left(h_{top} + \frac{h_{bot}}{2}\right)}{h_{top} + \frac{E_{bot}}{E_{top}} h_{bot}} \qquad (3.3)$$

Figures 3.22 and 3.23 show the variation in the LTGs and ELTGs between Cells 70 and 71. The summary statistics for the ELTGs that developed in both cells is presented in Tables 3.11 and 3.12. Cell 70 shows a higher frequency of occurrence of ELTGs close to zero than does Cell 71. As seen in Figures 3.22 and 3.23, the shapes of the frequency distribution curves of LTGs and ELTGs for Cell 70 are quite similar, whereas a considerable difference in shape is observed in Cell 71. Unlike the LTGs, the ELTGs for Cell 71 are evenly distributed over a broader range. To investigate the significance of variation in LTGs and ELTGs within a cell and the variation of each of these gradients between the cells, a paired *t*-test for two sample means was performed. Results from the *t*-test are given in Tables 3.11 and 3.12. The variation between the LTGs and ELTGs in Cell 70 is not significant, whereas it is quite significant for Cell 71. This indicates that the gradients that develop in the HMA/PCC pavements tend to be much more linear than in the PCC/PCC pavements, as is supported by the temperature profiles provided in Figure 3.21.

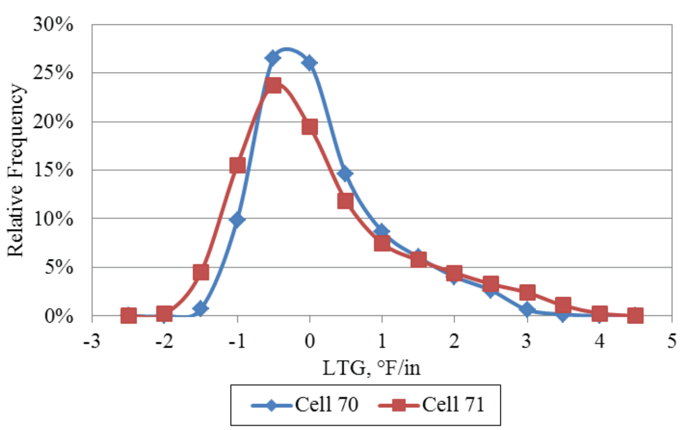


Figure 3.22. Comparison of relative frequencies for LTGs in Cells 70 and 71.

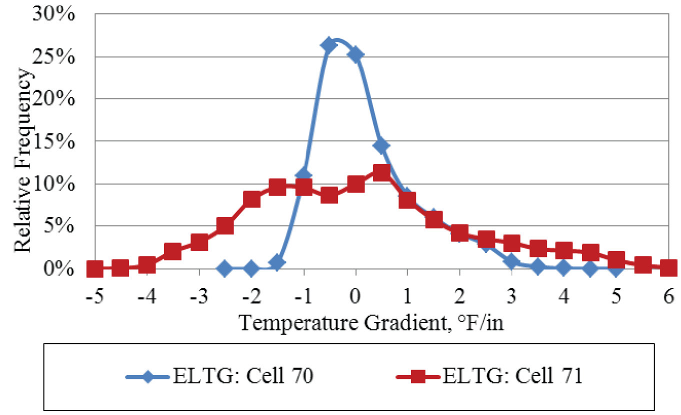


Figure 3.23. Comparison of relative frequencies for ELTGs in Cells 70 and 71.

The results of the paired *t*-test for the ELTGs between Cells 70 and 71 also conclude the difference in the ELTGs is significant at a 95% confidence level. The ELTGs in Cell 71 (PCC/PCC) are much higher over a larger period of time than the ELTGs for Cell 70 (HMA/PCC). It can clearly be seen in Figure 3.23 that the magnitude of the temperature gradients, as well as the frequency at which these higher gradients develop, is significantly greater for a PCC/PCC pavement than for an HMA/PCC pavement.

PCC Slab Moisture Profiles

In addition to temperature variation within the slab, the moisture variation through the depth is also important. This is because the moisture variation through the depth of the slab

Table 3.11. Comparing the LTG and ELTG Within Each Cell for Cells 70 and 71

	Cell 70		Cell 71	
	LTG	ELTG	LTG	ELTG
Average, °F/in.	–0.07	–0.07	–0.08	–0.20
Variance, °F/in.	0.79	0.829	1.29	3.85
Observations	28,649			
Hypothesized mean difference	0			
Degrees of freedom	28,648			
t-Statistics	1.43		14.64	
p-value	0.15		0	
t Critical two-tail	1.96		1.96	

Table 3.12. Comparing the ELTG Statistics for Cells 70 and 71

	ELTG, °F/in.	
	Cell 70	**Cell 71**
Average °F/in.	–0.070	–0.195
Maximum	3.540	5.951
Minimum	–1.864	–4.699
SD	0.911	1.961
Median	–0.292	–0.336
Paired *t*-test results		
Variance, °F/in.	0.829	3.845
Observations	28,649	
Hypothesized mean difference	0	
Degrees of freedom	28,648	
t-Statistics	15.11	
p-value	0	
t Critical Two-tail	1.96	

produces an upward warping of the slab (because the bottom of the slab is almost always saturated and the top typically goes through wet-dry cycles). To capture the variation in the moisture content through the depth of the concrete layers, relative humidity was measured at different depths and locations.

A total of 72 humidity sensors were installed in the two PCC/PCC cells. Ambient relative humidity, temperature, and solar radiation also were measured with the weather station on-site. The variation in the daily average ambient relative humidity at the project location, during the analysis period (May 2010 to March 2011), is shown in Figure 3.24. The range of the average daily ambient relative humidity is between 50% and 100%. The variation in the relative humidity in the concrete at very shallow depths might show a similar variation to the ambient relative humidity with time. However, it is more likely that the variation in the relative humidity in the concrete follows seasonal trends because of a slow diffusion of water through concrete. To assess the seasonal trends in the ambient relative humidity, the average ambient relative humidity for each month of the analysis period was calculated as shown in Figure 3.25. The figure shows that the ambient relative humidity increases in the winter, with the highest values in November and December and the lowest value in September.

As detailed in Appendix H, the relative humidity data collected by some sensors were not of an acceptable quality. Therefore, an initial quality check was performed on the data to select sensor locations with a suitable data set. On this basis, the relative humidity data for the midslab and corner of Slab 2 for Cell 71 and edge of Slabs 1 and 2 for Cell 72 were selected for analysis.

Figures 3.26 through 3.29 present the variation in relative humidity with depth for the locations in Cells 71 and 72. The common observation for all four figures is that for the first 2 to 3 weeks after paving, there is a significant drop in relative humidity that is uniform for all sensors throughout the depth of the concrete. This may be attributed to hydration of the concrete. It can be seen that in late November, the variation in the relative humidity increases suddenly, and continues throughout the winter and early spring. The increase in relative humidity during the winter months is the result of a decrease in the temperature and not a change in the moisture content. Unfortunately, the moisture content in the concrete cannot be measured directly and must be estimated based on the measured relative humidity. Therefore, when interpreting these data, it is important to remember that the relative humidity will increase when the temperature decreases, even when

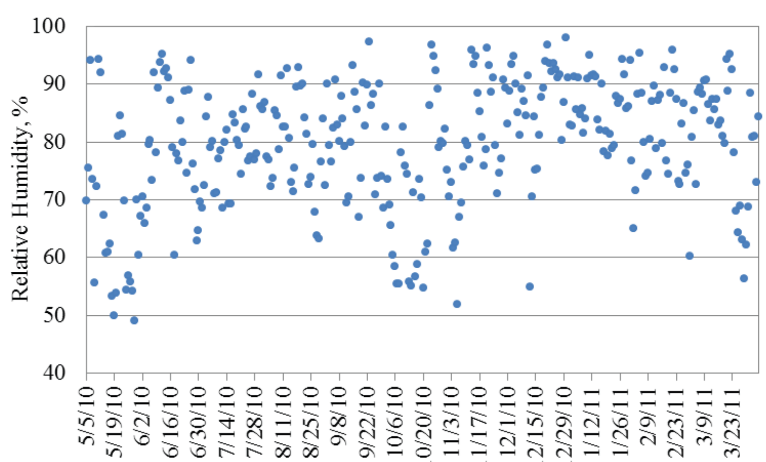


Figure 3.24. Daily average ambient relative humidity at MnROAD.

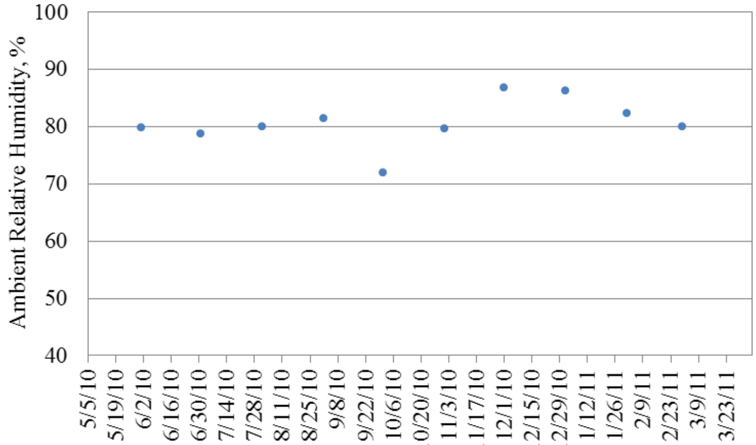


Figure 3.25. Monthly average ambient relative humidity at MnROAD.

the moisture content remains constant. For this reason, it is important to make comparisons between the concrete relative humidity measurements made at the same time of year over a period of 5 or 6 years. It typically takes about 5 to 7 years before all of the irrecoverable drying shrinkage develops at the surface of the slab. Unfortunately, the complete interpretation of the moisture data is not possible because less than a year's data were available at the time the analysis for this report was performed.

The most reasonable measurements of the relative humidity in the concrete were obtained from Cell 72. These measurements are shown in Figures 3.28 and 3.29. The relative humidity measured at the top of Slabs 1 and 2 for this cell show the largest daily variations. This is expected because the sensors close to the surface are most heavily influenced by the ambient conditions. However, the two top sensors in Cell 71 do not show the same behavior over the analysis period. The variation in the relative humidity between the lower sensors is small and remains constant over the year after initial drying of the concrete. It is possible that this variability is the result of variations in the sensor depth because the exact as-built depths are unknown. A better interpretation will be possible as more data become available over time.

Establishing Built-In Gradients

The built-in gradient includes the temperature and moisture gradient that "lock" into the slab at the zero-stress time (TZ). TZ occurs after the final set and is the point in time when the slab has grown sufficient strength (essentially changing from a semisolid to solid state), to respond to temperature changes. Although moisture gradients at TZ have been shown to be close to zero (Wells et al. 2006), temperature gradients at this point in time can have influential values. Built-in temperature gradients are important because as a result of this gradient the slab does not remain flat during its service life, even when temperature and moisture gradients are zero. Before TZ, the slab is flat regardless of the temperature gradient in

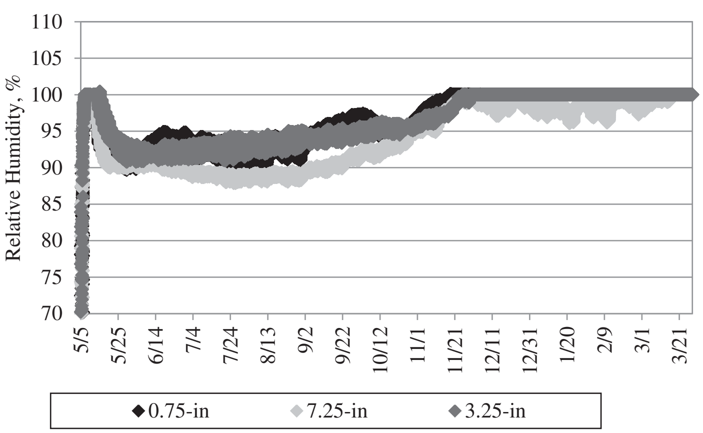


Figure 3.26. Relative humidity in the concrete at midslab for Slab 2 in Cell 71.

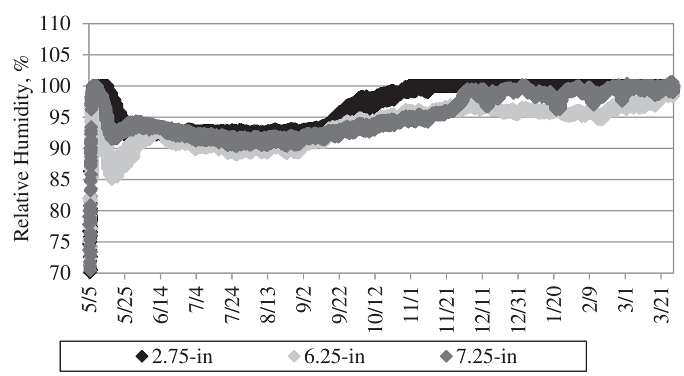


Figure 3.27. Relative humidity in the concrete for the corner of Slab 2 in Cell 71.

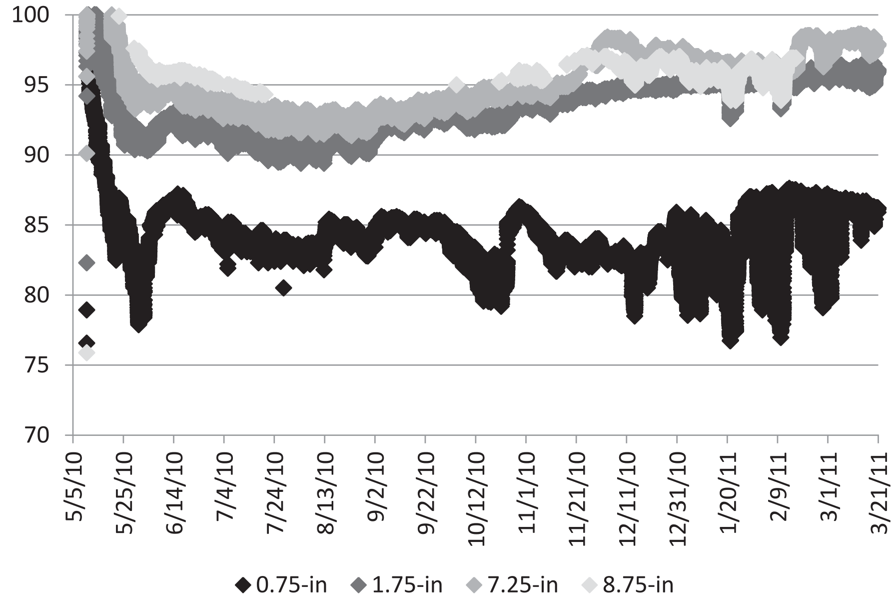


Figure 3.28. Relative humidity in the concrete at the edge of Slab 1 in Cell 72.

the slab. The temperature gradient that is present in the slab at TZ is "locked" into the slab.

The TZ, WAT, and built-in temperature gradient were established for each instrumented cell at MnROAD. To establish TZ, two methodologies were used, one based on the variation seen in the measured strain with respect to temperature changes in the slab (Method 1) and the other based on the initiation of curling in the slabs with respect to LTG (Method 2), as detailed in Appendix H.

For Cell 71, TZ was established as between 15 and 17 hours using Method 1 and between 14 and 15 hours using Method 2. The early-age data were unfortunately missing for Cell 72. TZ was determined based on the maturity concept for the upper layer in Cell 72 as between 16 and 20 hours, whereas TZ for the lower layer was estimated as sometime between 16 and 24 hours. Overcast conditions at the time of paving resulted in relatively constant temperature conditions in the slab over the first 24 hours after paving. Therefore, the climatic conditions

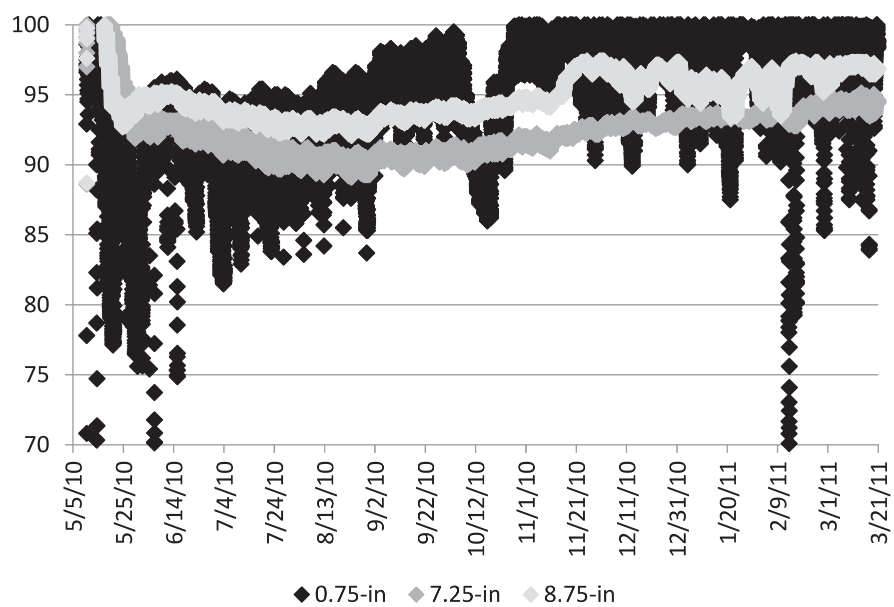


Figure 3.29. Relative humidity in the concrete at the edge of Slab 2 in Cell 72.

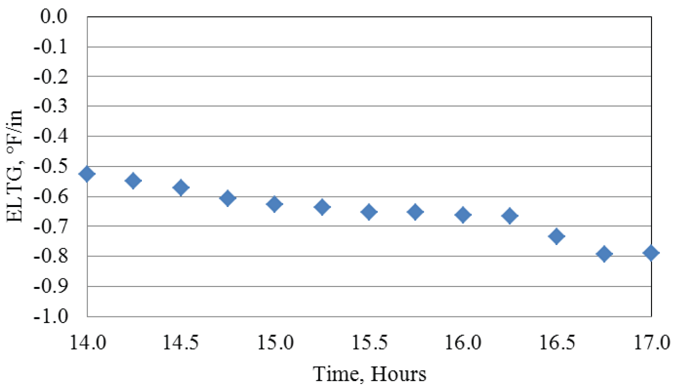


Figure 3.30. ELTG over the range of TZs for Slab 2 in Cell 71, using thermocouple data.

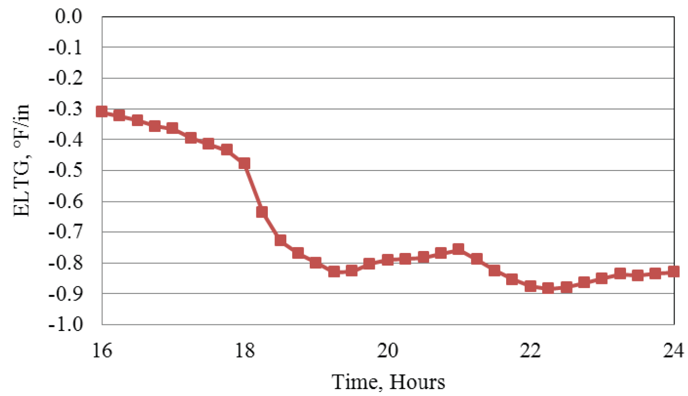


Figure 3.31. ELTG estimated using thermocouple data in Slab 2 for the range of the TZs for Cell 72.

in the slab at the TZ could be established relatively well, even with the limited data available.

The slab WAT at TZ is another parameter that needs to be established. This parameter is significant because it defines the amount of uniform thermal expansion and contraction in the slab. The WAT at hour 15, selected as the TZ, is approximately 79°F for the lower PCC in Cell 71. For the upper PCC layer in Cell 71, the WAT at TZ (hour 15) is approximately 75°F. For Cell 72, the WAT at the selected TZ (hour 20) is approximately 58°F for the upper PCC layer and approximately 62°F for the lower PCC layer.

The ELTG at TZ is the built-in temperature gradient that locks into the slab and influences its future shape. For Cell 71, the average TZ (based on both methodologies), for top and lower layers is between 15 and 16.25 hours. The ELTGs estimated using thermocouple data from Slab 2 between 14 and 17 hours after paving are shown in Figure 3.30. The thermocouple data from Slab 1 on this cell was not usable for the first 5 days. Figure 3.30 shows that the ELTG is approximately –0.7°F/in., which corresponds to a built-in temperature difference of approximately –6°F for a 9-in. PCC slab.

For Cell 72, the ELTG calculated based on the thermocouple readings over the time period of 16 to 24 hours after paving (the estimate of TZ), is shown in Figure 3.31. Because the ELTG in the entire time span shows a significant variation, hour 20, at which the ELTG stabilizes and remains constant thereafter, is selected as TZ for both layers in this cell. The ELTG at TZ is approximately –0.8°F/in., which corresponds to a built-in temperature difference of approximately –7°F for a 9-in. PCC slab.

PCC/PCC Interface Tensile Bond Strength Test

During the survey of agencies regarding PCC/PCC construction, a major concern expressed by many agencies had to do with the interface bond between the two PCC layers. A good bond between both the PCC layers is essential for the long-term performance of PCC/PCC composite pavements. The bond between the two PCC layers was tested by the FHWA Mobile Concrete Laboratory in August 2011, more than 1 year after construction. ASTM C1583-04 (Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension [Pull-Off Method]) was used to evaluate the bond.

The pull-off test involves applying a direct tensile load to a partial core (one that is advanced completely through the upper PCC layer but only partially through the lower PCC layer) until failure occurs. The tensile load is applied to the partial core through the use of a metal disk with a pull pin, bonded to the surface of the upper layer with an epoxy. A loading device with a reaction frame applies the load to the pull pin. The load is applied at a constant rate, and the ultimate load is recorded. Figure 3.32 illustrates the principle of the pull-off test.

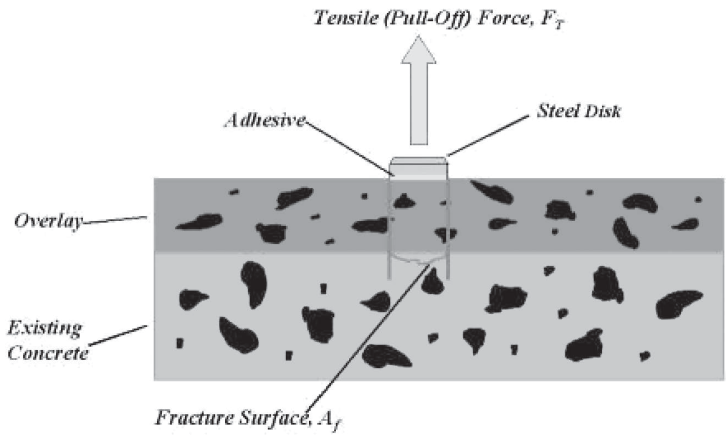
The pull-off strength (SPO) is defined as the tensile (pull-off) force (F_T) divided by the area of the fracture surface (A_f), using Equation 3.4:

$$SPO = F_T / A_f \tag{3.4}$$

The Mobile Concrete Laboratory staff used Proceq's DYNA Z15 to perform all the pull-off tests. Pull-off testing was performed on the passing lanes of Cell 71 and Cell 72. The weather conditions were windy, and testing on both the cells was performed on the same day. The results of the pull-off tests are shown in Tables 3.13 and 3.14 for Cell 71 and Cell 72, respectively. Figure 3.33 shows the fractured cores of the pull-off tests from both the cells.

The results from the tensile pull-off tests show the following:

- The tensile bond strength results of all the tests were higher than 1 MPa, which is generally considered good bond strength. Except for Test B4, results of all the tests in the two



Source: FHWA 2000.

Figure 3.32. Schematic of pull-off test.

Table 3.13. Pull-off Test Results for Cell 71

Core ID	Tensile Strength, (MPa)	Location of Fracture, Comment	Average Core Length (in.)
B1	na	Bond failure at the surface, epoxy	na
B2	na	Bond failure at the surface, epoxy	na
B3	2.0	Below interface in substrate concrete	3.4
B4	1.3	Below interface in substrate concrete	3.5
B5	1.8	Below interface in substrate concrete, pulled-off aggregate	3.8
B6	1.9	At the interface	3.3

Note: Core lengths are the average of three readings; na = not applicable.

Table 3.14. Pull-off Results for Cell 72

Core ID	Tensile Strength (MPa)	Location of Fracture, Comment	Average Core Length (in.)
A1	2.2	Below interface in substrate concrete	3.4
A2	2.4	Predominantly along the bond surface and partially in the substrate	3.3
A3	3.0	In the top layer	2.0
A4	3.5	In the top layer	1.0
A5	2.3	Partially along the bond surface and partially in the substrate	3.2
A6	1.8	Below interface in substrate concrete	3.6

Note: Core lengths are the average of three readings.

Figure 3.33. Fractured cores: Cell 72 (left) and Cell 71 (right).

cells were equal to or higher than 1.8 MPa. Thus, the composite pavement layers in Cell 71 and Cell 72 are well bonded to each other.

- Except for A3 and A4, failures occurred either in the lower PCC layer or partly at the interface and partly in the lower PCC layer. This shows that the upper PCC layer is stronger than the lower PCC layer, which is consistent with the results from the compressive and flexural strength testing.
- In the case of A3 and A4, fracture occurred in the upper PCC layer and the tensile strengths of both the tests were significantly higher (3 and 3.5 MPa) than were those of other tests (2.4 MPa or lower). It appears that the partial cores at A3 and A4 did not advance beyond the bond interface. This could be attributable to the upper PCC layer having greater thickness at the A3 and A4 locations. Therefore, at A3 and A4, tensile stress was applied only to the upper PCC layer, so fracture occurred at the weakest point in the upper PCC layer. The high tensile strength at A3 and A4 is because of the high strength of the upper PCC layer.
- The standard deviations of the results of four tests in Cell 71 and the four tests in Cell 72 (excluding the A3 and A4 tests) were low, which shows good repeatability of the test. The average tensile strength of the four tests of Cell 71 was 1.8 MPa and the average tensile strength of the four tests in Cell 72 was 2.2 MPa. This shows that the LC mix was slightly stronger than the RCA mix. The higher amount of fly ash in the LC mix compared with the RCA mix (60% versus 40%) could be the reason for the higher strength.
- Tests B1 and B2 had epoxy failure. The diamond grinding texture in this cell provided smaller surface area for the epoxy and may have contributed to the failure.

MEPDG JPCP Transverse Cracking Models for PCC/PCC

The *MEPDG* considers two mechanisms of transverse cracking: bottom-up and top-down cracking. When the truck axles are near the longitudinal edge of the slab, midway between the transverse joints, a critical tensile bending stress occurs at the bottom of the slab, as shown in Figure 3.34.

This stress increases greatly when there is a high positive temperature gradient through the slab (on a hot sunny day,

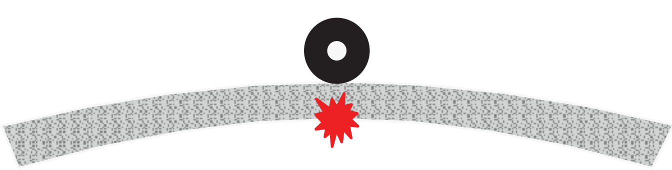


Figure 3.34. Curling of PCC slab caused by night-time negative temperature difference, plus critical traffic loading position. This resulted in high tensile stress at slab bottom.

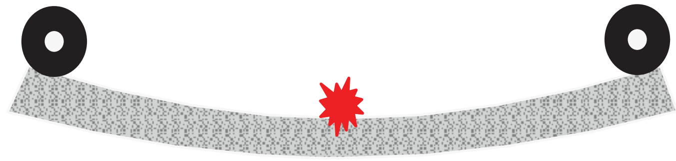


Figure 3.35. Curling of PCC slab caused by nighttime negative temperature difference plus critical traffic loading position. This resulted in high tensile stress at slab top.

the top of the slab is warmer than the bottom of the slab). The critical loading condition for top-down cracking involves a combination of axles that loads the opposite ends of a slab simultaneously. In the presence of a high negative temperature gradient, such load combinations cause a high tensile stress at the top of the slab near the middle of the critical edge, as shown in Figure 3.35.

During the analysis of the *MEPDG* for the SHRP 2 R21 project, numerous strange predictions for both bottom-up and top-down cracking resulting from changes of the thickness of layers in a PCC/PCC pavement were encountered. For example, equivalent PCC/PCC systems with a composite thickness of 9 in. and identical material properties were shown to produce vastly different performances in bottom-up damage, as shown in Table 3.15. Note that all pavements are equivalent (i.e., both layers have identical properties); their only difference is the top layer thickness. The *MEPDG* should not be sensitive to this if the pavements are otherwise equivalent. However, a 2.8 in.-over-6.2 in. PCC/PCC pavement is predicted to have ~200% more damage in bottom-up cracking than is a 3 in.-over-6 in. PCC/PCC pavement.

To address this damage issue, two points were considered:

1. The assumption of zero friction between PCC slab and the base layer is a reasonable one for bonded-PCC-over-JPCP, but for new construction of PCC/PCC slabs, such an assumption, is not reasonable. As a result, the model for transverse cracking, as applied to bonded-PCC-over-JPCP, was modified to account for the presence of full friction at the base over the initial period of the pavement life, as it is for new JPCP projects.
2. The EICM temperature analysis was analyzed for the composite slab. In previous versions of *MEPDG,* the raw EICM thermal data (at variably spaced nodes) was used to compute the temperature at equally spaced nodes through the slab. In subsequent versions of the *MEPDG,* the EICM raw outputs (temperature at variably spaced nodes) are used directly for the stress analysis. This creates a discrepancy between bonded-PCC-over-JPCP and structurally equivalent single-layer results. Because the design process for the single-layer JPCP involved extensive validation and calibration as part of NCHRP 1-37A and NCHRP 1-40B, it is

Table 3.15. Comparison of Predicted Performance in a 9-in. JPCP and Structurally Equivalent Bonded PCC Over JPCP Systems Using* MEPDG *Version 1.003

Top-Layer Thickness (in.)	Bottom-Layer Thickness (in.)	IRI (in./mi)	Percent Slabs Cracked	Faulting (in.)	Bottom-up		Top-down	
					Damage	Percent Difference[a]	Damage	Percent Difference[a]
3.1	5.9	68.8	0.8	0.006	0.0252	13.0	0.0808	–4.5
3.0	6.0	68.9	0.8	0.006	0.0223	na	0.0846	na
2.9	6.1	69.3	0.8	0.006	0.0434	94.6	0.0750	–11.3
2.8	6.2	69.4	0.9	0.006	0.0681	205.4	0.0624	–26.2
2.5	6.5	69.4	0.8	0.006	0.0488	118.8	0.0715	–15.5
2.0	7.0	69.4	0.7	0.006	0.0404	81.2	0.0722	–14.7
1.5	7.5	69.1	0.6	0.005	0.0136	–39.0	0.0740	–12.5

[a] Compared with 3-in. over 6-in. PCC/PCC pavement.

reasonable to validate two-layer predictions against those of a structurally equivalent single-layer JPCP as predicted by *MEPDG*. To make this possible, modification to the EICM analysis in the two-layer PCC case were made, as detailed in the following sections, so that it would be more compatible with the single-layer design.

Aside from changes to the *MEPDG* to allow for thinner upper lifts and full-friction between the slab and base, no modifications of substance were made to the *MEPDG* JPCP transverse cracking models. Provided that the appropriate revisions were made to the EICM, these models were deemed sufficient for predicting bottom-up and top-down cracking in PCC/PCC pavements using the Bonded-PCC-over-JPCP project.

Longitudinal Cracking Models for PCC/PCC

Longitudinal cracking caused by repeated loadings and large slab upward curling in JPCP is a phenomenon that has received little attention in pavement engineering but has occurred on several projects, especially with widened slabs on Specific Pavement Studies-2 (SPS-2) sites. The SHRP 2 R21 tour of PCC/PCC pavements identified no longitudinal cracking in the Netherlands, Austria, or Germany. The *MEPDG* does not consider longitudinal cracking; however, this is a potential topic for future research and development.

MEPDG JPCP Faulting Model for PCC/PCC

Repeated heavy axle loads crossing transverse joints create the potential for joint faulting. Faulting can become severe and cause loss of ride quality and require premature rehabilitation if any of the following conditions occurs: repeated heavy axle loads; poor joint load transfer efficiency (LTE); presence of an erodible base, subbase, or subgrade beneath the joint; and presence of free moisture under the joint.

The evaluation of the *MEPDG* JPCP faulting model determined that the model was applicable to PCC/PCC composite pavements. It was found that the program implemented in the *MEPDG* framework for the bonded-PCC-over-JPCP case assumes the modulus of the PCC overlay for the composite slab. In cases in which the elastic moduli of the two layers do not vary greatly, the error this creates in the faulting output is hardly detectable because the differences in faulting in the two cases are less than the precision of reported results. However, when the two layers have substantially different elastic moduli, as illustrated in the sensitivity analyses in Chapter 4, the error in implementation is higher.

In addition to the elastic modulus, other material properties of one layer in the system are being assumed for the composite slab in the *MEPDG,* rather than computing an effective property for an equivalent single-layered slab. In the case of the elastic modulus, in *MEPDG* v. 1.3000:R21 the program was modified so that the system uses an equivalent modulus generated from the moduli of the two layers, rather than simply forcing the upper-lift elastic modulus as the modulus of the overall system. An approach that accounted for the inequality of the CTEs by modifying the equivalent temperature gradients was developed. This approach is described in the *MEPDG* EICM modifications section below. Aside from the EICM modifications and their impact on faulting predictions, no other modifications of substance were made to the *MEPDG* JPCP Faulting Model, and this model was deemed sufficient for predicting faulting in PCC/PCC pavements using the Bonded-PCC-over-JPCP project.

MEPDG Enhanced Integrated Climatic Model (EICM) for PCC/PCC

In the original EICM thermal analysis (version 1.003 and versions before 1.014:9030A), 10 nodes were distributed through the PCC slab with an additional node at the bottom of the base layer, resulting in a total of 11 nodes used to represent the temperature through the PCC slab and base with respect to a reference temperature (NCHRP 2004; Larson and Dempsey 1997; Lytton et al. 1989). This distribution of nodes was then used to calculate the nonlinear stresses at the top and bottom of the slabs for damage calculations. During subsequent developments, to provide consistent results between PCC/PCC composite pavements and an equivalent JPCP, the EICM thermal analysis was revised in versions 1.014:9030A and 1.206:R21 for SHRP 2 R21. Rather than the 10 nodes being applied to the entire composite slab (approximated by the bonded PCC overlay project), each PCC layer was assigned 10 nodes, which resulted in the use of a minimum of 20 temperature nodes for the entire slab and base. These additional nodes presented two key challenges. The first was that their inclusion dramatically increased the run time for the damage calculation in PCC/PCC composite pavement. The second, and more important, was that the system with additional nodes threatened the self-consistency of the *MEPDG*. As noted, for a single-layer PCC pavement, EICM uses only 10 nodes. For a new PCC/PCC pavement, the thermal gradient was approximated by EICM using 20 or more nodes through the composite slab. This modeling difference rippled through the project runs and provided results for structurally equivalent systems that were significantly different, although they should have been nearly identical.

To address this inconsistency, the thermal gradient for a bonded PCC overlay was modified to use PCC layer thicknesses and the base layer thickness to develop 11 equally spaced nodes through the composite slab and a twelfth node at the bottom of the base layer, thereby creating 10 intervals in the composite slab and one for the base layer. The thermal node arrangement used in *MEPDG* versions 1.014:9030A and 1.206:R21 is described in Figure 3.36a. The modified thermal node arrangement (used in

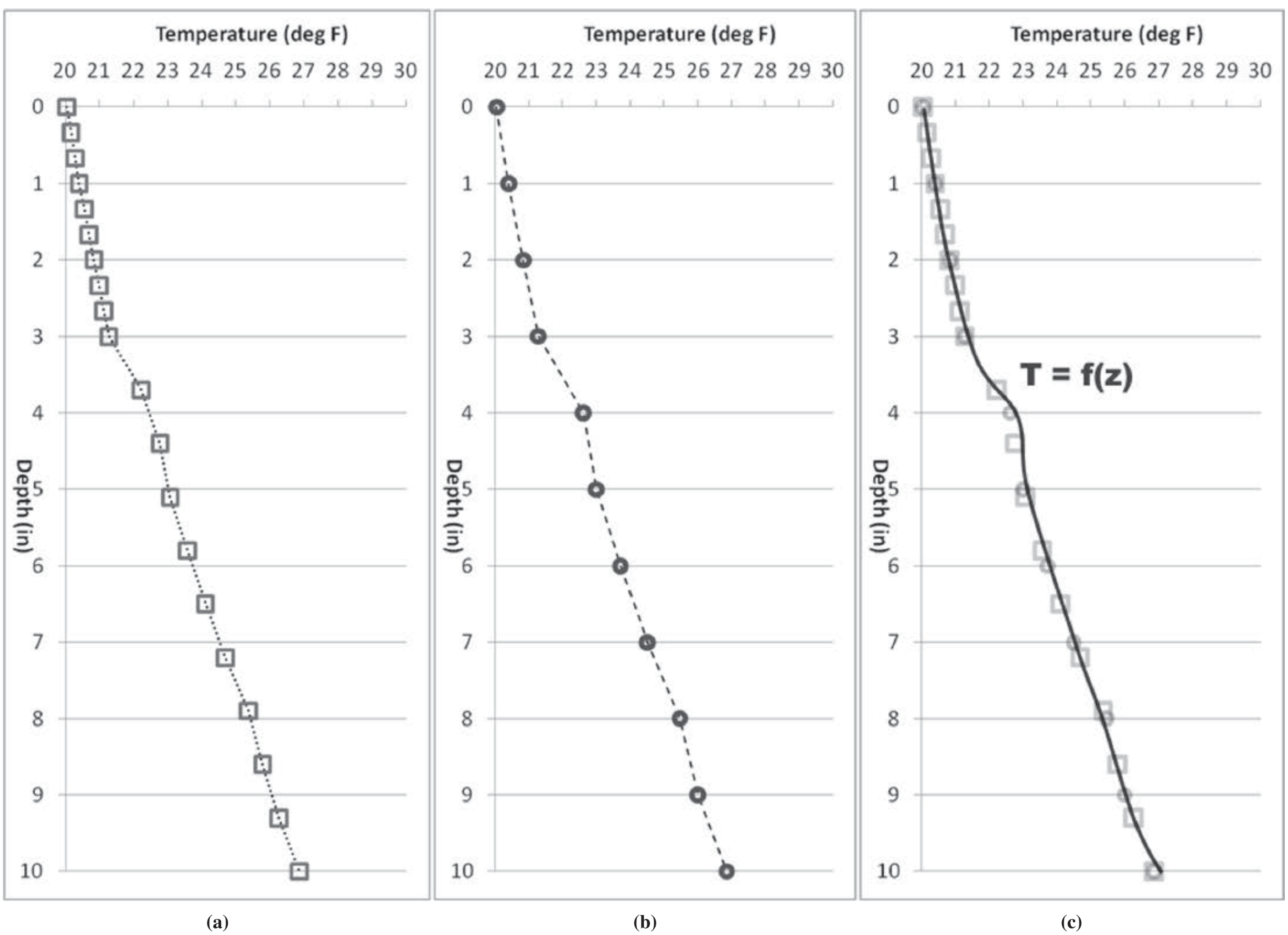


Figure 3.36. Modified thermal nodes through slab thickness in MEPDG *for (a)* MEPDG *version 1.014:9030A, (b)* MEPDG *v. 1.3000:R21, and (c) both approximations relative to nonlinear thermal gradient.*

MEPDG v. 1.3000:R21) is described in Figure 3.36b. The recommended modification implemented in *MEPDG* v. 1.3000:R21 ensured that PCC/PCC projects and their structurally equivalent single-layer JPCP projects have the same number of intervals through the PCC layers, whether the project is single-lift or two-lift.

EICM Calculation of Subgrade Response for Single-Layer and Composite Two-Layer Rigid Pavement Systems

In adapting the *MEPDG* and EICM to model newly constructed PCC/PCC composite pavements as bonded PCC overlays of existing pavements, a major consideration was the effect of this modeling choice on the calculation of the subgrade response, or *k*-value. For a typical single-layer PCC pavement, the effective dynamic *k*-value is obtained by first determining the deflection profile of the PCC surface using an elastic layer program, modeling all layers specified for the design (Figure 3.37). The subgrade resilient modulus is adjusted to reflect the lower deviator stresses that typically exist under a concrete slab and base course compared with the deviator stress used in laboratory resilient modulus testing. Next, the computed deflection profile is used to back calculate the effective dynamic *k*-value. Thus, the effective dynamic *k*-value is a computed value, not a direct input to the *MEPDG* design procedure (except in rehabilitation).

The effective *k*-value used in the *MEPDG* is a dynamic *k*-value, as opposed to traditional static *k*-values used in previous design procedures. The effective dynamic *k*-value of the subgrade is calculated for each month of the year and used directly to compute critical stresses and deflections in the incremental damage accumulation over the design life of the pavement. Factors such as water table depth, depth to bedrock, and frost penetration depth (frozen material) can significantly affect effective dynamic *k*-value. All of these factors are considered in the EICM.

However, this procedure is different for bonded PCC overlay projects. For a bonded PCC overlay, only the existing PCC layer is used to determine the deflection profile of the PCC using an elastic layer program. Thus, the stiffness contribution of the overlay is discounted. Figure 3.38 shows the monthly difference in *k*-values between structurally identical bonded PCC overlay of JPCP and new JPCP using a previous version of *MEPDG*. The bonded PCC overlay of JPCP (the proxy for PCC/PCC composite pavement) is 3 in. over 6 in, whereas the JPCP is 9 in., with all material properties for all PCC layers being identical. As described above, this difference arises because the *k*-value is backcalculated using the elastic layer deflection profile and 9 in. PCC for the JPCP but only 6 in. PCC for bonded PCC overlay of JPCP and consequently for PCC/JPCP. For *MEPDG* (v. 1.3000:R21) the bonded PCC overlay of PCC pavement was modified to include the overlay, or in terms of composite PCC/PCC, the subgrade response calculation includes both lifts of the two-layer PCC slab.

Figure 3.39 illustrates that modifications to calculation of the subgrade *k*-value in the bonded PCC overlay project have reduced the extreme differences in the *k*-value for the structurally equivalent systems. However, it may be valuable for future research to note that small differences (approximately 3%) still remain in the monthly calculation of the subgrade reaction, suggesting that additional modifications to the EICM calculation will be necessary to make the two designs identical.

Comparison of PCC/PCC and Structurally Equivalent Single-Layer JPCP

An important trial in the evaluation of *MEPDG* (v. 1.3000: R21) was the verification of the performance predictions for PCC/PCC (or bonded PCC over JPCP in the *MEPDG*) as

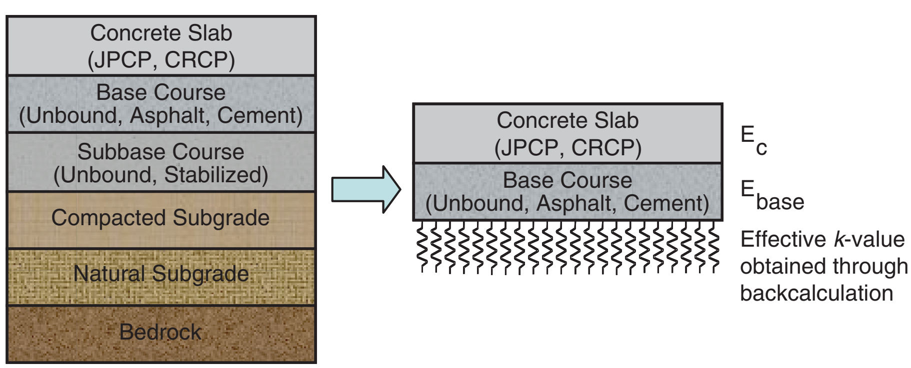
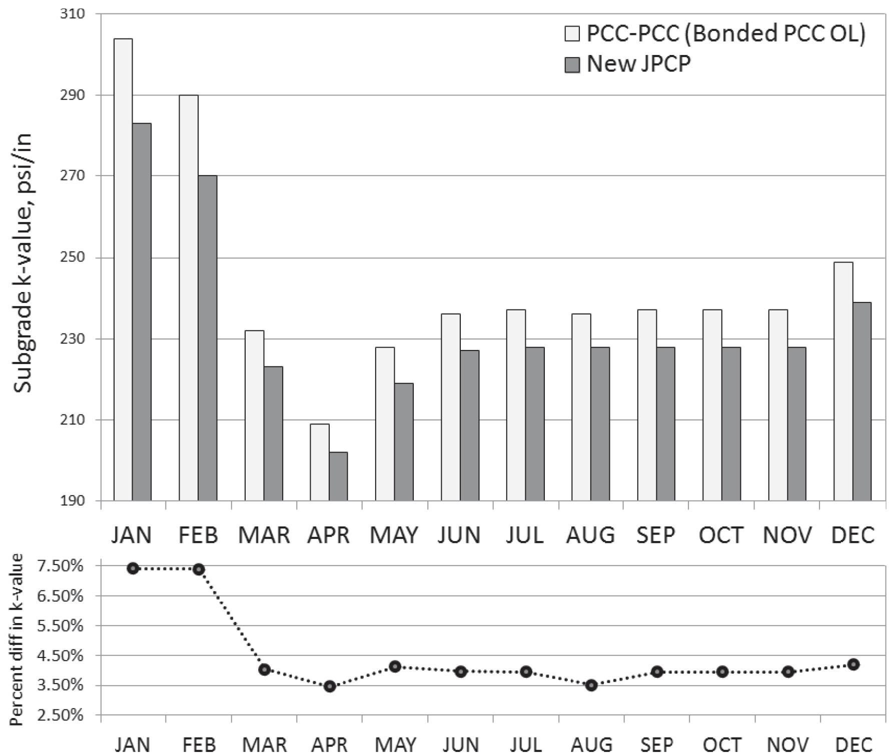
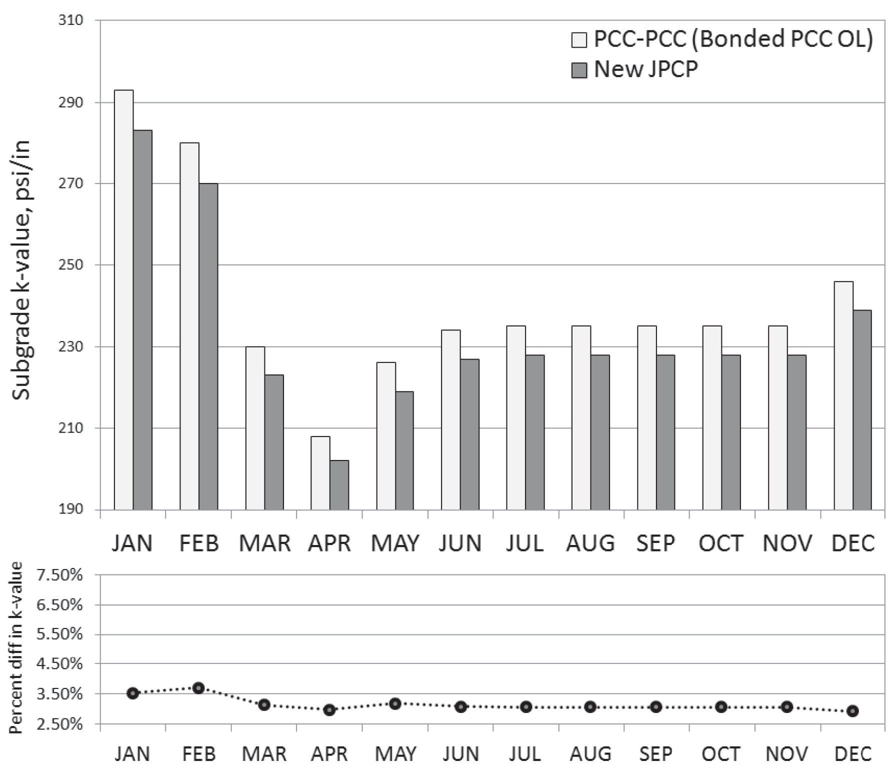


Figure 3.37. Structural model for rigid pavement structural response computations.



Note: OL = overlay.

Figure 3.38. Subgrade* k*-value calculation for a PCC/PCC pavement and its JPCP single-layer structural analog (versions 1.003 and 1.014:9030A of* MEPDG*).



Note: OL = overlay.

Figure 3.39. Subgrade* k*-value calculation for a PCC/PCC pavement and its JPCP single-layer structural analog (*MEPDG *v. 1.3000:R21).

Table 3.16. Comparison of 9-in. Total Thickness PCC/PCC and Structural Equivalent 9-in. JPCP Using* MEPDG *and Identical Properties for All PCC

Project	International Roughness Index (IRI), in./mi (Limit: 172)	Percentage Slabs Cracked (Limit: 15)	Mean Joint Faulting, in. (Limit: 0.12)	Bottom-up Cracking Damage	Top-down Cracking Damage
PCC/PCC	68.6	0.4	0.006	0.0427	0.0439
JPCP	67.1	0.4	0.006	0.0444	0.0430

Note: *MEPDG* (v. 1.3000:R21).

compared with that of a structurally equivalent single layer JPCP. Table 3.16 shows the results of comparing *MEPDG* performance prediction of two structurally equivalent 9-in. PCC systems (the baseline PCC/JPC and a single-layer JPCP). Note that all pavement layer properties for each case are exactly identical. It was expected that the performance of the systems would be very similar, if not identical.

This trial yields similar performance across all performance measures. It is worth noting that it took three revisions of the *MEPDG* as part of the SHRP 2 R21 project to ensure that structurally similar pavements perform similarly.

Influence of PCC Layer Thickness in the Performance of Structurally Equivalent PCC/PCC

Tests were also run to determine if the predicted cracking damage calculated using *MEPDG* (v. 1.3000:R21) in structurally equivalent PCC/PCC pavements are similar. These test runs involved structurally equivalent PCC/PCC pavements that differed only with respect to the relative thickness of their PCC layers. The combined thickness for both layers was always 9 in., and all properties for both layers were identical. The results are shown in Table 3.17. The results show that similar performance is achieved for all selected PCC layer thicknesses, as expected, and point to the fact that the *MEPDG* (v. 1.3000: R21) is functioning properly with respect to this issue. Note that the *MEPDG* will not allow a top-lift thickness of less than 1 in.; selecting 1 in. or less for the upper PCC will result in instabilities in the program.

Additional layer thicknesses were evaluated for a different base PCC/PCC case. Again, these test runs involved structurally equivalent PCC/PCC pavements that differed only with respect to the relative thickness of their PCC layers. The combined thickness for both layers was always 9 in., and all properties for both layers were identical. The results are shown in Table 3.18, which illustrates a pavement that fails in faulting (according to specified performance measures) consistently for all cases. One case is a JPCP project, and the rest are its PCC/PCC structural equivalents. Pass or fail, similar performance is achieved for all selected PCC layer thicknesses and project types for structurally equivalent PCC pavements. Again, the

Table 3.17. Comparison of Structurally Equivalent 9-in. PCC/PCC Pavements Using* MEPDG *(v. 1.3000:R21)

h1 (in.)	h2 (in.)	IRI, in./mi (Limit: 172)	Percentage Slabs Cracked (Limit: 15)	Mean Joint Faulting, in. (Limit: 0.12)	Bottom-up Cracking Damage	Top-down Cracking Damage
3.1	5.9	68.5	0.4	0.006	0.0427	0.0439
3.0	6.0	68.6	0.4	0.006	0.0427	0.0439
2.9	6.1	68.6	0.4	0.006	0.0427	0.0439
2.8	6.2	68.7	0.4	0.006	0.0427	0.0438
2.5	6.5	68.8	0.4	0.006	0.0426	0.0438
2.0	7.0	69.1	0.4	0.006	0.0427	0.0438
1.5	7.5	69.4	0.4	0.006	0.0426	0.0438

Note: h1 = thickness of the upper layer; h2 = thickness of the lower layer.

Table 3.18. Comparison of Additional Structurally Equivalent 9-in. PCC/PCC Pavements Using MEPDG

h1 (in.)	h2 (in.)	IRI, in./mi (Limit: 172)	Percentage Slabs Cracked (Limit: 15)	Mean Joint Faulting, in. (Limit: 0.12)	Bottom-up Cracking Damage	Top-down Cracking Damage
NA	9.0	147.5	11.1	0.136	0.3494	0
1.5	7.5	150.4	11.0	0.130	0.3475	0
2.0	7.0	150.0	11.0	0.131	0.3475	0
2.5	6.5	149.4	11.0	0.131	0.3475	0
3.0	6.0	149.0	11.0	0.131	0.3477	0
3.5	5.5	148.7	11.0	0.132	0.3474	0
4.0	5.0	148.2	11.0	0.132	0.3474	0
4.5	4.5	148.0	11.0	0.132	0.3477	0
5.0	4.0	147.6	11.0	0.132	0.3477	0
5.5	3.5	147.4	11.0	0.133	0.3478	0
6.0	3.0	147.1	11.0	0.133	0.3478	0
6.5	2.5	146.9	11.0	0.133	0.3474	0

Note: *MEPDG* (v. 1.3000:R21); NA = not available; h1 = thickness of the upper layer; h2 = thickness of the lower layer.

results point to the fact that the modifications to the *MEPDG* in v. 1.3000:R21 are robust.

Comparing these results with those obtained with previous versions of the *MEPDG* (an example of which is discussed earlier in this chapter and shown in Table 3.15), clearly indicates that the modifications to the *MEPDG* as part of SHRP 2 R21 project were necessary and valid and have resulted in a version of the *MEPDG* that is more appropriate for designing PCC/PCC composite pavements.

Overall *MEPDG* Performance Modeling

Table 3.19 presents *MEPDG* performance predictions for the R21 PCC/PCC database sections. These results were generated using the final version of the *MEPDG* modified as part of the SHRP 2 R21 project (*MEPDG* v. 1.3000:R21).

An additional measure of model capabilities is to examine the damage in top-down cracking and bottom-up cracking for specific sections relative to their observed field measured cracking. Figures 3.40 and 3.41 illustrate the ability of the revised *MEPDG* to account for the damage of a pavement relative to its percentage of cracked slabs. All PCC/PCC sections were used to make this important comparison. The s-shaped curves represent the national calibration curves obtained for JPCP one-layer slabs relating accumulated damage to transverse fatigue cracking of JPCP. The composite PCC/PCC pavements show good correspondence to these curves. Thus, the national models used in JPCP design also can be used for PCC/JPCP design of composite pavement. The same results were obtained for HMA/PCC composite pavements as described in Volume 1.

Lattice Modeling of PCC/PCC Interface Behavior (Debonding)

Debonding between the PCC layers may lead to premature failure of the pavement. However, debonding is not currently modeled by the *MEPDG*. Granju (2001) illustrates the process of debonding: crack initiation as a result of volume changes, thermal loads, and traffic loads, debonding occurring in the region of the crack, peeling of the top-lift (overlay) from the lower lift over time.

Lattice models for composite beam and composite slab simulations was determined by the SHRP 2 R21 research team to be the most effective manner of determining if debonding was a legitimate concern for PCC/PCC.

A lattice model consists of a triangular grid of points connected by one-dimensional spring elements. This network of springs represents the discretized medium. The lattice can be deformed by internal strains resulting from diffusive, thermal, or hygral processes or by external displacements or forces. Lattice models can differ in the representations of the constitutive relations used for individual springs (or, instead, the minimization of the stored elastic energy) (Schlangen and van Mier 1992). The varying properties of springs allow the lattice to simulate the behavior of heterogeneous media, such as concrete.

Table 3.19. Final Performance Predictions for R21 PCC/PCC Database Sections Using R21 Revised MEPDG

Section, Location	Climate	Traffic	PCC/PCC, Year Constructed, Joint Space, Dowels	IRI (in./mi)	Transverse Cracking (% Cracked Slabs)	Faulting (in.)	Comment
FL45 (3A), Fort Myers, Florida	Wet, Non-freeze	5 million trucks, 30 years	3 in. PCC, 9 in. Econocrete, 1978, 20 ft, 1 in.	Predicted: 96.9 Measured: 104	Predicted: 0.0 Measured: 0	Predicted: 0.081 Measured: 0.04	Econocrete had fc = 2,000 psi
FL45 (3B), Fort Myers, Florida	Wet, Non-freeze	5 million trucks, 30 years	3 in. PCC, 9 in. Econocrete, 1978, 20 ft, 1 in.	Predicted: 101.0 Measured: 112	Predicted: 6.8 Measured: 3	Predicted: 0.077 Measured: 0.07	Econocrete had fc = 1,250 psi
FL45 (2A), Fort Myers, Florida	Wet, Non-freeze	5 million trucks, 30 years	3 in. PCC, 9 in. Econo-crete, 1978, 15 ft, none	Predicted: 71.1 Measured: 119	Predicted: 0 Measured: 0	Predicted: 0.011 Measured: 0.13	Econocrete had fc = 2,000 psi
FL45 (2B), Fort Myers, Florida	Wet, Non-freeze	5 million trucks, 30 years	3 in. PCC, 9 in. Econo-crete, 1978, 15 ft, none	Predicted: 74.1 Measured: 113	Predicted: 0 Measured: 0	Predicted: 0.017 Measured: 0.09	Econocrete had fc = 1,250 psi
FL45 (2C), Fort Myers, Florida	Wet, Non-freeze	5 million trucks, 30 years	3 in. PCC, 9 in. Econocrete, 1978, 15 ft, none	Predicted: 71.5 Measured: 166	Predicted: 0.6 Measured: 5	Predicted: 0.011 Measured: 0.18	Econocrete had fc = 750 psi
K96, Haven, Kansas	Wet, freeze	2.1 million trucks, 14 years	3 in. PCC, 7 in. JPCP, 1997, 15 ft, 1 in.	Predicted: 92 Measured: NA	Predicted: 0 Measured: 0	Predicted: 0.043 Measured: 0.02	Predicted estimates measured well
K96, Haven, Kansas	Wet, freeze	2.1 million trucks, 14 years	3 in. PCC, 7 in. JPCP, 1997, 15 ft, 1 in.	Predicted: 90.4 Measured: NA	Predicted: 1 Measured: 0	Predicted: 0.041 Measured: 0.02	Predicted estimates measured well
I-10, Kansas	Wet, freeze	3.0 million trucks, 4 years	1.5 in. PCC, 11.8 in. JPCP, 2007, 15 ft, 1.25 in.	Predicted: 73.9 Measured: NA	Predicted: 0 Measured: 0	Predicted: 0.010 Measured: 0.03	Predicted estimates measured well
I-75, Detroit, Michigan	Wet, freeze	71.7 million trucks, 18 years	2.5 in. PCC, 7.5 in. JPCP, 1993, 15 ft, 1.25 in.	Predicted: 146.5 Measured: NA	Predicted: 0 Measured: 0	Predicted: 0.129 Measured 0.059	Predicted estimates measured well
I-94, MnROAD	Wet, freeze	0.7 million trucks, 1 year	3 in. PCC, 6 in. JPCP, 2010, 15 ft, 1.25 in.	Predicted: 65.3 Measured: NA	Predicted: 0 Measured: 0	Predicted: 0.001 Measured: NA	Predicted estimates measured well
A1, Austria	Wet, freeze	47.2 million trucks, 14 years	2 in. PCC, 7.9 in. JPCP, 1994, 18 ft, 1 in.	Predicted: 172.6 Measured: NA	Predicted: 4.4 Measured: 0	Predicted: 0.226 Measured: 0.10	Overpredicted faulting resulting from *MEPDG* model not accounting for EU base/subgrade prep to reduce faulting
A1, Austria	Wet, freeze	26.2 million trucks, 15 years	1.6 in. PCC, 8.3 in. JPCP, 1993, 18 ft, 1 in.	Predicted: 144.5 Measured: NA	Predicted: 0.2 Measured: 0	Predicted: 0.167 Measured: 0.10	Overpredicted faulting resulting from *MEPDG* model not accounting for EU base/subgrade prep to reduce faulting
A93, Germany	Wet, freeze	52.6 million trucks, 13 years	2.8 in. PCC, 7.5 in. JPCP, 1995, 16 ft, 1 in.	Predicted: 126.4 Measured: NA	Predicted: 0 Measured: 0	Predicted: 0.131 Measured: 0.10	Overpredicted faulting resulting from *MEPDG* model not accounting for EU base/subgrade prep to reduce faulting
N279, The Netherlands	Wet, freeze	11.9 million trucks, 8 years	3.5 in. PCC, 7 in. JPCP, 2000, 18 ft, 1 in.	Predicted: 119.8 Measured: NA	Predicted: 0.1 Measured: 0	Predicted: 0.122 Measured: 0.08	Overpredicted faulting resulting from *MEPDG* model not accounting for EU base/subgrade prep to reduce faulting

Note: *MEPDG* (v. 1.3000:R21); NA = not available; fc = 28-day compressive strength.

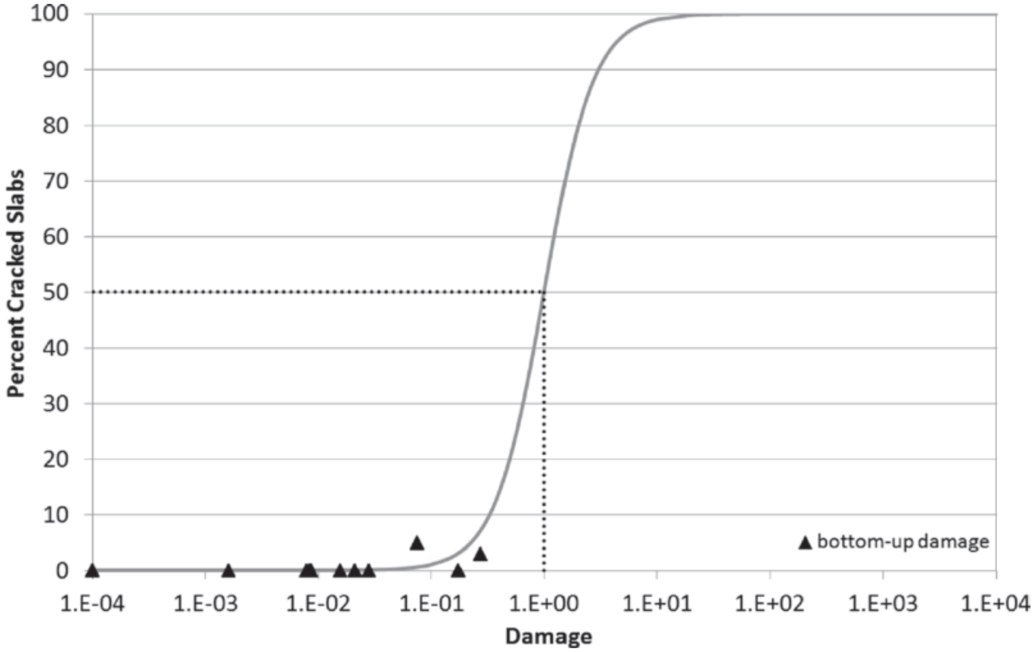


Figure 3.40. PCC/PCC measured cracking versus* MEPDG *(v. 1.3000:R21) predicted bottom-up damage.

The characterization of fracture, the essence of the debonding problem, often is simulated in pavement engineering using finite element methods (FEM) that begin with continuum equations. Although FEM can be successfully applied to a fracture problem, the success of the application depends largely on the homogeneity of the medium and the lack of disorder in crack propagation. Modeling of crack propagation in FEM requires the use of special elements in the crack path that must be specified a priori. Thus, if the medium is relatively homogeneous and the crack path can be anticipated, the FEM is more than adequate for simulating cracking. In cases exhibiting heterogeneity or nontrivial crack paths, the placement of the

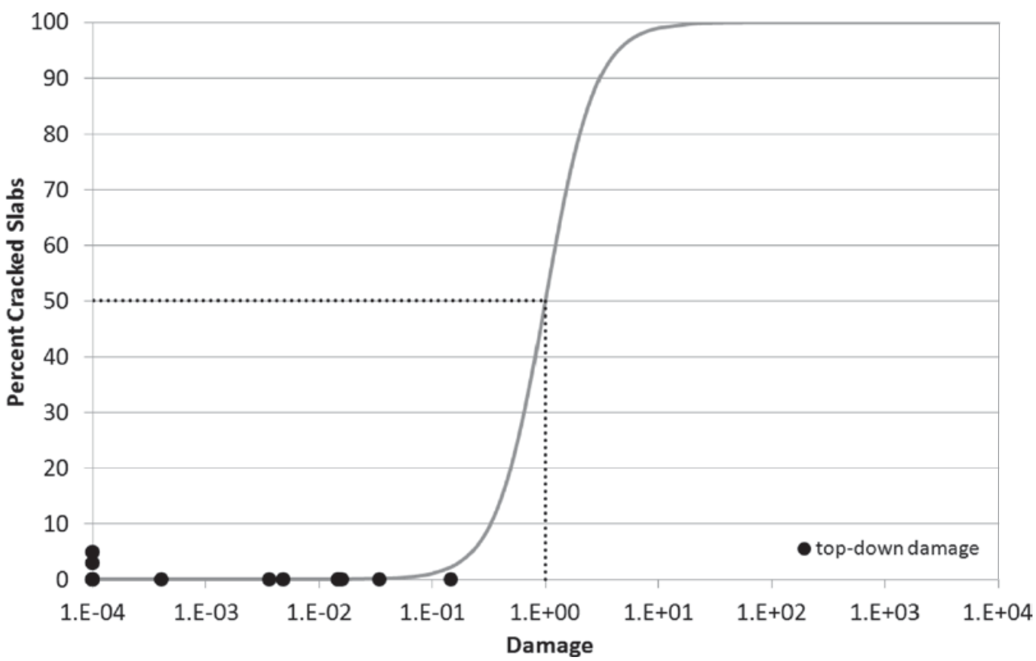


Figure 3.41. Measured cracking versus* MEPDG *(v. 1.3000:R21) predicted top-down damage.

specialized elements for fracture becomes a nontrivial problem. This problem is commonly solved using trial-and-error methods, with computationally expensive and cumbersome remeshing during the fracture process.

In light of these challenges, lattice models are a viable alternative to or a candidate to be coupled with FEM. In the lattice model, the simulation of cohesive cracking involves a reduction of stiffness and strength and the removal of individual springs. The lattice model does not require a priori knowledge of the crack path. The application of a specific lattice model (that is, a random network of spring elements) to the PCC/PCC debonding problem is prefaced with a summary of the model formulation and its simulation of fracture. The formulation of a lattice network of spring elements and the fracture rules assigned to the body are detailed below.

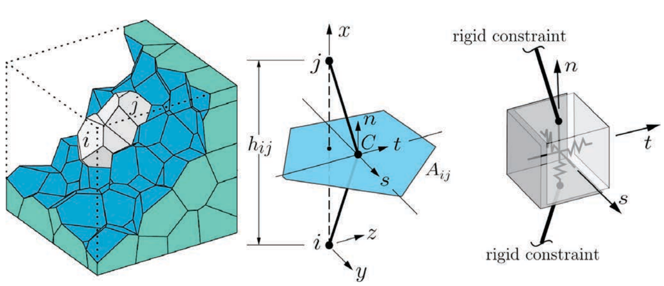
Basic Model Formulation

The lattice model applied to PCC/PCC composite pavements is a rigid-body-spring network (Bolander and Saito, 1998). Beginning with a region containing randomly distributed points, a Delaunay tessellation is used to define a network through the connection of these random points. The model then uses Voronoi diagrams to establish facets surrounding random points (to create nodes) and define nodal/facet volumes for later stress calculations. For each pair of neighboring nodes, we define an element *ij* connecting these nodes (Figure 3.42).

Element *ij* can be more easily characterized by the shared facet between the two nodes. Each facet is associated with a total of 6 spring constants corresponding to displacements in the *x, y, z* directions and rotations about each of these axes (see Equations 3.5 and 3.6).

$$k_x = k_y = k_z = E\frac{A_{ij}}{h_{ij}} \tag{3.5}$$

$$k_{\varphi x} = E\frac{J_p}{h_{ij}},\ k_{\varphi y} = E\frac{I_{22}}{h_{ij}},\ k_{\varphi z} = E\frac{I_{11}}{h_{ij}} \tag{3.6}$$



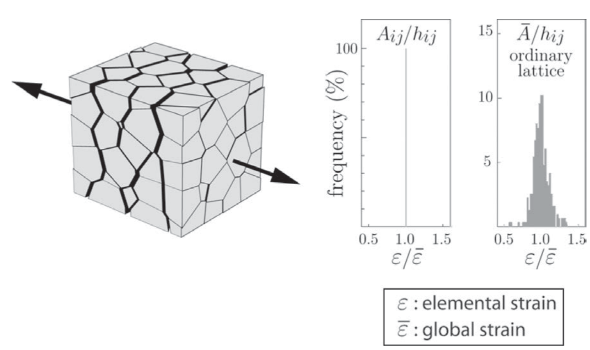
Source: Bolander 2008.

Figure 3.42. Element* ij *within network (*left*); facet and springs associated with* ij *(*right*).

The basis of the elastic equations for the lattice model is then, for representative element *ij*, $F = Kd$ where F is a load vector, K is a 12×12 stiffness matrix, and d is a vector of displacements. Taken for all elements in the lattice, this model represents a linear elastic system of equations. (Other factors not included in this formulation are those accounting for thermal and hygral diffusive processes, which can be and have been incorporated into lattice models.) More detail on the formulation and solution of these problems can be found in Schlangen and Garbocki (1996).

This formulation can be applied to any arrangement of elements in the lattice. The model described here uses a random geometry network to define the desired domain. The geometry is based on a Voronoi diagram for a given number of randomly generated points within the region. A major advantage to the use of random geometry networks is that these networks, as implemented in this model, are such that the model preserves elastic uniformity under loading. That is, the strain for each element in the random geometry network agrees with the global strain for the body under consideration (Schlangen and Garbocki 1996; Bolander and Sukumar 2005). Figure 3.43 illustrates the issue of elastic uniformity with histograms for elemental strains in a random geometry network and elemental strains in a regular lattice network for a cube under uniform tensile loading. The quantities A_{ij} and h_{ij} describe the area and length of element *ij*, respectively. In the instance of a regular or ordinary lattice, elemental area A_{ij} is described as uniform area $\bar{A}$ for all elements.

Furthermore, it should be noted that although regular, symmetric networks also ensure elastic uniformity, regular networks create bias in crack propagation. Random geometry networks eliminate this bias.



Source: Bolander 2008.

Figure 3.43. Random geometry networks are implemented in such a way that the model describes uniform elastic behavior under loading.

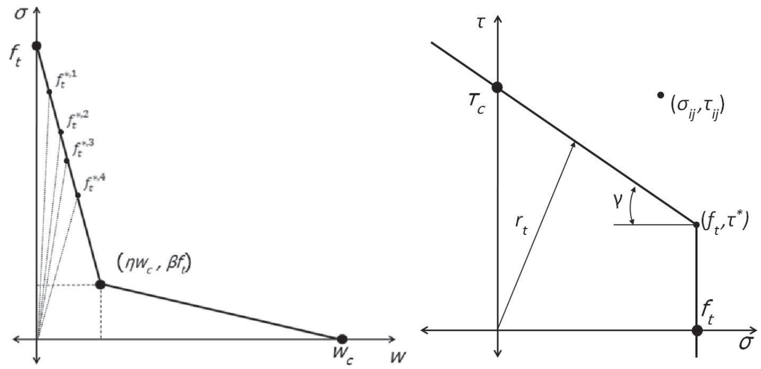


Figure 3.44. ***(*****Left*****) Bilinear softening to describe degradation of stiffnesses of elements under breaking. (Right) Mohr-Coulomb fracture criterion defines breaking in the described lattice model.***

Mixed-Mode Fracture Criteria

A key feature of this model, as alluded to previously, is that it does not adopt a continuum approach for fracture. Rather, its discrete representation allows for the development of parameters governing fracture in the body, as suggested by Jagota and Bennison (1994). These parameters are rules based on element response to critical stresses that are applied to the body after the constitutive equation has been solved for a given load-step.

In this model, crack initiation and propagation are governed by the tensile and shearing stresses at the facet-defining element *ij*. These stresses are considered in terms of a Mohr-Coulomb fracture criterion, in which both the tensile and resultant shear stresses acting on element *ij* are held against critical strength values for the element to determine if breaking occurs. For this model, after the solution of a given load-step, for each element the criterion $R = r/r_f$ is calculated, where $r = \sqrt{\sigma_{ij}^2 + \tau_{ij}^2}$, σ_{ij} is the normal stress for the facet, τ_{ij} is the resultant shear stress for the facet, and r_f is the distance to the fracture criterion curve in Figure 3.44. Other important parameters in the figure are the tensile strength f_t, pure shear failure criterion $\tau_c = \rho f_t$, approximate shear strength under critical tensile stress $\tau^* = \rho_2 f_t$, and the angle $\gamma = \tan^{-1}(\rho - \rho_2)$ to further specify the slope of the fracture criterion curve, where ρ and ρ_2 are specified material parameters through τ_c and τ^*.

Where $R > 1$, an element is defined as having undergone a fracture event. For instance, where more than one element has a value of R in excess of 1, only the element with the largest value of R is considered. More detail on the Mohr-Coulomb criterion and rules governing fracture for this model can be found in Bolander and Saito (1998). Later figures will refer back to the mixed-mode fracture curve of Figure 3.44 to define fracture events in simulations. Furthermore, the fracture rules assigned by this model include the degradation of spring stiffnesses to simulate softening. The model described here uses a bilinear softening relationship. For every successive fracture event according to the rules above, both the modulus of elasticity E and the tensile strength f_t are reduced according to a bilinear softening relation such as that of Figure 3.44.

Thus, there is a recursive effect between the softening relation and the mixed-mode fracture criterion itself. As fracture events are characterized for an element *ij* according to the Mohr-Coulomb criterion, both E and f_t for the element are consequently reduced. The softening in f_t then causes the σ and τ intercepts of the Mohr-Coulomb criterion, as represented in the curve in Figure 3.44 by f_t and τ_c, respectively, to become reduced as well. When extensive fracturing occurs, the mixed-mode fracture criterion will "collapse" gradually toward the origin as a result of the bilinear softening. This effect is summarized in Figure 3.45.

Later figures will depict simulations that involve mixed-mode fracture and characterizing these events, and the

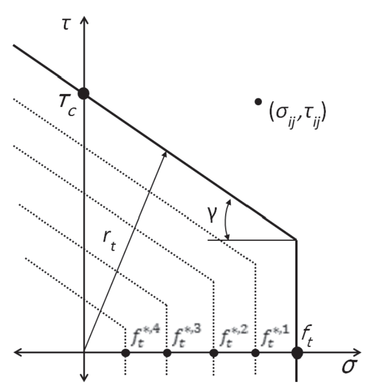


***Figure 3.45. This figure illustrates the so-called collapse of mixed-mode fracture criterion curve toward origin in the event of extensive fracture for a given element* ij.**

visibility of reduced criterion curves, such as is seen in Figure 3.45, will be visible in the simulations. Once the strength has been reduced to zero, the spring is removed from the lattice, and its contributions to the global stiffness matrix are consequently negated.

Beam Simulations to Understand Fracture and Interface Failure

To better develop models for debonding in composite pavements, laboratory tests for bond interface integrity were reviewed. Of the many available tests, the test developed at the Road Laboratory of Barcelona (Laboratorio de Caminos de Barcelona) was chosen because of its simple and direct experiment to induce the loss of bond integrity in a layered asphalt field core or laboratory sample. In the LCB procedure, cylindrical asphalt cores are clamped into the testing apparatus and subjected to loading under modified three-point bending. The intention is to generate shear stresses at the interface and avoid a bending moment by placing the interface very near the support. This procedure measures resistance to tangential stress at the interface and the displacement of the layers with respect to one another (Recasens et al. 2006).

Although the procedure is designed for asphalt composite cores or samples only, the LCB test also could be used for concrete composites. The specimens would be cast using wet-on-wet techniques to mirror the techniques used in the field. The LCB test was adopted for this research as the physical analogue for the debonding simulations. Figure 3.46 illustrates the general composition of the beams and an example of a two-layered beam with random lattice geometry and paired-node interface for simulation.

Multiple random geometry networks were generated to locate the interface at different locations measured from the roller support, as illustrated by l_i and corresponding hashed lines in Figure 3.46. The random point generators used to develop all specimens were oriented such that more points were distributed in the region surrounding the beam. Furthermore, each simulated beam was developed to accommodate nodes expressly to act as sites of forcing to accommodate three-point loading.

The next cases allow a closer examination of mixed-mode failure at a given interface. That is accomplished by loading a beam at midspan and gradually moving the interface away from the load toward a support. It was hypothesized that, as the interface was moved away from midspan, the nature of fracture would shift from one that is tensile to one that is mixed. In the interface strength problems, a controlled displacement of 0.01 mm is applied until 0.1 mm is reached, and a controlled displacement of 0.001 mm is applied thereafter. The controlled displacement is applied at midspan ($L/2$ = 500 mm) until simulated failure occurs.

The problems use six distinct random geometry lattices for a domain of 80 × 1,100 × 250 mm, where supports are placed 50 mm from the end to yield an effective span of L = 1,000 mm. The six lattices differ in the location of the interface, which is near the load at l_1 = 490 mm, near the support at l_6 = 25 mm, and at four locations in between the load at midspan and the support (l_2 = 330 mm, l_3 = 190 mm, l_4 = 110 mm, l_5 = 50 mm). Beam properties are indicated in Table 3.20. Layers PCC1 and PCC2 were chosen to be equivalent to isolate the effects of the interface.

The value of f_t = 0.2 MPa was selected to provide a sufficiently weak interface. For the six cases described here, specimen failures in the two extremes of this problem are illustrated in Figure 3.47.

One would expect that the location of the weak interface has an influence on the load capacity and failure of the beam. Figure 3.48 illustrates the reduction in the ultimate load as the interface is moved from the support toward midspan. Note again that span length L = 1,000 mm.

The hypothesis being tested by the six cases is the expectation that as the interface was moved within the span, the nature of the initial fracture would shift from being largely tensile under midspan (as is commonly observed in three-point bending tests) to a mixed-mode. Thus, the analysis

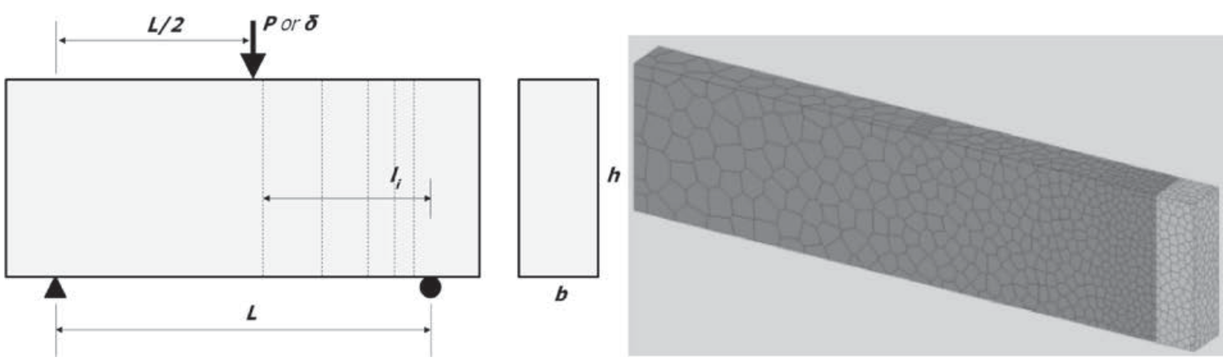


Figure 3.46. General composition of three-point beam (left). Composite beam with paired-node interface near support (right).

Table 3.20. Beam Properties for Multiple Interface Locations

	E (MPa)	f_t (MPa)	τ_c (MPa)	τ^* (MPa)	NFE	COBP (mm)	TFCO (mm)
PCC1	32000	4.0	6.0	2.80	100	0.002	0.008
PCC2	32000	4.0	6.0	2.80	100	0.002	0.008
Interface	32000	0.2	0.3	0.14	100	0.002	0.008

Note: τ^* = approximate shear strength under critical tensile stress; NFE = maximum allowable number of fracture events; COBP = crack opening at the break point; TFCO = traction-freecrack opening.

begins with $l_1 = 490$ mm, when the interface is almost directly beneath the controlled displacement at midspan. In this instance, we would expect the failure to be classical Mode I fracture. The first 1,000 fracture events are plotted against the fracture criterion curve from above (Figure 3.49).

Fracture events in Figure 3.49 are represented by a single black dot. The aggregate of these dots into one large mass illustrates that the initial fracture events are purely tensile in nature and in the neighborhood of 0.25 MPa, which exceeds the interface critical strength of $f_t = 0.2$ MPa. In this regard, the model simulation confirms the initial hypothesis. Only the initial (for these purposes, the first 1,000) events are plotted to make plots more legible and to further distinguish the separate cases, whose most striking differences in fracture are in initiation.

The interface is moved gradually toward the roller support, the beam loaded, and the nature of fracture recorded. The first 1,000 fracture events in each of the following four cases (where $l_2 = 330$ mm, $l_3 = 190$ mm, $l_4 = 110$ mm, and $l_5 = 50$ mm) are presented together in Figure 3.50.

Beginning with the case $l_2 = 330$ mm, we observe that the nature of the fracture is mostly tensile; however, some events feature a shear component as high as ~0.1 MPa. As we continue clockwise, the shear component becomes more pronounced and reaches values as high as ~0.3 MPa at $l_i = 950$ mm. The final case is the placement of the interface at $l_6 = 25$ mm. The first 1,000 fracture events for the final case are shown in Figure 3.51.

Figure 3.51 illustrates the observation that near the support, the shear component of fracture events is larger than the tensile events; shear stresses are nearly 0.4 MPa in this case. It is not until later events (only a few of which occur within the first 1,000) that normal (tensile) stresses exceed those of shear. Furthermore, the normal stresses for the first of the fracture events are compressive in nature. This phenomenon is most likely attributable to the proximity of the support, which acts to both compress and shear elements in the vicinity to instigate fracture.

The six cases support the concept of mixed-mode fracture. Although the idea is not novel given that the model determines the criterion, the simulations satisfy expectations of fracture behavior in particular situations: in Mode I (opening) fracture situations, the tensile strength contributes more to the determination of failure, whereas in predominately Mode II (in-plane

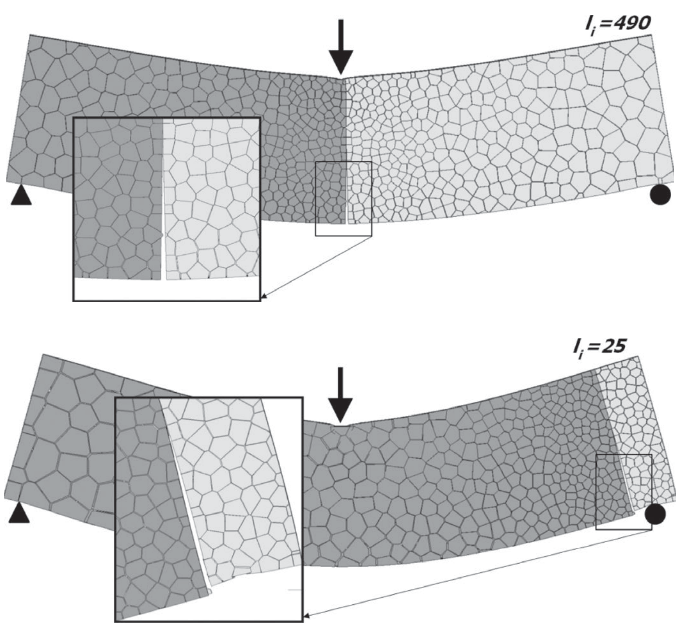


Figure 3.47. Failure in composite beams with weakened interface near midspan and near support under loading.

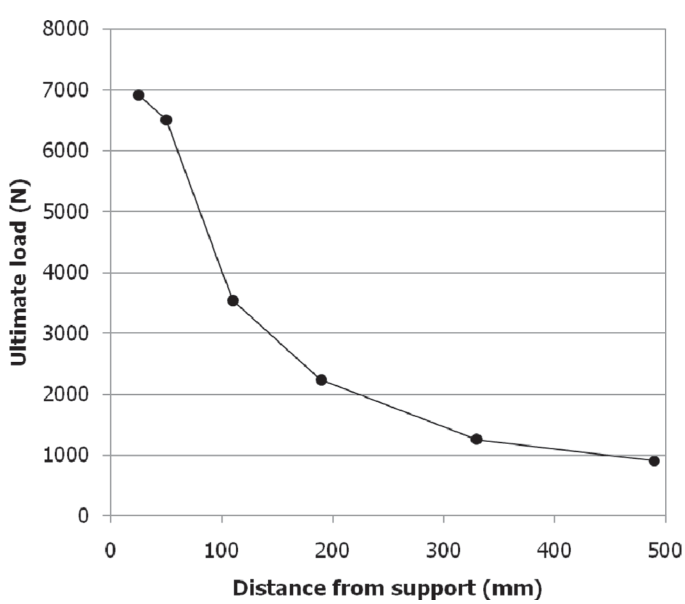


Figure 3.48. Reduction of ultimate load as weak interface is moved toward midspan.

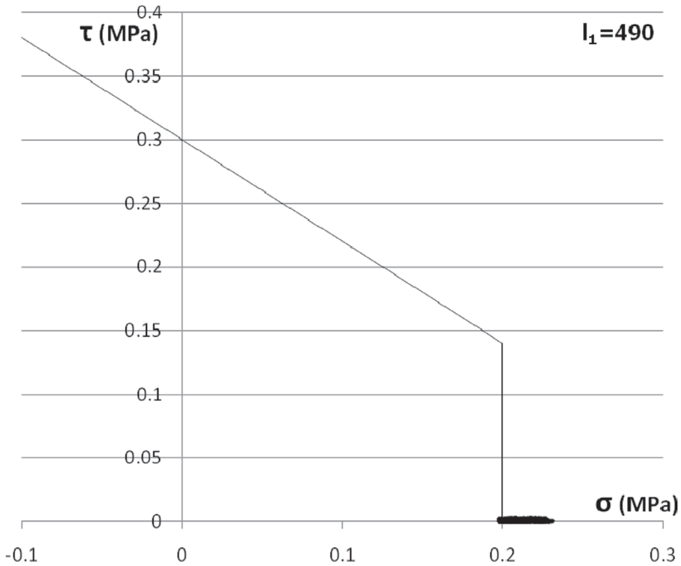


Figure 3.49. Failure in tension predominates at a weakened interface near midspan (l_1 = 490 mm).

shearing) fracture situations, the shear strength weighs more heavily in predicting failure. These capabilities in the combined lattice–FEM model for fracture in composite slabs give the ability to better simulate fracture behavior at the interface.

Slab Simulations to Characterize Debonding under Thermal Gradients

If the two lifts of a PCC/PCC pavement are constructed within a reasonable time frame (less than 2 hours), there will be no debonding at the PCC/PCC interface. This conclusion is supported by field observations of PCC/PCC composite pavements, from discussions with R21 project consultants in Europe, by ultrasound measurements of PCC/PCC at the MnROAD demonstration slab and mainline sections, and the pull-off tests conducted by the FHWA Mobile Concrete Laboratory.

However, we briefly consider the possibility that there is a possibility of unexpected delays in the placement of the upper PCC layer that is significantly greater than 2 hours.

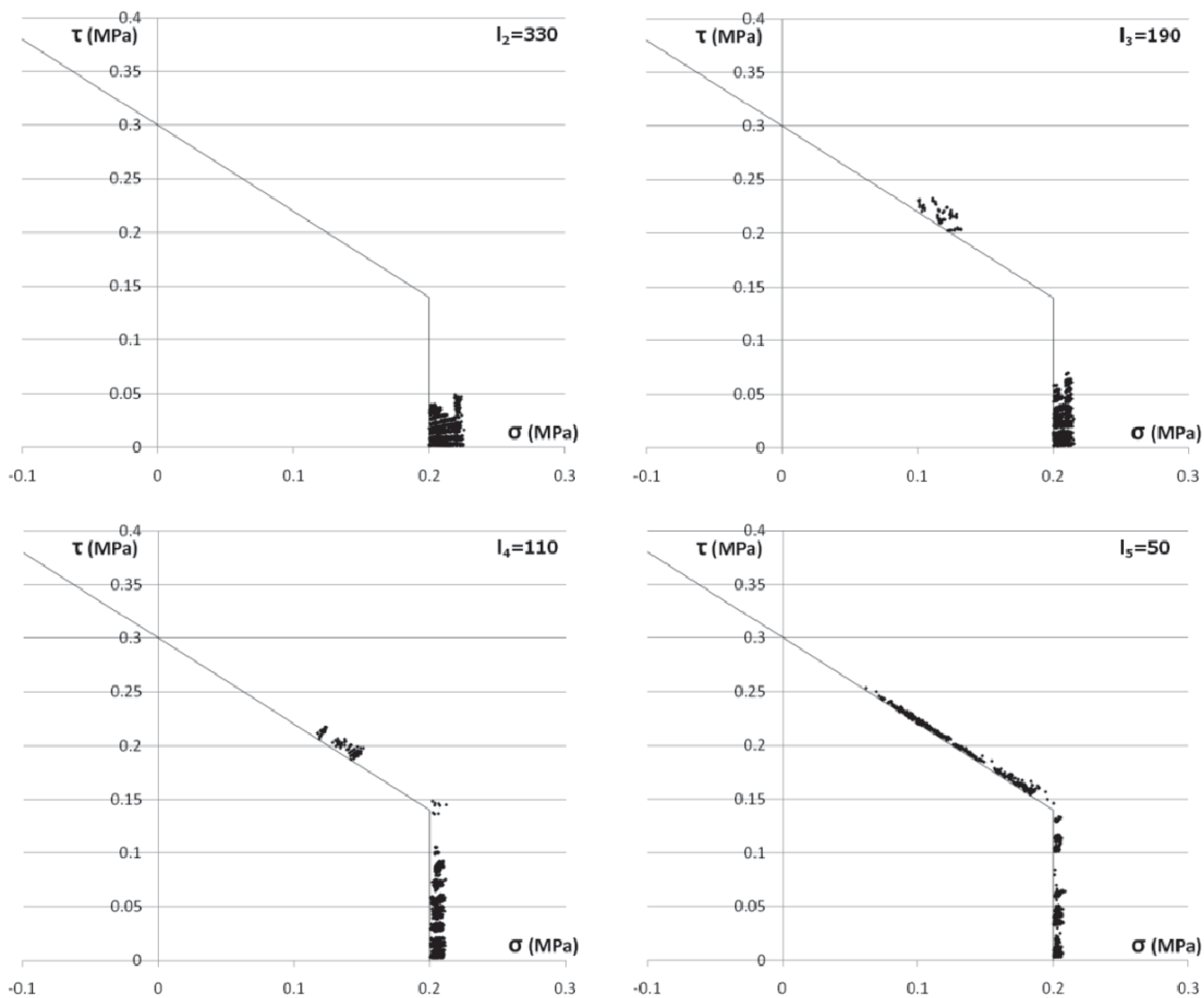


Figure 3.50. Evolution of mixed-mode fracture as interface is moved from midspan toward support (beginning* top left *with l_2 = 330 mm, moving clockwise, and finishing* bottom left *at l_5 = 50 mm).

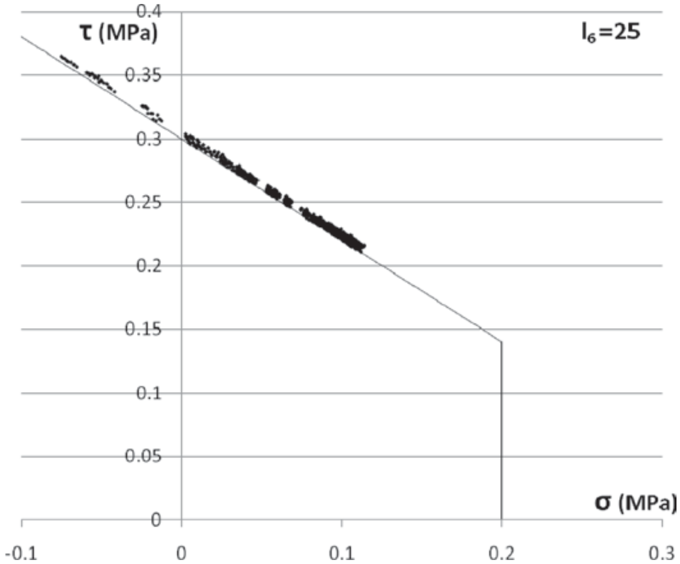


Figure 3.51. Shear events characterize mixed-mode failure at a weakened interface near support (l_6 = 25 mm).

If significant delays occur, full bond at the interface may be compromised. This may cause debonding resulting from differential (thermal and hygral) shrinkage strains in the upper and lower PCC layers. A coupled Lattice-ISLAB2005 model was used to simulate this situation.

The following simulation considers a lattice embedded in a composite slab undergoing differential shrinkage strains in the top PCC layer. This uses a bilinear temperature gradient, for which the upper layer of the composite slab experiences a negative temperature gradient while the lower layer experiences a constant temperature equivalent to the temperature at the bottom of the upper PCC layer. For this example, a half-slab with dimension 90 × 72 in. with finite element size 6 in. × 6 in. was used. The mesh for the plate is depicted in Figure 3.52.

ISLAB2005 input parameters are indicated below. Layer properties are adapted from those of the MnROAD PCC/PCC test section. The coefficient of thermal expansion (CTE) of the upper layer is exaggerated to develop a "worst case scenario," wherein the thermal properties of the two layers are vastly different. Note that this was not the case at MnROAD, as described in Chapter 2, where the measured CTE results for the two layers were nearly identical. Lattice properties for the two concretes in the composite slab are shown in Table 3.21, properties of the assumed interface are listed in Table 3.22, and the undeformed two-lift lattice is shown in Figure 3.53.

a. **ISLAB2005, Geometry**
- Custom mesh.
- *x*-direction.
  1. Length: 72 in.
  2. Number of nodes: 13 (equally spaced).
- *y*-direction.
  1. Length: 90 in.
  2. Number of nodes: 16 (equally spaced).

b. **ISLAB2005, Layers**
- Layer 1.
  1. Element type: Plate.
  2. Thickness: 3 in.
  3. Poisson's ratio: 0.15.
  4. Elastic modulus: 4,266,000 psi.
  5. Coefficient of thermal expansion: 7×10^{-6}/°F.

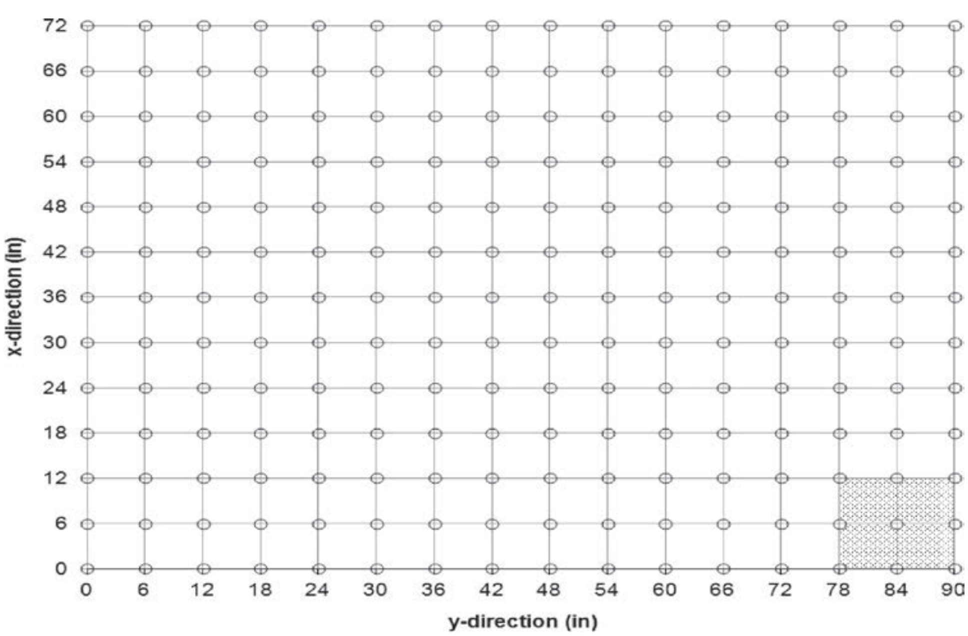


Figure 3.52. Finite element mesh for composite slab with location of lattice region shaded.

Table 3.21. Slab Properties for Two Lifts of PCC in Differential Strain Example

	E (psi)	f_t (psi)	τ_c (psi)	τ^* (psi)	NFE	CTE
PCC1 (upper)	4,266,000	725	1087.5	507.5	100	7E-06
PCC2 (lower)	3,827,000	627	940.5	438.9	100	5E-06

Note: τ^* = approximate shear strength under critical tensile stress; NFE = maximum allowable number of fracture events; CTE = coefficient of thermal expansion.

Table 3.22. Slab Properties for the Weakened Interface of Two Lifts of PCC in Differential Strain Example

Assumed Interface	*E* (psi)	f_t (psi)	τ_c (psi)	τ^* (psi)	NFE	CTE
100% of PCC2	3,827,000	627.00	940.500	438.900	100	5.00E-06
60% of PCC2	2,296,200	376.20	564.300	263.340	60	5.00E-06
40% of PCC2	1,530,800	250.80	376.200	175.560	40	5.00E-06
20% of PCC2	765,400	125.40	188.100	87.780	20	5.00E-06
15% of PCC2	574,050	94.05	141.075	65.835	15	5.00E-06
10% of PCC2	382,700	62.70	94.050	43.890	10	5.00E-06
5% of PCC2	191,350	31.35	47.025	21.945	10	5.00E-06

Note: τ^* = approximate shear strength under critical tensile stress; NFE = maximum allowable number of fracture events; CTE = coefficient of thermal expansion.

- Layer 2.
  1. Element type: Plate.
  2. Thickness: 6 in.
  3. Poisson's ratio: 0.15.
  4. Elastic modulus: 3,827,000 psi.
  5. Coefficient of thermal expansion: 5×10^{-6}/°F.

c. **ISLAB2005, Subgrade**
- Subgrade *k*-value: 200.

d. **ISLAB2005, Temperature (Case 1, −20°F in upper layer)**
- Layer 1.
  1. Type: Nonlinear.
  2. Reference: 65°F.

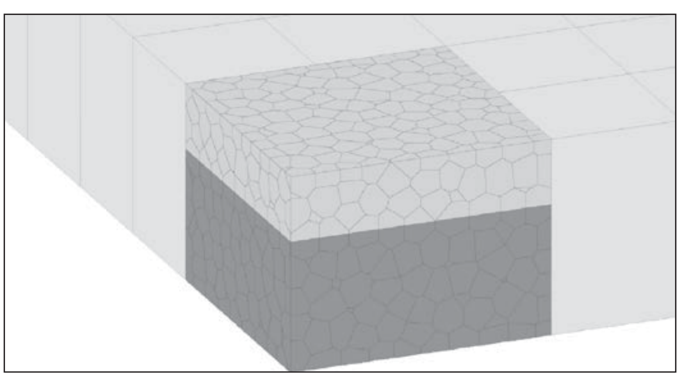


Figure 3.53. Undeformed lattice of the composite slab, located in the corner of the plate mesh.

  3. Node 1 (0 in.): 45°F.
  4. Node 2 (1.5 in.): 55°F.
  5. Node 3 (3 in.): 65°F.
- Layer 2.
  1. Type: Nonlinear.
  2. Reference: 65°F.
  3. Node 1 (0 in.): 65°F.
  4. Node 2 (3 in.): 65°F.
  5. Node 3 (6 in.): 65°F.

e. **ISLAB2005, Temperature (Case 2, −50°F in upper layer)**
- Layer 1.
  1. Type: Nonlinear.
  2. Reference: 65°F.
  3. Node 1 (0 in.): 15°F.
  4. Node 2 (1.5 in.): 40°F.
  5. Node 3 (3 in.): 65°F.
- Layer 2.
  1. Type: Nonlinear.
  2. Reference: 65°F.
  3. Node 1 (0 in.): 65°F.
  4. Node 2 (3 in.): 65°F.
  5. Node 3 (6 in.): 65°F.

The simulations of the composite slab used seven different assumed interfaces for the two temperature differences through the upper PCC lift, as shown in Table 3.22. These simulations considered the two cases of differential thermal

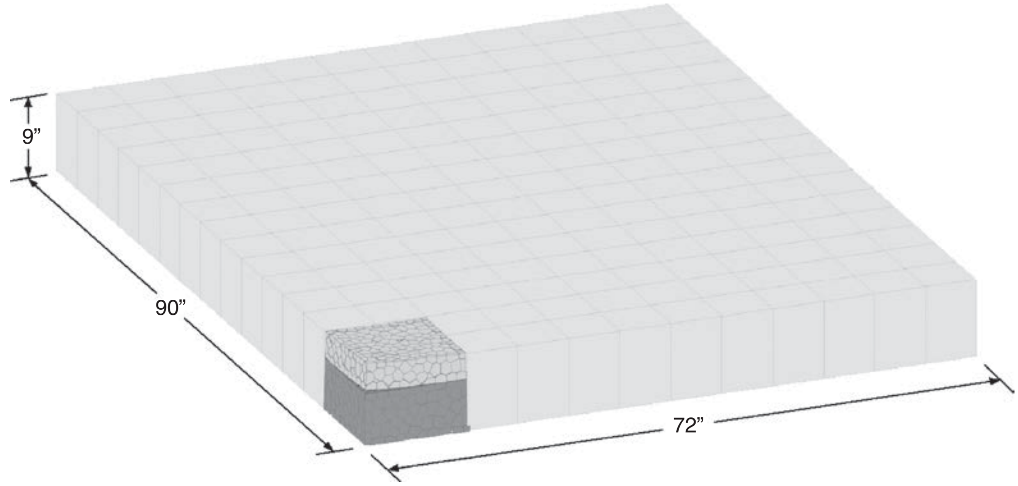


Figure 3.54. Debonded composite lattice in global slab view.

strain described in ISLAB2005 inputs: a −50°F temperature difference through the top layer and a −20°F temperature difference though the top layer. The −50°F is an extreme situation and is clearly unrealistic for an upper layer only 3 in. thick and even −20°F is unrealistic for only the top layer. However, these conditions were assumed to exaggerate the thermal differences between the two layers: to determine extremes for simulation and determine what interface properties are required for failure.

For the assumption of a −20°F temperature difference through the upper PCC layer, debonding at the interface did not occur in the simulations until the interface was degraded to 10% of the lower layer PCC (Table 3.22). This corresponds to the interface having flexural strength 62.7 psi and shear strength of 44 psi. The fracture and debonding that occurred are depicted in Figures 3.54 and 3.55. Displacements are scaled higher to depict debonding (fracture) behavior in Figures 3.54 and 3.55, which complicates depicting the deformation of the surrounding slab. The curl of the entire slab is more easily viewed using ISLAB2005, depicted in Figure 3.56.

Given that (1) measured shear bond strengths from MnROAD PCC/PCC laboratory specimens were well in excess of these levels, (2) CTE properties in the upper layer are overestimated to exaggerate thermal strains, and (3) thermal gradients through the slab are not as demanding as the bilinear gradient assumed, debonding in PCC/PCC appears to be an unlikely event, even in extreme cases.

For the assumption of a −50°F temperature difference through the upper lift, debonding at the interface did not occur in the simulated cases until the interface was degraded to 15% of the strength properties of the lower layer PCC (see Table 3.22). For the composite structure to bear this kind of extreme, unrealistic thermal load and differential strains in the PCC layers in these simulations once again suggests that debonding would require very special and unconventional circumstances that far exceed degraded interface strength properties, material differences in PCC layers, and extreme conditions.

Conclusions from Lattice Modeling

Overall, the lattice models support the conclusion reached in the field many decades ago by PCC/PCC practitioners and researchers in Europe: if the two lifts of a PCC/

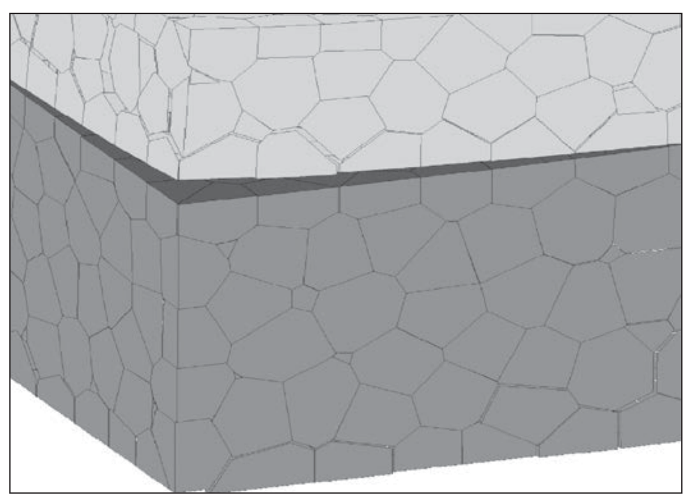


Figure 3.55. Local view of debonding at a degraded interface (10% strength properties of lower layer PCC) under −20°F temperature difference in upper PCC layer.

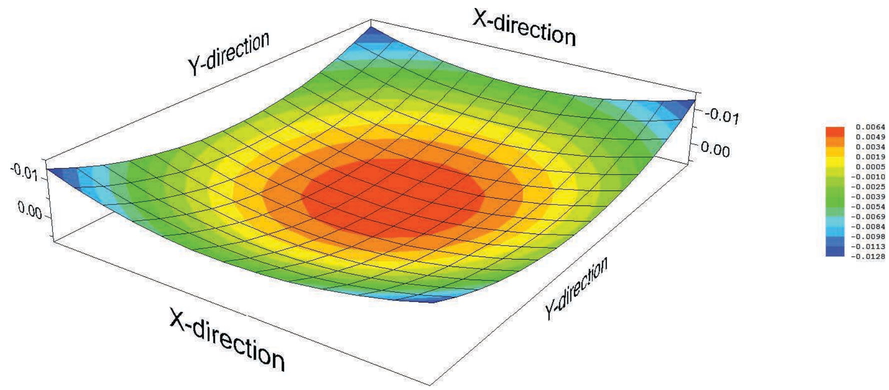


Figure 3.56. Slab curl in the simulated MnROAD PCC/PCC composite slab under −20°F temperature difference in upper PCC layer.

PCC pavement are constructed within a reasonable time frame there will be no debonding. Furthermore, the R21 2008 Survey of European Composite Pavements team was unable to locate field observations of PCC/PCC debonding with the assistance of project consultants in Europe. No PCC/PCC composite pavements surveyed in the United States exhibited any debonding either, even after 30 years for the Florida PCC/PCC sections. Finally, the pull-off tests conducted by FHWA Mobile Concrete Laboratory for the MnROAD sections confirmed that the bond strength is just as strong, if not stronger, than the strength of the lower PCC layer. Thus, debonding was determined to be a concern only in the case of PCC overlays of existing old PCC pavements, which is out of the scope of the SHRP 2 R21 project.

CHAPTER 4

PCC/PCC Design Guidelines

Note on Versions of the *MEPDG*

MEPDG version 1.014:9030A (03/11/2009) was the first build of *MEPDG* tailored to the R21 project. Results from 1.014:9030A initially were compared with *MEPDG* version 1.003, which was the public release build of the *MEPDG* for many years until late in the *MEPDG*'s life span (just before the release of DARWin-ME). In December 2010, a second build tailored to the SHRP 2 R21 project was released. This second build was designated as 1.206:R21. After continued analysis of this version of the *MEPDG* and its performance predictions for the factorial of R21 PCC/PCC cases, v. 1.3000:R21 was developed and released in May 2011. This was the final build tailored to the R21 project, and it is the last build examined by the R21 research team. Note that before DARWin-ME can be used to design PCC/PCC composite pavements in its present form (version 1.0.18), even using "Bonded PCC over JPCP", all of the R21 modifications must be made to the program because it may not produce correct results.

The revisions made to the *MEPDG* models included in the *MEPDG* (v. 1.3000:R21) were done in such a way as to not require any changes to the underlying *MEPDG* and EICM framework. These modifications were 100% compatible with the *MEPDG* and the EICM that existed before the R21 project: No additional coding was required for R21, only modifications to existing source code. The R21 recommended modifications have made the *MEPDG* more robust in terms of the different pavement types that it can assist engineers in designing and understanding.

Guidelines and Design Procedure Using AASHTO *MEPDG*

One of the major successes of the SHRP 2 R21 project is that the AASHTO *MEPDG* is able to be used for newly constructed PCC/PCC design and performance analysis with no impact on the way the user previously interacted with the *MEPDG*. The user needs only to accept the use of a "Bonded PCC Overlay of JPCP" project as a modeling simplification for the newly constructed PCC/PCC pavements addressed by the R21 research.

Once the user has selected a Bonded PCC Overlay of JPCP project, the remaining steps of the *MEPDG* design process to characterize the pavement system are identical to those for other rigid pavement projects. Here the user will characterize two PCC layers, and the remainder of the pavement input proceeds as in any other design. Design recommendations for use of R21-modified *MEPDG* (v. 1.3000:R21) to design PCC/PCC composite pavements are provided below:

1. **General use of *MEPDG* for composite pavements**
   a. Design Type: Select "PCC Overlay". As detailed above, this Project Type represents a PCC/PCC pavement for the purposes of design and analysis with the *MEPDG*.
   b. Pavement Type:
      - Select "Bonded PCC/JPCP".
      - Select "Bonded PCC/CRCP".
   c. Design Life: Select desired life of structural design until major rehabilitation is needed. Composite pavements are appropriate for a long structural life, exhibiting little structural deterioration over many years. *MEPDG* can design pavements for a design life as long as 100 years. To design a long-life pavement, select a life of more than 40 years. The PCC surface can be renewed as needed through diamond grinding, but the PCC slab will remain over the long design life with little structural fatigue damage.
2. **Trial Design: Select all design inputs for a trial design.** The inputs for a PCC/PCC composite pavement are as follows:
   a. Design reliability and performance for composite pavements:
      - Design reliability should be based on traffic level of the highway. Higher traffic levels warrant higher

reliability levels. For example, interstate, freeways, and divided highways warrant 95% to 99% and other highways and urban collectors/arterials warrant 90% to 94%. Local residential and farm-to-market roads warrant 75% to 89%.

- Structural fatigue cracking and punchouts:
  1. JPCP: 10% slabs (range: 5 to 15) transverse fatigue cracking.
  2. CRCP: 10 punchouts per mile (range: 5 to 15).
- Smoothness, Terminal International Roughness Index (IRI) should be based on traffic level of the highway. Higher traffic levels warrant lower terminal smoothness levels.
  1. Interstate, freeways, divided highways: 150 in./mi;
  2. Other highways and urban collectors/arterials: 160 in./mi; and
  3. Locals and farm-to-market: 175 in./mi.
- Joint faulting: 0.15 in.
- Initial IRI: The initial IRI for PCC/PCC composite pavements can be very low because of the multiple layering of the pavement. Initial IRI values as low as 50 in./mi have been achieved.

b. Type of PCC surface layer. The type depends on the design objectives.
   - If a design objective is to reduce noise levels to a minimum, then either an EAC can be used or special diamond grinding (such as the next-generation grind that showed the lowest noise level of all surfaces at MnROAD) of the PCC surface can be done.
   - If a design objective is durability against polishing and long life of the surface, then use of the highest quality, hard, nonpolishing aggregate possible is required. The next-generation grind may also be useful in this situation.
   - If neither of these objectives is applicable, a more conventional texturing of the surface that includes the hard aggregates would be sufficient.

c. Thickness of PCC surface layer. Layer thickness should be the minimum possible to provide durability and surface characteristics desired for a given truck traffic and climate. Thicknesses ranging from 1.5 in. to 3 in. have been built successfully.

d. Type (JPCP or CRCP) and thickness of the lower PCC layer. This is the load-carrying capacity layer for the composite pavement. The trial design should start with an estimate based on the total required for a one-layer slab minus the surface layer thickness. If the pavement is being designed to last for a long time period, during which multiple surface renewals (e.g., diamond grinding) will be needed, the designer may consider adding a small increment to the lower layer thickness design (e.g., 0.25 in./renewal).

e. Joint design for JPCP. Joint design includes joint spacing and joint load transfer.
   - Joint spacing is considered directly in the *MEPDG* analysis and affects transverse fatigue cracking as well as joint faulting. A shorter joint spacing requires a thinner slab to control fatigue cracking.
   - Joint load transfer requirement is similar to one-layer JPCP design in that dowels of sufficient size are required to prevent erosion and faulting for any significant level of truck traffic. The greater the dowel diameter, the higher the joint LTE and the more truck loadings the pavement can carry to the terminal faulting level.
     1. Simplified dowel design: the dowel diameter should be at least ⅛ the slab thickness. For example, a total PCC thickness of 12 in. requires a dowel diameter of at least $12/8 = 1.5$ in. For exceptionally heavy truck traffic highways, it may be necessary to add 0.25 in. diameter.
     2. Low-volume roadways where dowels would not normally be used for one layer JPCP do not require dowels for composite pavement.

f. Lower PCC layer recommendations. The formed concrete to be used for the lower layer of a PCC/PCC composite pavement can vary widely, as described here:
   - Typical concrete used in one-layer JPCP can be used with no changes. There are no special requirements different from those for one-layer pavement.
   - Lower cost concrete based on local aggregates or recycled concrete. The strength, modulus of elasticity, CTE, and drying shrinkage of the concrete can be varied as it is a direct input to the *MEPDG* software.
     1. The MnROAD experimental PCC/PCC clearly showed that properly recycled concrete from a local roadway can be used for the lower layer.
     2. The MnROAD experiment PCC/PCC also showed that a local aggregate source can be used successfully for the lower layer.
   - Both of these alternatives provide for substantial sustainability advantages and cost savings yet show adequate durability.
   - Certainly attention must be paid to good construction practices to locate dowels and tie bars properly and to saw all joints to the greater of one third of the total PCC thickness and the thickness of the upper layer plus ½ in.

g. Base layer and other sublayers should be selected that are similar to those used for one-layer JPCP designs, based on minimizing erosion, construction ease, and cost effectiveness. No attempts should be made to reduce the friction between the slab and the base. Good friction also helps control erosion and pumping and reduces stress in the slab.

3. ***MEPDG* Design Output Interpretation**
   a. Run the trial design using *MEPDG* (v. 1.3000:R21) and examine the outputs. Transverse fatigue cracking, IRI, and faulting must all meet the design reliability requirements for a trial design to be feasible.
   b. If any of these do not "Pass" at the reliability level, a modification in the design is required. Some guidelines for making modifications are as follows:
      - Excess transverse cracking: increase slab thickness, shorten joint spacing, add a tied PCC shoulder or 1-ft widened slab, use a stabilized base course, increase PCC strength (with appropriate change in the modulus of elasticity), or use a different aggregate source (one with lower CTE).
      - Excess IRI: reduce transverse cracking or require a smoother initial pavement. Two-layer PCC/PCC composite pavements can be constructed with exceptionally low initial IRI (e.g., 50 in./mi). Include incentive smoothness specifications with significant incentives so that the initial IRI is reduced. Smoothness incentives have been used with great success over several decades to improve initial IRI.

Illustrative Designs

PCC/JPC Composite Design for Interstate Highway: Albertville, Minnesota

a. Initial project detail
   - Design life: 20 years.
   - Design reliability: 90%.
   - Construction details.
     1. Existing pavement: May 2010.
     2. Pavement overlay: May 2010.
     3. Traffic open: June 2010.
   - Project type: bonded PCC over JPCP.

b. Analysis parameters
   - Initial IRI: 63 in./mi.
   - Rigid pavement analysis.
     1. Terminal IRI: 172 in./mi, 90% reliability.
     2. Transverse cracking: 15% cracked slabs, 90% reliability.
     3. Mean joint faulting: 0.12 in., 90% reliability.

c. Traffic
   - Two lanes in each direction.
   - Initial two-way average annual daily truck traffic (AADTT): 2,000 (directional distribution = 50%; lane distribution: 90%).
   - Growth rate: 4% compound (*MEPDG* default).
   - Use site-specific *MEPDG* defaults for other traffic inputs.

d. Climate
   - A virtual weather station was created for this site using the three closest stations.

e. Structure, design features
   - Permanent curl and warp efficiency temperature difference: −10°F.
   - Joint design: joint spacing, 15 ft; dowel diameter, 1.25 in.; dowel spacing: 12 in.
   - Base properties: erodibility index, 3 = erosion resistant; PCC-base interface, full friction contact; loss of friction, 360 months.

f. Structure, Layer 1 PCC properties
   - General: thickness, 3 in.; unit weight, 150 lb/ft^3; Poisson's ratio, 0.2.
   - Thermal specification: CTE, 5.5×10^{-6}/°F; thermal conductivity, 1.25 Btu/[(ft)(hr)(°F)]; heat capacity, 0.28 Btu/[(lb)(°F)].
   - Mix design: cement content, 675 lb/ft^3; water-cement ratio: 0.42; coarse aggregate type: basalt.
   - Strength properties: 28-day modulus of rupture, 750 psi.

g. Structure, Layer 2 PCC properties
   - General: thickness, 6 in.; unit weight, 150 lb/ft^3; Poisson's ratio, 0.2.
   - Thermal specification: CTE, 5.5×10^{-6}/°F; thermal conductivity, 1.25 Btu/[(ft)(hr)(°F)]; heat capacity, 0.28 Btu/[(lb)(°F)].
   - Mix design: cement content, 550 lb/ft^3; water-cement ratio, 0.42; coarse aggregate type, chert.
   - Strength properties: 28-day modulus of rupture, 625 psi.

h. Structure, base/subbase properties
   - Type: granular, A-1-a.
   - Thickness: 8 in.
   - Strength properties: modulus of elasticity, 40,000 psi.

i. Structure, subgrade properties
   - Type: soil, A-6.
   - Thickness: semi-infinite.
   - Strength properties: modulus of elasticity, 14,000 psi.

j. Rigid rehabilitation inputs
   - Existing distress: before, 0; after, 0.
   - Foundation support: dynamic modulus of subgrade reaction, not available.

According to *MEPDG* performance predictions, the PCC/PCC constructed at Albertville, Minnesota, will perform well within its 20-year design life, as illustrated in Table 4.1. Furthermore, the performance predictions suggest it will perform adequately well beyond the specified 20-year design life.

Table 4.1. Performance Predictions for PCC/JPC Constructed at Albertville, Minnesota, at 90 Percent Reliability

Location	IRI in./mi (Limit: 172)	Percent Slabs Cracked (Limit: 15%)	Mean Joint Faulting, in. (Limit: 0.12)	Bottom-up Cracking Damage	Top-down Cracking Damage
Albertville, Minnesota	121.9	2.2	0.052	0.139	0.044

MEPDG Design Comparisons

The following subsections describe the development of composite PCC/PCC projects in *MEPDG* (v. 1.3000:R21), based on sections in the R21 PCC/PCC database, and determine a single-layer JPCP alternative for these pavements. Each of the following comparisons is for 20 years of service, and the projects being compared differ only in the composition of the concrete slab: either it is a single, homogeneous PCC lift (JPCP) or it is two heterogeneous PCC lifts (PCC/JPC). PCC properties of the slabs differ as indicated below.

PCC/PCC Composite Design for Interstate Highway, Albertville, Minnesota (MnROAD Test Section)

a. Design reliability and performance requirements
   - Design life: 20 years.
   - *R:* 90%.
   - Transverse slab cracking: 15% maximum.
   - Transverse joint faulting: 0.12-in.
   - IRI: 172 in./mi (initial IRI assumed 63 in./mi).
b. Materials
   - Upper PCC: cement content 675 lb/yd^3, and 28-day flexural strength is 750 psi.
   - Lower PCC: cement content 550 lb/yd^3, and 28-day flexural strength is 625 psi.
   - Aggregate base course: AASHTO A-1-a is used to simulate Minnesota DOT Class 5 aggregate; *MEPDG* defaults are used for base material properties.
c. Site conditions
   - Traffic.
     1. Two lanes in each direction
     2. Initial two-way AADTT: 2,000 (directional distribution = 50%, lane distribution = 90%).
     3. Growth rate: 4% compound.
     4. Use site-specific *MEPDG* defaults for other traffic inputs.
   - Subgrade: assume subgrade soil A-6 and other *MEPDG* defaults.
   - Climate: a virtual weather station was created for this site using the three closest stations.
d. Trial composite design
   - Upper PCC: 3-in. thickness.
   - Lower PCC: 6-in. thick JPCP layer with a 15-ft joint spacing and dowel diameter of 1.25-in.
   - Base: 8-in. thickness placed directly on the fine-grained, prepared and compacted subgrade.
e. Output results for composite design
   - Total number of trucks in design lane over 20 years: 10.3 million.
   - Transverse cracking of JPCP: $R > 90\%$, Pass.
   - IRI: $R > 90\%$, Pass.
   - Faulting: $R > 90\%$, Pass.
f. Final composite design
   - 3-in. PCC upper lift.
   - 6-in. JPCP (containing recycled PCC from existing roadway).
   - 8-in. dense-graded aggregate base.

   This design passes all of the requirements for slab fatigue transverse cracking, faulting (and thus good joint LTE), and IRI. The PCC surface may need to be rehabilitated after 10 to 20 years because of various weathering problems that occur in this harsh climate.
g. Comparative single-layer JPCP design

   Given the design obtained for the composite pavement, what would be an equivalent design for a single-layer JPCP at this location and for these traffic levels? The *MEPDG* was run for a single-layer JPCP with PCC properties of cement content 550 lb/yd^3 and 28-day flexural strength of 650 psi; all other inputs were identical to the composite design. The JPCP thickness design shown in Table 4.2 was required.

Table 4.2. Equivalent PCC/PCC and Single-layer JPCP Designs Modeled Using MEPDG *for MnROAD Test Section*

Design	PCC/PCC Pavement	JPCP
PCC Surface	3-in. PCC	None
JPCP	H = 6 in. Dowels = 1.25 in.	H = 8.75 in. Dowels = 1.25 in.
Base	8-in. Untreated aggregate	8-in. Untreated aggregate
Reliability	>90%	>90%

Note: *MEPDG* (v. 1.3000:R21); H = PCC thickness.

For this level of reliability, the *MEPDG* suggests a minimum of 8.75-in. JPCP thickness to function as an equally performing single-layer alternative to the PCC/PCC. Given that there are cost savings both in terms of materials used in the PCC/PCC layers and over the course of the life span of the pavement, the PCC/PCC pavement is a viable option in situations in which quality aggregates are in short supply, supplementary cementitious materials (SCMs) are available for high levels of cement replacement, or any other readily available alternative materials can be used in the lower lift of the PCC/PCC. In addition, the comparison here does not consider the benefits in durability and texturing options offered by the high-quality surface PCC. Note that the thickness of the JPCP is lower than the total thickness of the two layers of the PCC/PCC composite pavement. The difference arises from the material properties of the JPCP relative to the lower PCC layer (especially PCC flexural strengths: 650 psi for JPCP versus 625 psi for the lower lift of the PCC/PCC).

PCC/PCC Composite Design for Interstate Highway, Abilene, Kansas

a. Design reliability and performance requirements
   - Design life: 20 years.
   - *R:* 90%.
   - Transverse slab cracking: 15% maximum.
   - Transverse joint faulting: 0.12-in.
   - IRI: 172 in./mi (initial IRI assumed 63 in./mi).

b. Materials
   - Upper PCC: cement content 650 lb/yd^3 and 28-day flexural strength is 750 psi.
   - Lower PCC: cement content 548 lb/yd^3 and 28-day flexural strength is 620 psi.
   - Aggregate base course: cement stabilized granular base, *MEPDG* defaults used for base material properties.

c. Site conditions
   - Traffic
     1. Two lanes in each direction.
     2. Initial two-way AADTT: 4,000 (directional distribution = 50%, lane distribution = 90%).
     3. Growth rate: 4% compound.
   - Use site-specific *MEPDG* defaults for other traffic inputs.
   - Subgrade: assume subgrade soil A-6 and other *MEPDG* defaults.
   - Climate: a virtual weather station was created for this site using the three closest stations.

d. Trial composite design
   - Upper PCC: 1.5-in. (40-mm) thickness.
   - Lower PCC: 11.8-in. (300-mm) JPCP layer with a 15-ft joint spacing and dowel diameter of 1.5 in. (40 mm).
   - Base: 6-in. thickness. This will be placed directly on the fine-grained, prepared and compacted subgrade.

e. Output results for composite design
   - Total number of trucks in design lane over 20 years: 20.7 million.
   - Transverse cracking of JPCP: $R > 90\%$, Pass.
   - IRI: $R > 90\%$, Pass.
   - Faulting: $R > 90\%$, Pass.

f. Final composite design
   - 1.5-in. PCC upper lift.
   - 11.8-in. JPCP (containing recycled PCC from existing roadway).
   - 6-in. cement treated granular base.

   This design passes all of the requirements for slab fatigue transverse cracking, faulting (and thus good joint LTE), and IRI.

g. Comparative single-layer JPCP design

   Given the design obtained for the composite pavement, what would be an equivalent design for a one-layer JPCP at this location? The *MEPDG* was run for a single-layer JPCP with PCC properties of cement content 525 lb/yd^3 and 28-day flexural strength is 650 psi. All other inputs are identical to the composite design. The JPCP thickness design shown in Table 4.3 was required.

For this level of reliability, the *MEPDG* suggests a minimum of 13-in. JPCP thickness to function as an equally performing single-layer alternative to the PCC/PCC. Given that there are cost savings both in terms of materials used in the PCC/PCC layers and over the course of the life span of the pavement, the PCC/PCC pavement is a viable option in situations in which quality aggregates are in short supply, SCMs are available for high levels of cement replacement, or any other readily available alternative materials can be used in the lower lift of the PCC/PCC. In addition, the comparison here does not consider the benefits in durability and texturing options offered by the high-quality surface PCC. Note that the thickness of the JPCP is lower than the total thickness of the two layers of the PCC/PCC composite pavement. The difference arises from the material properties of the JPCP relative to the lower PCC layer (especially PCC flexural strengths: 650 psi for JPCP versus 620 psi for the lower lift of the PCC/PCC).

Table 4.3. Equivalent PCC/PCC and Single-layer JPCP Designs Modeled Using* MEPDG *for Interstate Highway in Abilene, Kansas

Design	PCC/PCC Composite	JPCP
Surface	1.5-in. PCC	None
JPCP	H = 11.8 in. Dowels = 1.5 in.	H = 13 in. Dowels = 1.5 in.
Base	8-in. Untreated aggregate	8-in. Untreated aggregate
Reliability	>90%	>90%

Note: *MEPDG* (v. 1.3000:R21); H = PCC thickness.

Table 4.4. Sensitivity of Pavement Performance to Flexural Strength of the Upper PCC Layer Using MEPDG

Flexural Strength of Upper PCC Layer (psi)	IRI, in./mi (Limit: 172)	Percent Slabs Cracked (Limit: 15)	Mean Joint Faulting, in. (Limit: 0.12)	Bottom-up Cracking Damage	Top-down Cracking Damage
650	68.6	0.4	0.006	0.0427	0.0439
300	88.7	100	0.007	0.0433	359.0657

Note: *MEPDG* (v. 1.3000:R21).

Sensitivity Analysis

Influence of Upper-lift PCC Flexural Strength in PCC/PCC Performance

The upper-lift PCC flexural strength was varied to determine what effect it would have on performance of the PCC/PCC composite pavement. The only parameter changed was the flexural strength and corresponding moduli of elasticity because these are inherently related PCC properties. All other parameters were kept constant across all trials. A performance comparison of two PCC/PCC composite pavements with upper-layer flexural strength varying from 650 psi to 300 psi is presented in Table 4.4. It was expected that a decrease in the modulus of rupture of the upper PCC layer would lead to an increase in predicted top-down cracking without much effect on the predicted faulting or bottom-up cracking. These results meet expectations in that the top-down cracking fatigue damage increases dramatically when the flexural strength of the upper lift of the PCC/PCC is reduced.

Influence of Lower-Lift PCC Flexural Strength in PCC/PCC Performance

The lower-lift PCC flexural strength was varied to determine what effect it would have on performance of the PCC/PCC composite pavement. The only parameter changed was the flexural strength and its corresponding moduli of elasticity because these are inherently related PCC properties. All other parameters were kept constant across all trials. A performance comparison of two PCC/PCC composite pavements with lower-layer flexural strength varying from 650 psi to 300 psi is presented in Table 4.5. It was expected that a decrease in the modulus of rupture of the lower lift PCC would lead to an increase in predicted bottom-up cracking without much effect on the predicted faulting or top-down cracking. Once again, the results meet expectations as the bottom-up cracking fatigue damage increases dramatically because of the reduction in flexural strength of the lower-lift PCC.

Influence of CTE in PCC/PCC Pavement Performance

CTE was varied from 5.3×10^{-6}/°F to 10.0×10^{-6}/°F in the upper-layer PCC, and the results were compared as shown in Table 4.6. The expectation that accompanied this modification is that performance in all aspects would become worse. The performance predicted because of the change in the CTE met expectations.

Influence of Modulus of Elasticity in PCC/PCC Pavement Performance

The PCC modulus of elasticity was reduced from 4,000,000 to 2,000,000 psi in both the upper- and lower-layer PCC and compared. The expectation that accompanied this modification was that faulting in a doweled pavement is improved less by a decrease in the elastic modulus of the upper layer than it would be improved by a decrease in the elastic modulus of a thicker lower layer. This is because in a PCC pavement, reduced stiffness improves the ability of the dowels to transfer a load from one slab to the other. For these trials, the upper and lower PCC thicknesses are 3 and 6 in., respectively, but the sections were modified to use reduced dowel thickness, increased dowel spacing, and a higher base erodability to exaggerate faulting. The comparison is presented in Table 4.7.

Table 4.5. Sensitivity of Pavement Performance to Flexural Strength of the Lower PCC Layer Using MEPDG

Flexural Strength of Lower PCC Layer (psi)	IRI, in./mi (Limit: 172)	Percent Slabs Cracked (Limit: 15)	Mean Joint Faulting, in. (Limit: 0.12)	Bottom-up Cracking Damage	Top-down Cracking Damage
650	68.6	0.4	0.006	0.0427	0.0439
300	151	100	0.007	97.6881	0.0439

Note: *MEPDG* (v. 1.3000:R21).

Table 4.6. Sensitivity of Pavement Performance to CTE of the Upper PCC Layer Using MEPDG

CTE of Upper PCC Layer (10^{-6}/°F)	IRI, in./mi (Limit: 172)	Percent Slabs Cracked (Limit: 15)	Mean Joint Faulting, in. (Limit: 0.12)	Bottom-up Cracking Damage	Top-down Cracking Damage
5.5	68.6	0.4	0.006	0.0427	0.0439
10	153.7	100	0.012	0.0045	69.8954

Note: *MEPDG* (v. 1.3000:R21).

For doweled pavements, reductions in the modulus of elasticity for either PCC layer result in a relatively minor improvement in overall faulting, so the performance in faulting predicted by *MEPDG* met expectations. This trial is one of many reasons the JPCP faulting model was not modified because the existing model appears to capture composite slab faulting behavior adequately. Furthermore, this trial shows that doweling is as important to the performance of composite pavements as it is to conventional JPCP. Table 4.7 also shows that changing the modulus of elasticity of a layer affects the PCC damage and amount of cracking. Lowering the modulus of the surface increases the amount of cracking. Thus, a higher modulus for the upper-layer PCC is desirable, which will always be the case for PCC/PCC composite pavements, as defined for this SHRP 2 R21 project.

PCC Surface Material and Texture Design Options

The designer has several options to consider for the thin top lift of PCC/PCC and the type of texture to be performed. These options are listed as follows with their advantages and disadvantages.

Option 1: EAC Mixture and Exposed Aggregate Surface

This is a PCC material conforming to a high quality EAC mixture that includes hard nonpolishing aggregates. The surface texture would be an exposed aggregate surface. This is the traditional European PCC/PCC composite pavement design that has performed very well over 20 years. Advantages and disadvantages of this approach are as follows:

- *Advantages:* The EAC mixture is very durable because no deterioration was observed in cold climate Europe for as long as 20 years with this layer under heavy traffic. The EAC mixture does not polish significantly, even in heavy snow and icy highways. When the mixture is properly designed and constructed, the noise level is relatively low and comparable with dense-graded HMA, and the surface remains smooth.
- *Disadvantages:* The brushing technique can be challenging to achieve the proper texture depth but can be mastered by construction crews. The construction process is more complicated than that for conventional concrete and requires greater effort to perform all tasks properly.

Option 2: High-Quality PCC with Highly Durable Aggregate Concrete and Conventional Texturing or Diamond Grinding

This is a PCC material conforming to a high-quality PCC mixture that includes hard, nonpolishing aggregates. The surface texture could be conventional texturing or diamond grinding of some type. Advantages and disadvantages of this approach are as follows:

- *Advantages:* The high quality PCC mixture would be very durable. The diamond grinding or the conventional texturing would polish less than with softer aggregates and should have a long life, even for highways with heavy snow and

Table 4.7. Sensitivity of Pavement Performance to PCC Elastic Modulus Using MEPDG

Elastic Modulus of Upper PCC Layer (psi)	Elastic Modulus of Lower PCC Layer (psi)	IRI, in./mi (Limit: 172)	Percent Slabs Cracked (Limit: 15%)	Mean Joint Faulting, in. (Limit: 0.12)	Bottom-up Cracking Damage	Top-down Cracking Damage
4,000,000	4,000,000	90.0	0.4	0.047	0.0433	0.0438
2,500,000	4,000,000	94.6	8.3	0.043	0.0119	0.2968
4,000,000	2,500,000	85.5	0.0	0.039	0.0076	0.0000

Note: *MEPDG* (v. 1.3000:R21).

ice. The noise level is relatively low for the diamond grind (and next-generation grind is even lower, as demonstrated at MnROAD). The high-quality PCC top lift and either diamond grinding or conventional texturing is relatively easy to construct.
- *Disadvantages:* The conventional textured surface (even longitudinal tining) would have good but not exceptional low noise characteristics as compared with the diamond grinding or EAC texture. The diamond grinding texture (particularly the next-generation grind) may be expensive because of the hardness of the aggregate.

Option 3: Normal-Quality PCC and Conventional Texturing or Diamond Grinding

This is a PCC material conforming to a conventional quality PCC mixture used for PCC paving. The surface texture could be conventional texturing or diamond grinding of some type. Advantages and disadvantages of this approach are as follows:

- *Advantages:* The only advantage is that the top lift would be the typical conventional mixture used in a state and thus would be lower in cost than the EAC or high-quality PCC layer described above. The surface could be conventionally textured or diamond ground (either conventional or next-generation). The conventional PCC top lift and either diamond grinding or conventional texturing are relatively easy to construct.
- *Disadvantages:* The conventional textured surface (even longitudinal tining) would have good but not exceptional low noise characteristics, as would diamond grinding or EAC texture. In harsh climates, its durability would not be as good as the EAC or high-quality concrete and aggregate surfaces.

Cost Analysis and Pavement Type Selection

PCC/PCC composite pavements have not been constructed widely in the United States. Therefore, guidelines on pavement type selection and cost analysis will be helpful to state highway agencies and others. When engineers and contractors in the Netherlands, Germany, and Austria were asked the question "Are PCC/PCC pavements cost effective in your country," they all responded "yes they are" and explained why. The main reason was that high-quality aggregates are very expensive, and if their use can be restricted to the top 2- to 3 in. of a PCC pavement and use lower-cost aggregates in the thicker lower portion of the PCC pavement, there could be significant savings to pay for the additional manpower and equipment needed for PCC/PCC construction.

Although the performance of these PCC/PCC composite pavements has been excellent, particularly in Europe, few agencies in the United States typically consider them in their pavement selection procedures. This may be because of the perception that they are more expensive to build than conventional PCC pavements. However, given the need to consider pavement alternatives that not only have long-term structural load-carrying capacity (e.g., long life) but also have long-term excellent surface characteristics, a competitive life-cycle cost, and can be rapidly rehabilitated in the future as needed, the interest in and use of PCC/PCC composite pavements may increase at both state and local highway agencies.

The information and technology assembled and developed under the SHRP 2 R21 project gives highway agencies much additional information related to PCC/PCC composite pavements:

1. Performance of this type of composite pavement on interstates and other major highways: The PCC/PCC composite pavement could certainly be used on lower-volume highways or urban streets for long life pavement with major sustainability benefits if the costs were competitive.
2. Validation of a rational mechanistic-based AASHTO design procedure (e.g., *MEPDG* v. 1.3000:R21).
3. Construction guidelines and recommendations for building quality composite pavements.

This section of the report provides recommendations for pavement selection procedures and life-cycle cost analysis (LCCA) of PCC/PCC pavements. This information will aid highway agencies in including composite pavements in their routine pavement selection process and conducting the LCCA process properly. NCHRP Report 703 (Hallin et al. 2011) is recommended as a good process for addressing the selection process and the LCCA of composite pavements. Below is a step-by-step process that focuses on PCC/PCC composite pavements and closely follows the NCHRP report recommendations.

Step 1 Establish LCCA Framework

- *Analysis period.* The analysis period for PCC/PCC composite pavements should reflect the time over which the highway agency wants the pavement to perform without major structural damage (e.g., transverse fatigue cracking in PCC/JPC or punchouts in PCC/CRC) at a desired level of reliability. The surface of a high-quality EAC or a diamond-ground, high-quality PCC surface should last well beyond 20 years. Thus, the longer a PCC/PCC composite pavement is designed to exhibit low structural damage, the more cost-effective and sustainable it will likely be because small increases in structural design capacity result in long-term extension of fatigue damage (e.g., slightly thicker lower slab). The design could be made for only 20 years; however, this would result in reduced cost competitiveness and reduced sustainability benefits. Thus, a structural life of 40 years or more is recommended. The PCC/PCC composite design becomes essentially a long-life pavement with rapid surface renewal at intermediate times. One PCC/JPC composite pavement located on US-45 in Florida was constructed in 1978 and received no surface rehabilitations (grinding or overlay) or structural repairs for 30 years, but at 30 years, it was in need of a rapid renewal through diamond grinding. This pavement exhibited almost no transverse fatigue cracks after carrying 8 million heavy trucks in the outer lane without a single slab replacement. The ideal analysis and design period for this project should have been about 40 to 50 years because it will not exhibit significant structural distresses and likely will need only one surface retexturing during that time period.

- *Discount rate.* Long-term real discount rate values provided in the latest edition of the Office of Management and Budget (OMB) Circular A-94, Appendix C, should be used.
- *Economic analysis technique.* The net present value (NPV) method using constant or real dollars and a real discount rate in NPV computations is recommended.
- *LCCA computation approach.* There are two approaches to NPV analysis: deterministic and probabilistic. Either one could be used for PCC/PCC composite pavements. Estimation of the variabilities involved in the probabilistic approach is a major challenge. In either approach, the service life must be estimated with sufficient accuracy and also the standard deviation and distribution must be estimated for the probabilistic approach. Recommendations for PCC/PCC composite pavement:
  1. *Service life (PCC layer structural life).* As part of this SHRP 2 R21 project, PCC/JPC composite pavements were analyzed and found that they fit into the nationally calibrated *MEPDG* fatigue damage models for JPCP (using v. 1.3000:R21) so that the prediction of structural life until the terminal level of cracking is reached can be obtained from the software output. The mean 50% prediction curve should be used as the life estimate when it crosses the critical transverse cracking limit. If this does not occur during the analysis period, the slab should be considered as having a long life and would have a significant remaining life at the end of the analysis period.
  2. *Service life (PCC top layer functional life).* Chapter 2 included several examples of numerous EAC pavements in Europe and the Florida conventional PCC upper layer that have performed from 20 to more than 30 years without renewal. There are two main performance indicators that would result in terminal life for the thin PCC type of surface.
     - Wearing of the surface in wheelpath from studded tires or chains: The long A1 motorway across Austria is subject to harsh winter ice and snow but did not show much polish or wear over 18 years. Thus, the hard durable aggregates in the surface course are expected to last well over 20 years.
     - Raveling: This must be estimated by the local highway agency because it cannot be predicted. Very little of this was observed on the EAC surfaces over many years.
  3. *Survival curves.* Often the best way to estimate the mean service life of a pavement is to use survival analysis. Unfortunately, there are not enough older PCC/PCC pavements constructed with sufficient survival history to do that at this time. However, additional construction and monitoring of their performance will allow that in the future. The 50th percentile should be used as the mean life and the standard deviation for the probabilistic approach.

Step 2 Estimate Initial and Future Costs

- *Initial construction costs.* This includes the cost of the pavement structure, including the PCC top layer considering the selected form of texturing (e.g., EAC, diamond grinding, conventional texture), PCC bottom layer, and base and other embankment layers. Information from the Kansas I-70 project with all of the above mentioned types of surface texturing is available for consideration. A cost comparison example using the MnROAD data is provided later in this chapter.
- *Future rehabilitation and maintenance costs.* Future costs include some routine maintenance and the rapid renewal of the PCC surfacing through some type of diamond grinding. New and innovative techniques to be developed into the future will make this operation even more cost effective and produce even better surface characteristics.
- *Salvage costs.* Estimated cost of the pavement at the end of the design analysis period.
- *Initial and future highway (extra) user costs.* There are several components of extra highway user costs. The word "extra" is used to indicate that these are in excess of those obtained for smooth pavements because of increased roughness, accidents, and lane closures for maintenance and rehabilitation. The FHWA RealCost program includes estimates of most of these extra costs and reasonable procedures to estimate them. The timing of the surface rehabilitation and its duration must be estimated.
- *Develop expenditure stream diagrams.* Basically, for PCC/PCC composite pavements, there will be the initial construction, future routine maintenance, and future rehabilitation of the surface layer (typically removal and replacement with a better product at the time). An example is shown in Table 4.8, where the design analysis period is 40 years and, given the climate and traffic, a 20-year life is expected for the PCC surface layer texture. Some user delay and other user costs are expected every time lane closures are programmed, even if they are done during off peak traffic hours.

Table 4.8. Example of Expenditure Stream Table for Performing LCCA

Time, Years	0	5	10	15	20	25	30	35	40
User (U) $ Delay, etc.	$ 0	$ 0	$ U	0	$ U	$ 0	$ U	$ U	$ 0
Maintenance (M) $ Routine	$ 0	$ 0	$ M	0	$ M	$ 0	$ M	$ M	$ 0
Renewal (R) $ Surface Layer	$ 0	$ 0	$ 0	$ 0	$ R	$ 0	$ 0	$ 0	$ 0
Initial (I) $ Salvage (SAL) $	$ I	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ SAL

Step 3 Compute Life-Cycle Cost Analysis

The FHWA's RealCost EXCEL spreadsheet software is convenient and efficient for entering all of the above costs and properly computing an NPV cost estimate for a composite pavement. RealCost also will compute highway user's cost for project conditions. RealCost can use deterministic and probabilistic approaches for conducting the LCCA.

Step 4 Select Preferred Pavement Alternative

A composite pavement can be compared directly with conventional HMA or PCC pavement alternatives in terms of costs (e.g., NPV) and noneconomic selection factors. Although the NPV can be computed from all of the associated direct and indirect costs, it can be evaluated separately as follows:

- Initial construction cost;
- Highway user's costs during initial construction; and
- Future direct cost to highway agency for lane closures such as
  - Maintenance;
  - Rehabilitation; and
  - Salvage.
- Future highway user's costs during maintenance and rehabilitation lane closure activities; and
- Total costs NPV.

The non-economic factors are important and include the following, based on NCHRP Report 703. In some cases, a composite pavement has advantages over conventional asphalt or concrete.

- *Roadway/lane geometrics:* PCC/PCC composite pavement would be similar to any other PCC pavements in terms of dealing with lane widths, shoulders, turning movements, and so forth, but may have some advantages with respect to some conventional HMA with regard to total thickness of all pavement layers.
- *Continuity of adjacent pavements:* If this is desired, then wherever the adjacent pavement is PCC, a PCC/PCC composite would be appropriate because the user would see the same type of surface.
- *Continuity of adjacent lanes:* When widening is being designed, it is usually good design practice to continue the widening with similar materials. When the existing pavement is PCC that is in an acceptable condition, a PCC/PCC composite for the additional lanes has distinct advantages. The main advantage is ease to the driver in maintaining consistency across all lanes. There is also advantage in connecting the existing and new traffic lanes together so that they will not separate.
- *Availability of local materials and experience:* A PCC/PCC pavement can be built using recycled concrete aggregate (RCA) or recycled asphalt pavement (RAP) from the existing or nearby old highway, or from a local pit with some types of substandard aggregates (such as softer aggregates susceptible to polishing). The PCC surfacing will provide a smooth and durable surface for traffic.
- *Conservation of materials/energy:* PCC/PCC offers significant advantage in conservation of materials and energy. See *Sustainability* for PCC/PCC advantages.
- *Local preference:* There could be local preference for long-life PCC pavement but with low noise and low maintenance/rehabilitation needs. The PCC/PCC composite pavement can provide low noise with the EAC surface or with a special diamond-ground surface that will last a long time because of the high-quality aggregates in the surface layer.
- *Stimulation of competition:* A PCC/PCC composite pavement with low life-cycle costs stimulates increased competition.
- *Noise issues:* PCC/PCC can be designed with an EAC surface or with a diamond-ground surface that will provide low noise levels for many years. Either of these surfaces can be renewed easily and rapidly into the near future.
- *Safety considerations:* The EAC surface provides long-term high friction. The diamond-ground, high-quality aggregate surface provides high friction over a longer time period than is seen with conventional aggregates: It also provides lower probability for hydroplaning.

- *Experimental features:* Building a PCC/PCC composite pavement with distinct experimental features and sustainability benefits is a good way to get one built in a state or local highway agency.
- *Future needs:* A PCC/PCC can be designed to have a very long structural life with only the retexturing of the thin surface every 20 years (or longer).
- *Maintenance capability:* Little surface routine maintenance has been needed on the older EAC/PCC in Europe. The high quality concrete surface can be renewed rapidly through diamond grinding.
- *Sustainability:* This is where PCC/PCC has several distinct advantages over conventional pavements:
  - The lower PCC layer can be designed for a very long fatigue damage life, such as 40 to 100 years with minimal fatigue cracking repair. This results in a concrete slab that will remain structurally sound over decades while requiring only that the high-quality PCC surface be retextured every 20 to 40 years. Thus, there will be minimal, if any, full-depth slab replacements, which are expensive and require days to replace.
  - This renewal of the surface through some form of diamond grinding will provide excellent surface characteristics, including smoothness, low noise, and good friction. The hard aggregates will not polish as easily as often occurs with conventional softer aggregates.
  - This composite pavement design will thus reduce the amount of lane closures over the long design life of the pavement. This has a major sustainability impact because of the reduction in emissions caused by the extra congestion due to lane closures for maintenance and rehabilitation.
  - Reduction of the use of natural resources also contributes to improved sustainability. Recycled concrete was successfully used in the lower PCC slab at MnROAD. The existing concrete from I-94 was recycled as 50% of the coarse aggregate. There may be many projects for which such recycling of existing old PCC and old HMA/PCC pavements into new composite pavement would result in a major reduction in the haul distances involved, which would then result in lower energy use and costs. Use of recycled concrete results in a savings of natural aggregates.
  - Increased use of fly ash contributes to a substantial reduction in portland cement content in the lower PCC slab. The lower layer of the two MnROAD PCC/PCC composite sections contained 40% and 60% fly ash replacement, respectively. This reduces the carbon dioxide emissions and improves the sustainability of construction.
  - There exist highways in certain states where studded tire wear is the major cause of deterioration and needed rehabilitation. A highly durable wearing surface such as EAC could be used for a PCC/PCC composite pavement. If wear-down occurs over time, the surface can be renewed through diamond grinding.

Various methods are available for weighting economic and noneconomic factors. These include alternative-preference screening matrix, as described in NCHRP Report 703.

Example Cost Analysis of PCC/PCC Pavement

Initial Cost Analysis

An illustrative example of LCCA for a PCC/PCC composite pavement with EAC texture was prepared by the MnROAD paving contractor on behalf of this SHRP 2 R21 research. This also includes a direct comparison with a conventional JPCP at the same site.

The composite paving, or wet-on-wet paving, is a process that involves paving the roadway in two lifts. The first lift being one thick, lower-cost layer of concrete using recycled concrete as the main aggregate with lower percentages of higher-quality aggregates in the mix design. The second lift is a fairly thin (2 to 3 in.) high-quality layer that has high-quality aggregate (with none of the recycled material present) and higher cement content with less SCMs. The benefit of composite paving is expected to be in areas where high-quality aggregates are of a high cost or low supply (typically these are interrelated), and low-quality materials throughout one layer is not an option because of their polishing tendencies or durability as a surface material. Composite paving allows the lower layer to be produced using less expensive recycled material, allowing the higher-priced or scarce high-quality aggregates to be used in the upper layer. The recycled material in the lower layer is not expected to affect the structural quality of the slab as a whole, and this may require an increased thickness if strength is lower.

This project (used for the example LCCA) is based on an actual project located in Minnesota, in an area not readily accessible to high-quality aggregates. This situation commonly exists in many locations in the United States. The original conventional pavement bid is compared with the expected costs of paving had it been bid using composite paving techniques. The extra cost of two paving operations as well as two batch plants are compared with the expected costs for the aggregates using recycled material instead of Class A material for the lower layer. The objective is to find what the savings on the recycled material would have to be to break even in comparison with the conventional method.

General Information

The project was concrete paving along U.S. Highway 14 near Waseca, Minnesota. The project involved 90,000 cubic yards of concrete, 80,000 cubic yards of which was for paving 310,000 square yards of mainline pavement and 10,000 cubic yards of which was for crossroads and ramps. The project totaled 19.5 miles of paving. Twenty-two days of mainline

Table 4.9. Comparison of Conventional JPCP with Composite PCC/PCC with EAC Texture

Conventional	Composite
1 Boom truck	1 Boom truck
1 Paver	2 Pavers
1 Belt placer	2 Belt placers
1 Cure/texture	2 Cure/texture
1 Skid steer	1 Skid steer
1 Pickup truck	1 Pickup truck
1 Service truck	1 Service truck
1 Water truck	1 Water truck
	1 Steel bristle broom
13 Crew members	18 Crew members
Assumed mainline paving production of 0.90 mi/day	Assumed identical production of 0.90 mi/day
Unit cost to pave/tie/green saw: $2.98/square yard, or a total of $923,800	Unit cost to pave/tie/green saw: $3.70/square yard, or a total of $1,147,000
Mobilize and operate 1 plant	Mobilize and operate 2 plants. Marginal cost to mobilize second plant of $50,000 or a $.55/cubic yard premium
Plant operations cost $1.60/cubic yard to batch mix; cost includes plant operator, loader, and operator.	Operations cost of running 2 plants of $3.82/cubic yard to batch mix; cost includes 2 plant operators, loader, and operator.

paving were scheduled. The pavement was 27 ft wide, with a thickness of 9 in. For this project, the closest Class A aggregate source was New Ulm Quartzite. That source was a 2-hour round-trip haul from the project site.

Comparison

A comparison of the crew and equipment used for a conventional JPCP paving operation and the crew and equipment that would be necessary for composite PCC/PCC paving (with EAC texture) is shown in Table 4.9. Note the assumption of two paving operations and two PCC plants for the PCC/PCC paving operation.

Table 4.10 shows the expected differences in the extra cost to place the pavement compared with the savings in producing the structural concrete. Table 4.11 details the difference in the amount of aggregates used, as well as the cost differential between Class A aggregates and RCA.

The composite PCC/PCC showed comparable costs (difference of $44,800 or 0.7% of the total cost of approximately $6.7 million), while having substantial advantages. Also note that had a conventional texture been used (rather than the EAC texture) the costs would be even closer or actually lower for the PCC/PCC composite pavement. The price differential between the composite and the conventional pavement is mainly attributable to the increased costs of placing the two-layer concrete. Placement costs for the PCC/PCC composite paving increased $0.72 compared with conventional JPCP as a result of the increased costs to run an extra paver, belt placer, and larger crew size. However, the savings from the concrete aggregates was equal to $2.23 per cubic yard. This savings was due entirely to the use of recycled aggregate. These savings are achieved by crushing concrete on or near the site and using the recycled material as the main aggregate source in the thick lower layer. By substituting RCA instead of Class A as the course aggregate material, the amount of high-quality Class A aggregates needed for the job is reduced. The cost savings per ton of the RCA is between $5 and $6. In addition, the haul time for high-quality aggregates was a 2-hr round trip, but by crushing concrete on-site,

Table 4.10. Example of Conventional versus Composite Paving Costs for U.S. Highway 14 near Waseca, Minnesota

	Conventional Paving	Composite Paving
Pave, Tie, Green Saw		
Square yards	310,000	310,000
$ per Square yard	$2.98	$3.70
Total cost	$923,800	$1,147,000
Structural Concrete		
Cubic yards	80,000	80,000
$ per Cubic yard	$71.54	$69.31
Total cost	$5,723,200	$5,544,800
Total Conventional cost	$6,647,000	$6,691,800

Table 4.11. Example of Aggregate Comparisons for U.S. Highway 14 Near Waseca, Minnesota

Type	Tons
Conventional Aggregates	
¾ in. Class A	34,270
1½ in. Class A	37,213
Total tons	**71,483**
Class A	
Material $/ton	$12.78
Trucking (2 hour)	$7.46
Total $ per ton	**$20.24**
Composite Aggregates	
¾ in. Class A	11,310
1½ in. Class A	12,280
Recycled aggregate	47,893
Total tons	**71,483**
Recycled	
Material $/ton	$7.00
Trucking (2 hour)	$1.45
Total $ per ton	**$8.45**

the haul time could be reduced to a 20-min round trip. This results in a savings of just over $6 per ton in trucking costs.

Had this project been on the interstate with much heavier truck traffic, the lower layers would have been much thicker and the cost difference in favor of the PCC/PCC composite pavement. Even a difference of 1 to 3 in. (for a total PCC thickness of 10 to 12 in.) would tilt the cost advantage substantially in favor of the PCC/PCC composite pavement in this example.

MEPDG Performance Comparison

In addition to the initial construction cost, a performance analysis of the US-14 section using *MEPDG* (v. 1.3000:R21) was conducted. This analysis involved a comparison of the conventional JPCP with a PCC/JPC composite pavement. The designs are shown in Table 4.12. The inputs for this comparison used the same climate, traffic, and subgrade. The initial truck traffic was two-directional AADTT of 1,020. This traffic value was prescribed for heavy commercial traffic along US-14 in the vicinity of Waseca by the Minnesota DOT 2006 Trunk Highway Traffic Volume Map.

Performances for these sections with respect to faulting, cracking, and IRI are illustrated in Figure 4.1 through Figure 4.3, respectively. As evident in these figures, the performances of these structurally equivalent pavements are nearly identical. Considering the comparable initial costs (and potential initial cost savings if conventional texture was used), this suggests that the use of composite paving can provide cost benefits over the design life of a project with no sacrifice in performance.

Table 4.12. PCC Properties Used in Comparison of Conventional JPCP with Composite PCC/JPC Pavement for US-14 Section Using MEPDG

Design Feature	Conventional JPCP	Composite PCC/JPC
Slab thickness	9 in.	3-in. high-quality concrete (granite aggregate) 6-in. low-cost concrete (50% RCA, 60% fly ash substitution)
Joint spacing and load transfer	15 ft 1.25-in. dowels at 12-in. spacing	15 ft 1.25-in. dowels at 12-in. spacing
Base course	8-in. unbound crushed aggregate	8-in. unbound crushed aggregate
Shoulders	HMA	HMA

Note: *MEPDG* (v. 1.3000:R21).

Conclusions

These examples were prepared using a real-life project and the numbers it took to be a low bidder. The examples have shown that in the areas of the state where Class A aggregates are not readily available or are very expensive, PCC/PCC composite paving is a viable alternative to conventional paving. The heavier the truck traffic, the thicker the lower layer of lower cost concrete would become and the greater the difference in cost between the conventional and the composite PCC/PCC pavement. Although this example was a case of having no readily available Class A concrete aggregates, it has shown that it is possible for an alternative technique such as composite paving to compete essentially equally with the costs of a conventional paving process.

The *MEPDG* performance prediction of both sections illustrates the ability of PCC/PCC to equal its single-layer JPCP structural equivalent in performance and service life. Note that a life-cycle cost comparison of these two alternatives would have to assume that the future maintenance and rehabilitation were the same, at least over the first 20 years because of the predictions from the *MEPDG*. However, from 20 years on, the performance may be different, depending on how the surfaces of each pavement would perform in the harsh climate where this project would be constructed. It is expected that the PCC/PCC composite alternative would show better durability to harsh climate conditions because of the top high-quality PCC surface.

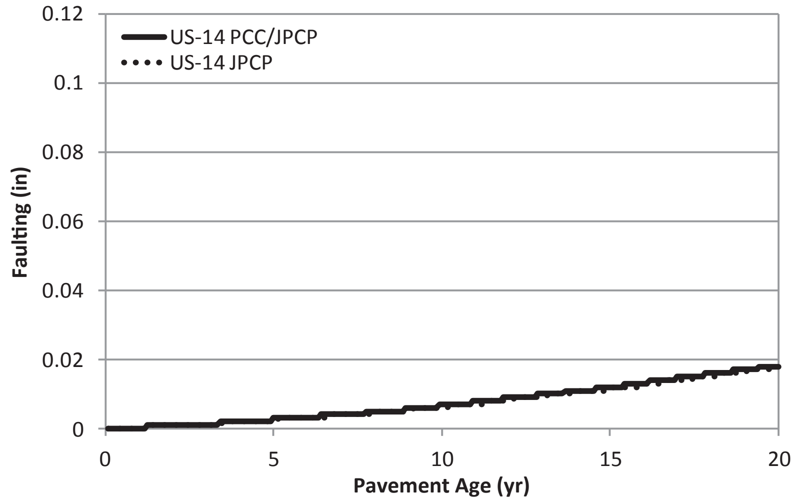


Figure 4.1. Joint faulting performance of the US-14 JPCP and its PCC/JPCP structural equivalent over a 20-year design life predicted using* MEPDG *(v. 1.3000:R21).

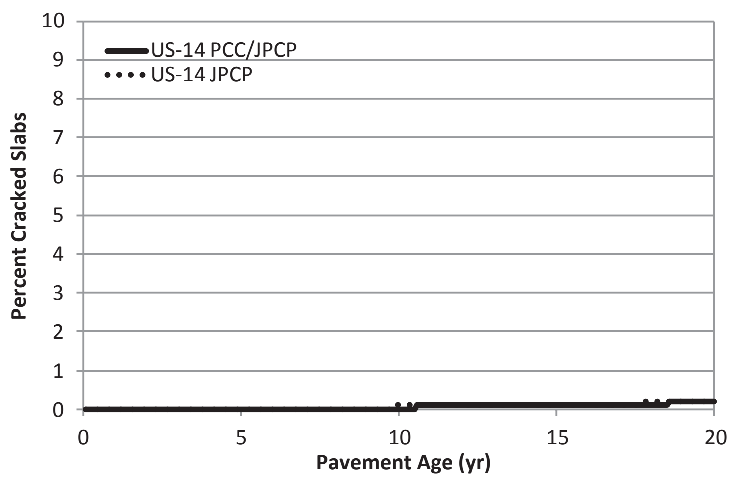


Figure 4.2. Percent cracked slabs for the US-14 JPCP and its PCC/JPCP structural equivalent over a 20-year design life predicted using* MEPDG *(v. 1.3000:R21).

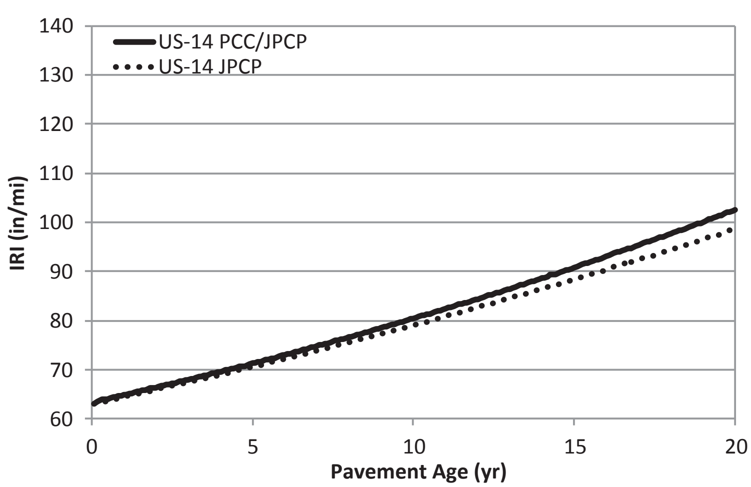


Figure 4.3. IRI predictions for the US-14 JPCP and its PCC/JPCP structural equivalent over a 20-year design life using* MEPDG *(v. 1.3000:R21).

CHAPTER 5

PCC/PCC Construction Guidelines

Introduction

Although the construction of the PCC/PCC composite pavement requires an additional paver and careful planning to ensure the production and placement of two PCC mixes, the process of construction itself does not use technologies and equipment that are not already available in the United States. The key steps in the construction include

- Prepare the sublayers (including subgrade, subbase, and base courses).
- Place the lower PCC layer and tied shoulders (if specified).
  - For PCC/CRC construction, the steel reinforcement needs to be securely placed on chairs on top of the base course before paving the lower PCC layer. The reinforcement depth for CRC should be at middepth or higher of the total PCC thickness.
  - For PCC/JPC, dowels may be placed in dowel baskets that are securely attached to the base course before paving the PCC layer. Alternately, dowel bar inserters (DBIs) may be used. The dowels should be located at middepth of the total PCC thickness.
- Place the upper PCC layer within 15 to 90 minutes (ideally within 60 minutes) of the lower PCC.
- For conventional textures (tining, burlap drag, turf drag, and so forth), texture the PCC surface as per specifications.
- Apply curing compound to the surface PCC layer. If EAC texture is specified, the curing compound should also include a surface retardant.
- For EAC texture, brush concrete surface with a rotating wire brush (within 5 to 24 hours of placement) to create EAC surface texture. Several brushings may be required.
- Saw cut and seal (if specified) joints in the PCC layer(s) (for PCC/JPC composite pavements).
- Place the shoulders (if not paved monolithically with the mainline PCC). PCC shoulder must be tied to the traffic lanes regardless of when they are placed.

Guidelines for each of the steps mentioned are included below. Sample specifications are included in Appendix W for PCC construction, texturing, curing, saw cutting, and sealing.

Construction Details

Prepare the Sublayers

The uniformity of the support conditions beneath a PCC/PCC pavement is critical for the long-term performance of the pavement, such as is in the case of conventional PCC pavements. For the purposes of preparing the sublayers, there is no difference between a PCC/PCC composite pavement and conventional PCC pavements such as JPCP or CRCP. Figure 5.1 shows an example of grading and compacting the subgrade and base in preparation for PCC placement. The same procedures and specifications that have been used by an agency to prepare the sublayers for PCC construction should be used and specified for the construction of PCC/PCC composite pavements. An agency may choose to incorporate one or more of the following to prepare the sublayers:

- Cement or lime treatment of the subgrade soils;
- Asphalt- or cement-treated base course;
- Permeable base courses with drainage features (such as edge drains); and
- Recycled pavement materials (such as RCA or RAP) that are not used in the PCC mix.

Reinforcing steel (for PCC/CRC composite pavements) and dowel baskets (if used for PCC/JPC composite pavements) can be placed directly on the base course following normal agency practices. Dowels should be placed middepth of the total PCC thickness (and not middepth of the lower PCC layer). They need to be securely fastened to the base course to ensure that they are not pushed by the paver. CRCP longitudinal steel must be placed on chairs and securely fastened to the base course.

Figure 5.1. Grading and compacting the subgrade and base in preparation for PCC placement.

Place the Lower PCC Layer

The lower PCC layer can be paved following the same procedures and guidelines for PCC/PCC composite pavements as for conventional JPCP or CRCP (Figure 5.2). Because the lower layer serves an entirely structural function in the pavement system, concerns about ride quality or surface texture durability are not applicable. The only concerns in the construction of the PCC/PCC lower layer are meeting structural and durability criteria (such as compressive strength, flexural strength, air content, consolidation around dowel bars), as is the case for conventional JPCP and CRCP. It is recommended that the locations of random dowels across transverse joints be measured using a probe just after completion of paving operations to ensure they are being placed in the proper location. The same goes for depth of CRC reinforcement, for which having the proper depth is even more critical. Agency QA/QC practices for testing materials and monitoring construction activities for JPCP or CRCP should be followed for the lower PCC placement of PCC/PCC composite pavements. This includes testing for slump, mix temperature, entrained air, and so forth (Figure 5.3).

The lower PCC layer is placed such that the paving width is 1.5 to 2 in. less than the final desired paving width. This allows for a clearance of 0.75 to 1 in. on both sides for paving the top lift PCC. The lower PCC is a stiffer "dryer" mix with low slump (~1 in.) to support the weight of the upper PCC (Figures 5.2 and 5.4). The mix typically is stiff enough to support the weight of an average person without the person sinking (see footprints and member of paving crew walking on the lower PCC in Figure 5.4) but still has enough moisture to form a good wet-on-wet bond with the upper PCC layer.

If DBI is used to place dowel bars in JPC, vibration of the lower PCC is not necessary (as is the practice in Europe). However, if dowel baskets are used for the JPC or CRC is used for the lower PCC, vibration of lower PCC may be necessary for adequate consolidation of the stiff mix around the dowel

Figure 5.2. Paving the lower PCC layer.

Figure 5.3. QA testing of the PCC layer for entrained air and slump.

baskets or reinforcing steel. The vibrators should be higher than the top of the dowel basket assembly (or reinforcing steel) to prevent dragging or moving the dowel baskets (or steel rebars).

Delivering the appropriate concrete mix to the appropriate paver is a detail that needs to be addressed before construction. At the I-70 construction in Kansas, both concrete mixes were batched and mixed at the same central mix batch plant. Batch plant personnel would identify to the driver which mixture they were transporting through the use of a green card for the lower lift or a red card for the top lift. Truck drivers then displayed a card of the appropriate color in the windshield of their truck. The belt placers used for supplying concrete to the pavers were also identified with green and red paint. During the PCC/PCC construction at MnROAD, the contractor had personnel specifically responsible for this task. Because both mixes were produced at the same ready-mix plant, the contractor requested, via radio communication with the plant, the desired mix, and a paving foreman directed the trucks to the appropriate pavers once the truck arrived at the job site based on the mix ticket.

Although important for all slipform paving operations, consistent delivery of uniform concrete is even more crucial for PCC/PCC construction, as was detailed in Chapter 2. In the placement of both layers, it is important that construction logistics in the delivery of PCC mixes for the layers be understood by all parties involved. This allows the paving train to move at a regular pace and place both layers within the required time frame. Location of the dowels and tie bar steel in the lower lift must be checked constantly.

Place the Upper PCC Layer

The upper PCC layer must be placed between 15 and 90 minutes (ideally less than 60 minutes) after the placement of the lower PCC layer. Placing the second lift soon after the placement of the first is important to ensure that the two PCCs in the layers bond well at their interface. Typically, because of

Figure 5.4. Paving the upper PCC layer.

Figure 5.5. Vibrating the upper lift PCC and edge of upper lift paver relative to the lower PCC layer.

logistical constraints, the upper PCC is placed over the lower PCC using a belt placer (Figure 5.4).

During the Kansas construction, the belt placer for the top lift was modified slightly with the addition of a grid fabricated from square steel tubing that was placed under the discharge end of the belt to reduce the potential for deformation of the bottom PCC while concrete was being unloaded from the belt onto the bottom PCC.

During both the Kansas construction and the construction at MnROAD, the vibrators were raised above the elevation of the extrusion pan to prevent comingling of mixes in the upper and lower PCC (Figure 5.5). As an added precaution, at MnROAD, the vibrations were reduced to 4,000 vpm.

Width and alignment of the two pavers also require special attention. Even a slight drift to one side or another on the part of either paver can result in edge problems because the upper lift PCC edge is less than 1 in. from the edge of the lower lift PCC (Figure 5.5).

Texture the Upper PCC Layer

Conventional Texture

Typical agency guidelines for texturing the PCC surface (for conventional JPCP or CRCP) through tining, turf drag, burlap drag, and so forth (Figure 5.6) can be followed, and no modifications specific to PCC/PCC composite pavements are necessary.

To control rapid moisture loss from the surface of the wet PCC to prevent rapid surface drying and early-age cracking, the upper layer PCC should be cured adequately. Agency guidelines and specifications to control moisture loss, such as application of curing compound (or other practices, such as wet burlap) should be specified (see Figure 5.6).

EAC Texture

The upper PCC should be finished smooth and then sprayed with an adequate retarding/curing compound. The PCC/PCC

Figure 5.6. Longitudinal tining texture (*left*) to create adequate surface texture on top of the upper lift PCC and application of curing compound to control surface moisture loss (*right*).

Figure 5.7. Spraying of the curing/retarding compound over the upper PCC at MnROAD.

should be sprayed immediately after finishing, the compound application should be uniform, and the compound application should be sufficiently thick. If the finishing is uneven, the later texturing efforts can be compromised: areas of lighter compound application will result in less surface paste removed than will areas with the heavier application. Once the compound is sufficiently thick, a slight increase in its thickness does not affect the depth of etch.

The compound can be applied using an automated spray bar, with spray nozzles tuned to apply the compound at a regular rate (Figure 5.7). Spray nozzle overlap should be accounted for, and dripping nozzles should be repaired quickly. The most critical aspect of the compound application is that enough of the compound is applied. Only in the event that not enough of the compound is used does nonuniformity in application become a serious problem. The spray nozzles may need to be protected from wind gusts by installing protective shields (Figure 5.7).

In Kansas, a combined curing/retarding compound was not used, as was done at MnROAD. During the trial construction, curing compound applied over the surface retardant appeared to interfere with the effectiveness of the brushing operations. As a result, during final paving, the initial curing was accomplished using polyethylene sheeting placed after the surface retardant was sprayed, instead of a liquid membrane curing compound.

The timing of the EAC brushing requires a great deal of "hands on" experience that only comes with constructing the texture in the field. The timing is determined by the ease of dislodging an aggregate embedded in the concrete. If the aggregate moves easily, the concrete should continue to cure. Once aggregates resist dislodging somewhat, a push broom can be used to gauge the readiness of the surface paste for brushing at approximately 20-ft intervals (Figure 5.8). The surface requires more curing time before brushing if (1) the surface paste and curing/retarder compound clumps together when

Figure 5.8. Gauging the brushing readiness of the EAC surface with a push broom.

Figure 5.9. Rotating sweeper brush with mechanical arm extended to relieve pressure on the brush (left) and close-up of the brush bristles (right).

brushed with the hand broom instead of coming off as individual particles or (2) the shearing of the sweeping action of the broom dislodges aggregates.

If the brush tests adequately remove paste from the surface without dislodging aggregates, the next step is to create the EAC texture. This is done with a wire brush mounted to a grader, as shown in Figure 5.9. The brush bristles should be stiff and sufficiently durable to remove the mortar but not overly stiff such that they damage the PCC surface or dislodge the aggregates. The stiffness of the brush is one of the factors that affect the number of passes needed (typically two to five) to obtain the desired EAC surface texture. A properly brushed EAC surface is shown in Figure 5.10.

Much of the brushing will be a trial-and-error process involving passes with the wire brush and quick assessments using the sand patch test. The brush should be raised or lowered to provide the pressure necessary to etch the desired texture depth (at MnROAD this was 0.8 to 1.2 mm). The sand patch test should follow ASTM E965 (Standard Test Method for Measuring Pavement Macrotexture Depth Using a Volumetric Technique) and be conducted on interspersed locations across the PCC/PCC pavement.

It is important to note that attempting an EAC finish is not advisable if there is any threat of precipitation. Approximately 250 ft of MnROAD R21 EAC finish appeared marbled and lacked a uniform etch depth because the surface, covered with curing/retarder compound, was saturated with rain. The contractor attempted to cover the 250 ft length of pavement with plastic sheeting, but the rain came too fast and was too heavy to avoid damage. The water mixed with the curing/retarder compound and cement paste at the surface. As a result, the slabs had to be saw cut before the surface was ready to be brushed—approximately 15 hours after the curing/retarder compound had been applied. Furthermore, the water from the saw cutting removed some of the curing/retarder compound at the joints.

Figure 5.10. Properly brushed EAC surface.

Figure 5.11. Next-generation diamond grinding texture (**left lane*****) and conventional diamond grinding texture (*****right lane*****) at MnROAD.***

Diamond Grinding Texture

If specified, the upper lift PCC can be diamond ground after it has cured sufficiently. Agency specifications for diamond grinding of conventional JPCP or CRCP can be used, and no changes are necessary. Diamond grinding options include conventional diamond grinding (with closely spaced diamond blades and a deeper texture) or next-generation diamond grinding (a combination of shallow surficial grind and deep grooves), as shown in Figure 5.11.

Saw Cut Joints in the PCC/JPC

Joints in PCC/JPC should be saw cut and sealed (if specified) following conventional agency practices. Typical specifications for JPCP require a minimum saw cut depth of ⅓ the PCC thickness. For PCC/PCC composite pavements, a minimum saw cut depth of the greater of ⅓ the total PCC thickness (of both layers) or thickness of the upper PCC layer plus 0.5 in. should be specified to ensure proper opening of the joints. Table 5.1 shows the minimum recommended saw cut depths for PCC/PCC composite pavements.

Table 5.1. Minimum Recommended Saw Cut Depths for PCC/PCC Pavements

		Upper PCC Thickness (in.)				
		1.5	**2.0**	**2.5**	**3.0**	**3.5**
Lower PCC Thickness (in.)	**6.0**	2.50	2.67	3.00	3.50	4.00
	6.5	2.67	2.83	3.00	3.50	4.00
	7.0	2.83	3.00	3.17	3.50	4.00
	7.5	3.00	3.17	3.33	3.50	4.00
	8.0	3.17	3.33	3.50	3.67	4.00
	8.5	3.33	3.50	3.67	3.83	4.00
	9.0	3.50	3.67	3.83	4.00	4.17
	9.5	3.67	3.83	4.00	4.17	4.33
	10.0	3.83	4.00	4.17	4.33	4.50
	10.5	4.00	4.17	4.33	4.50	4.67
	11.0	4.17	4.33	4.50	4.67	4.83
	11.5	4.33	4.50	4.67	4.83	5.00

Note: Cut depths are measured in inches.

CHAPTER 6

PCC/PCC Conclusions and Recommendations for Future Research

Conclusions

The SHRP 2 R21 project on composite pavement systems conducted extensive in-depth research on two types of composite pavements:

1. High-quality, relatively thin, hot-mix asphalt (HMA) surfacing over a new portland cement concrete (PCC) structural layer (e.g., JPC, CRC, jointed RCC, or LCB/CTB).
2. High-quality, relatively thin PCC surfacing atop a thicker structural PCC layer (e.g., JPC, CRC).

Composite pavements have been proven in Europe and in the United States to provide long lives with excellent surface characteristics (low noise, smoothness, and high friction), structural capacity, and rapid renewal when needed, and they use recycled and lower-cost materials in the lower PCC layer. Composite pavements also reflect the current direction of many highway agencies to build economical sustainable pavement structures that use recycled materials and make use of locally available materials.

Both types of composite pavements have strong technical, economical, and sustainability merits in fulfilling the key goals of the SHRP 2 program, including long-lived pavements, rapid renewal, and sustainable pavements. The objectives of this research were to investigate the design and construction of new composite pavement systems for all levels of highways and streets. This chapter provides a summary of research conclusions for PCC surfaces over a PCC lower layer. PCC for the thin surface course is used generically to cover surfaces with various surface textures, including:

1. High-quality concrete with exposed aggregate concrete (EAC) texture.
2. High-quality concrete with very durable nonpolishing aggregate that can be diamond ground.
3. High-quality concrete that is conventionally textured.
4. Normal quality concrete that is diamond ground or conventionally textured.

The lower PCC layer includes JPC or CRC but may include recycled or alternative materials (RCA, RAP), increased use of more local and less expensive aggregates (e.g., aggregates susceptible to polishing), and higher substitution rates for cementitious materials (higher fly ash or other SCM contents). As part of this research, the R21 research team

1. Determined the behavior, material properties, and performance for PCC/PCC composite pavement under many climate and traffic conditions. Experimental composite pavements were constructed at the MnROAD research site and were instrumented and monitored under climate and heavy traffic loadings. Extensive field surveys were performed in the United States, Canada, and Europe of 15 sections of PCC/PCC composite pavements. The bottom line on these performance studies is that PCC/PCC composite pavements have clear advantages over conventional PCC pavements in terms of cost, sustainability, and long structural lives coupled with rapid renewal of the surface.
2. Evaluated, improved, and further validated the various structural, climatic, material, performance prediction models, and design algorithms that are included in the AASHTO *MEPDG*, and the Lattice 3D PCC/PCC bonding model. A special version of R21 *MEPDG* Overlay design procedure for bonded PCC over JPCP or CRCP (*MEPDG* version 1.3000:R21) can be used for "New" PCC/PCC composite pavements. A summary of the key analysis and performance findings were included in Chapter 3.
3. Detailed recommendations provided for inputs and modifications to the DARWin-ME software for composite pavements. Recommended revisions for the AASHTO *MOP* are also provided.
4. Developed practical recommendations for construction specifications and techniques, life-cycle costing, and training materials for adoption by the transportation community.

The products developed as part of SHRP 2 R21 will result in improved design and life-cycle cost procedures for composite pavements. The guidelines, techniques, and specifications developed in SHRP 2 R21 will greatly advance the state-of-the-practice of constructing composite pavements. Composite pavements are congruent with SHRP 2 Renewal philosophy because they are designed to be long-lasting pavements that can be rapidly renewed. For highway engineers, designers, and decision makers at agencies, composite pavements provide another tool in the designers' arsenal and can be a cost-effective alternative to conventional concrete and asphalt pavements over the life cycle of the pavement. Together these reports, software, and guidelines provide information for adoption by the transportation community and for these technologies to become widely adopted.

Based on the comprehensive results achieved from this SHRP 2 R21 study, the key characteristics of PCC/PCC composite pavement was determined as follows:

- Excellent surface characteristics from the thin, high-quality concrete surface layer. These include low noise, high friction, very good initial smoothness, minimal wear over time, and high surface durability over a long time period beyond 20 years even under harsh weather conditions.
- Ability for rapid renewal of a thin surface course as it wears under traffic and weather (e.g., diamond grinding in various forms or some other type of retexturing).
- Long life structural design of the lower PCC layer (e.g., designed for minimal fatigue damage over a 40- to 100-year period).
- Avoidance of certain distress types that occur regularly in conventional pavements but are rare or nonexistent in PCC/PCC composite pavements. Table 6.1 shows the direct comparison for conventional PCC and composite PCC/PCC pavement.
- Improved life-cycle costs over a long life span because of overall lower construction costs (e.g., increased recycling, local aggregates, cement substitution amounts) and low future maintenance (e.g., high-quality PCC surface will be more durable) and rehabilitation costs over time (e.g., no full-depth repairs of PCC slab because of reduction in fatigue damage).
- Improved sustainability practices through structural and materials design of the lower PCC layer: increased use of recycled materials (RCA, RAP), increased use of more local and less expensive aggregates, and higher substitution rates for cementitious materials (higher fly ash contents and other SCMs).

Intended Audience, Usage, Value Added to State of the Practice and State of the Art, Potential Benefits of Acceptance and Implementation

The key products of the SHRP 2 R21 project on composite pavements include

- Examples of the performance of composite pavements;
- Design procedures;
- Construction guidelines;
- Pavement selection type guidelines; and
- Training materials.

The key intended audiences for these products are as follows:

- State highway agency managers, engineers, and consultants. New composite pavements need to be added to the

Table 6.1. Comparison of Conventional PCC Pavement with Composite PCC/PCC Pavement for Several Key Distress Types

Distress Type	Conventional JPCP and CRCP	Composite PCC/JPC and PCC/CRC Pavement
Bottom-up slab fatigue cracking	Yes, major design concern.	Yes, major design concern.
Top-down slab fatigue cracking and top-down fatigue cracking for punchouts	Yes, major design concern.	Yes, major design concern. If higher strength PCC is used in the top surface, this is less of a concern and this damage mechanism may be reduced.
Top-down longitudinal fatigue cracking and corner breaks	Yes, this has occurred on some projects.	No, this was not observed on any PCC/PCC composite project. Higher strength PCC surface layer may be beneficial here.
Surface wear-down (rutting)	Yes, this has occurred on projects in studded tire states/areas.	Minor on most composite pavements in nearly 20 years in Austria because of high quality aggregates and high cement content.
Polishing of surface (poor friction)	Yes, this has occurred on many projects resulting from polishing aggregates. Higher quality aggregates are costly if used throughout the entire PCC slab thickness.	EAC surfacing in Austria and Germany, where there is extensive snow and ice, has shown only minor polishing in the wheelpaths over 20 years. Use of the highest quality aggregates in the thin top layer would minimize polishing from occurring and lower construction costs.
Joint faulting	Yes, major design concern.	Yes, major design concern.

routine pavement type selection procedures of state and local highway agencies. The performance examples, design tools, pavement selection type guidelines, construction guidelines and specifications, and training materials will all provide significant value-added technology to these engineers and managers regarding composite pavements. If DARWin-ME is upgraded to include composite pavements as a new pavement type along with the conventional types, this will be a major advancement in the consideration of composite pavements in the industry.
- Federal Highway Administration management and engineers. FHWA managers and engineers need to be made more aware of the past performance and benefits of composite pavements so they can discuss the possibility of adding them to their regular pavement selection process. The SHRP 2 R21 products provide the needed information.
- Researchers from academia, federal and state agencies, and industry. There are lots of additional opportunities to improve on the design and construction of composite pavements that open up the future of additional research. Making faculty more aware of the advantages of composite pavements and getting this instruction into the classroom is key to educating students about the benefits of composite pavements. The training materials and various documents from the R21 project will be valuable for universities for future course development.

The key benefits of state and local highway agencies incorporating PCC/PCC composite pavements into their routine pavement type selection process include the following:

- PCC/PCC composite pavements provide more flexibility in the highway agency pavement management strategy for new and especially for future rehabilitation and are another tool in the designers' toolbox. The design of composite pavement to achieve lower life-cycle costs, increased sustainability, and longer life will require additional efforts in design, materials, and construction specifications by the highway agency to achieve these goals because new technology is involved.
- A PCC/PCC composite pavement can be designed cost-effectively to have a life span similar to that of a conventional PCC pavement. It also can be designed to be a much longer term (long-life) pavement with minimal structural fatigue damage over many years, and the surface will provide long life, but it can be rapidly renewed as needed by retexturing (grinding) with no deep structural problems that require additional lane closures to repair and cure. The long-term LCCA and sustainability should show favorable results for this type of design strategy.
- Excellent surface characteristics can be provided. This includes the following:
  - Very smooth surfaces (e.g., initial IRI of 50 in./mi has been achieved for PCC/PCC composite pavements).
  - Low noise surfaces (e.g., EAC, conventional and ultimate diamond grinding).
  - High friction over the long term with high-quality nonpolishing aggregates in the top layer.
- Rapid retexturing of the PCC surface through diamond grinding or other technology that may evolve over time. This would reduce traffic congestion over many years. The diamond grinding or other technology would last longer because it has a very hard nonpolishing aggregate in the surface.
- Improved sustainability can be provided for composite pavements in several ways:
  - Increased composite pavement longevity is a key to improved sustainability.
    - The PCC/PCC type of composite pavement can be designed for a very long fatigue damage life, such as 40 to 100 years using the *MEPDG* (version 1.3000:R21) at a high level of reliability. Slab thicknesses required are comparable to those for single-material JPCP or CRCP. The design can include minimal slab fatigue cracking over the design period. Longevity is a key component of sustainability.
    - The thin, high-quality PCC surface should also last a long time (e.g., >20 years in harsh climates) based on field projects surveyed in Europe.
    - The rapid renewal of the surface through some type of diamond grinding will provide excellent surface characteristics, including smoothness, low noise, and good friction over the life of the pavement.
    - This PCC/PCC composite pavement design will thus reduce the amount of lane closures over the long design life of the pavement. This has a major sustainability impact because of the reduction in emissions caused by the extra congestion due to lane closures for maintenance and rehabilitation.
  - Reduction of the use of natural resources is another key for improved sustainability. RCA was used successfully in the lower PCC slab in the MnROAD R21 section. The existing concrete from Minnesota I-94 was recycled as 50% of the coarse aggregate. There may be many projects in which such recycling of existing old PCC and old HMA/PCC pavements into the new composite pavement would result in a major reduction of the haul distances involved, which would result in lower energy use and costs. Of course, the use of recycled concrete results in a savings of natural aggregates.
  - Increased use of fly ash in achieving a substantial reduction in portland cement content in the lower

PCC slab. The lower layer of the two I-94 composite sections contained 40% and 60% fly ash replacement, respectively. The use of RCA, RAP, and fly ash offers environmental advantages by diverting the material from the waste stream, reducing the energy investment in processing virgin materials, conserving virgin materials, reducing carbon dioxide emissions, and minimizing pollution.

- There exist highways in certain states of the United States where studded tire wear is the major cause of deterioration and needed rehabilitation. A high-quality PCC surface, whether it is EAC, ultimate diamond ground, or conventional texturing, would provide a more durable surface that would resist wear of studded tires caused by the high-quality aggregates used in the surface layer.
- Increased use of lower-cost local aggregates in the lower PCC layer because the lower PCC is no longer the wearing course that will polish. The higher quality PCC surface provides some protection from freeze–thaw damage and wet–dry cycling. The use of local aggregates improves sustainability by reducing resources spent in hauling aggregates over long distances.

Recommendations for Additional Development or Refinement of the Products

Each of the R21 products is listed in Table 6.2 along with recommendations on required future development and refinement that are needed for full implementation.

Table 6.2. SHRP 2 R21 Project Recommendations for Additional Development of PCC/PCC Products

SHRP 2 R21 Product	Implementation Status	Additional Development Required	Comment
MEPDG R21 version software	R21 improvements to “Bonded PCC/PCC” to simulate new PCC/PCC and address limitations of existing structural and environmental models for PCC/PCC. Can be used for design of PCC/JPC, PCC/CRC, HMA/JPC, and HMA/CRC.	None.	*MEPDG* (version 1.3000 R21) is available from SHRP 2 and AASHTO. Use “Overlay” design procedures for “New” PCC/PCC composite pavements with appropriate inputs.
AASHTO DARWin-ME software	“Overlay” design cannot be used to design “New” PCC/PCC composite pavement. Modifications are needed as described.	Significant modifications for PCC/PCC that were made in the R21 version need to be made to DARWin-ME software. User interface requires revision to show PCC/PCC and HMA/PCC composite pavements as “New” pavement alternatives.	Improvements should be made as soon as possible for highway agencies to design “New” composite pavement.
AASHTO *MOP*	Detailed recommendations were prepared to include “New” composite pavements.	None.	Revision can be made in tandem with modifications to the DARWin-ME software.
MnROAD PCC/JPC test section	Two PCC/PCC composite sections constructed, instrumented, and being monitored under heavy interstate traffic for one full year. Construction and first year’s performance measured.	Monitoring of the section over time would produce valuable longer-term information to convince highway agencies to build composite pavements.	These sections should be monitored at least twice a year. Full performance will not be known for more than 10 years. Many major findings will be discovered over time for structural, texture, and sustainability. These data can be used to update calibration coefficients and further verify long-term *MEPDG* structural responses to traffic and thermal loading and refine models (e.g., IRI and reflection cracking as appropriate).
2008 Survey of European Composite Pavements	Report completed and available online: www.trb.org/Main/Blurbs/163693.aspx.	None.	Already available to the public. There has been a great deal of interest.

(continued on next page)

Table 6.2. SHRP 2 R21 Project Recommendations for Additional Development of PCC/PCC Products (continued)

SHRP 2 R21 Product	Implementation Status	Additional Development Required	Comment
Database of PCC/PCC Composite Pavements	Data collected for 15 PCC/PCC composite sections are available in Excel spreadsheet and *MEPDG* input files.	None.	These data may be of interest to agencies wishing to develop designs for new composite pavements.
Lattice model for PCC/PCC bond	The model was further developed and connected to an FEM to analyze PCC/PCC bond conditions.	Additional work remains to be done to extend this valuable research software into a practical tool.	This program showed that new PCC/PCC composite has sufficient bond if constructed properly.
JPCP fatigue cracking models in *MEPDG*	The JPCP model was validated for both HMA/JPC and PCC/JPC composite pavement data.	None. The global coefficients are sufficient.	JPC fatigue damage in composite pavements is critical to their structural design.
Life-cycle cost analysis (LCCA) guidelines	Recommendations using the FHWA RealCost spreadsheet for composite pavements were developed.	None.	The *MEPDG* predictions can provide pavement life estimation for use in LCCA.
Instrument data	Extensive instrumentation data exist for MnROAD and included as SHRP 2 R21 database.	The full analysis of these data was not possible under R21, and much additional analyses can be accomplished.	Some valuable data on temperature, moisture, strains from climatic change, and dynamic strains from loadings.
Examples of PCC/PCC composite designs	A range of examples of composite pavement design and performance were used in R21.	Additional research into additional aspects of composite pavements can be accomplished with these data.	The performance of most composite pavements was very good. These sections can be used to demonstrate this to highway agencies.
Construction specifications for PCC/JPC	The MnROAD specifications are available and cover a variety of aspects of PCC/PCC construction.	None.	Key aspects are PCC bonding, PCC lower layer mixture RCA characteristics, SCM replacement, upper layer mixture characteristics, brushing of EAC, curing/retarding of PCC, texturing, and wet-on-wet paving.
RILEM CIF concrete freeze–thaw standard	Equipment was checked out and many PCC samples tested. Very useful results were obtained.	Additional testing on all quality levels of aggregate is recommended. This equipment should be more fully evaluated for U.S. applications.	An excellent field simulation for freeze–thaw damage of a given PCC.
Training products	Presentations on design, construction, materials, performance, and examples of both types of composite pavements.	None.	A variety of presentations are available for use in promoting composite pavements, as is technical training of engineers and contractors.
Advantages of composite pavements	R21 has brought to light the many advantages of PCC/PCC pavements.	Development of design and cost comparisons for conventional design versus composite designs at specific sites.	Direct comparison of designs and costs makes a strong convincing case for PCC/PCC composite pavements.

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Appendices A–V

Appendices A through V are available online: www.trb.org/Main/Blurbs/168145.aspx. The appendices are as follows:

Appendix A: History and Background of HMA/PCC Composite Pavements
Appendix B: History and Background of PCC/PCC Composite Pavements
Appendix C: Highway Agency Survey
Appendix D: Distress Mechanisms of HMA/PCC Composite Pavements
Appendix E: Distress Mechanisms of PCC/PCC Composite Pavements
Appendix F: Construction of Test Sections at MnROAD
Appendix G: Construction of Test Sections at UCPRC
Appendix H: Instrumentation and Analysis of Instrumented Data at MnROAD
Appendix I: HMA/PCC Rutting Model
Appendix J: Measurement and Analysis of PCC Slab Temperature Profiles at UCPRC
Appendix K: HVS Cracking Tests at UCPRC
Appendix L: HVS Rutting Tests at UCPRC
Appendix M: Joint Movement Monitoring at UCPRC
Appendix N: Laboratory Testing of HMA Mixes at UCPRC
Appendix O: HMA/PCC Bonding and Friction Literature
Appendix P: CalME Model
Appendix Q: Laboratory Testing of PCC Mixes
Appendix R: *MEPDG* Modifications for PCC/PCC Pavements
Appendix S: Lattice 3D Model Background
Appendix T: Recycled Concrete Aggregates in PCC
Appendix U: Freeze-Thaw Durability Testing of PCC Mixes
Appendix V: Brushing and Exposed Aggregate Concrete

Related SHRP 2 Research

Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform (R02)

Modular Pavement Technology (R05)

Real-Time Smoothness Measurements on Portland Cement Concrete Pavements During Construction (R06E)

Using Existing Pavement In Place and Achieving Long Life (R23)