

# National Conference on Preservation, Repair, and Rehabilitation of Concrete Pavements

St. Louis, Missouri  
April 22–24, 2009

**Sponsored by:**

Federal Highway Administration

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American Association of State Highway  
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**Concrete Pavements—Safer, Smoother, Longer Lasting**



U.S. Department of Transportation  
**Federal Highway Administration**



*Proceedings*

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**Disclaimer**

The information presented at the St. Louis conference and contained in these proceedings does not represent any formal endorsement of techniques, materials, or processes by the sponsoring organizations. The information presented in these proceedings should be used judiciously by experienced concrete pavement technologists.

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## Foreword

Well-designed and well-constructed concrete pavements can be expected to provide low-maintenance service life, well exceeding the as-designed service life. The majority of pavements in the U.S. Interstate and primary systems were designed on the basis of a 20- to 25-year initial service life, and many miles of these pavements are in service after more than 30 to 35 years. However, concrete pavements do deteriorate with time and traffic loadings and because of concrete material failures. But, sound corrective measures performed in a timely manner can greatly extend the service life of existing concrete pavements. These corrective measures include preservation treatments, repair/restoration activities, and rehabilitation. The goal of the corrective measures is to extend the useful life of concrete pavements (structural capacity and functional characteristics) with the least life cycle costs. Timely preservation activities can delay the need for repairs, and timely repairs can delay the need for rehabilitation. Delays in timely preservation, repair, and rehabilitation (PRR) or improper PRR activities can lead to pavements that are in such poor condition that the only option remaining is reconstruction, which is more costly.

Over the last two decades, there has been much progress in developing effective PRR techniques. However, many gaps remain, and many practices are not implemented consistently from one region to another. An important technical limitation is associated with our ability to rationally determine what treatments need to be performed at what stage in the pavement's life and what are the consequences of delaying needed treatments. In today's environment, where the highway agency budgets cannot fully meet the needs for managing pavement assets yet there is no lessening in traffic growth and public expectations, it is important that the limited funds available to maintain our highway systems are expended in an optimal manner.

This 2 1/2-day National Conference on Preservation, Repair, and Rehabilitation of Concrete Pavements was organized as a part of technology transfer activities conducted under the U.S. Concrete Pavement Technology Program, which operates within the Federal Highway Administration. The conference objective was to provide a national forum to address the technology needs related to the PRR of concrete pavements.

The editor would like to thank the authors for supporting the objective of this conference by developing comprehensive papers related to the conference themes. The papers included in the proceedings were peer-reviewed for technical content, and the editor would also like to thank the conference steering committee members and the many reviewers who participated in the paper review process.

*Editor:*

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## **Part 1**

# **Pavement Condition Evaluation, Impact, and Durability**



# Minimizing Reflective Cracking With Applications of the Rolling Dynamic Deflectometer and Overlay Tester

Dar-Hao Chen,<sup>1</sup> Moon Won,<sup>2</sup> Tom Scullion,<sup>3</sup> and John Bilyeu<sup>4</sup>

## ABSTRACT

Since reflective cracking is related to both the existing pavement condition and the properties of the overlay material, quantitative methods are required to assess both the vertical movements of the cracks (or joints) for the entire project and the reflection cracking resistance of the overlay material. Since 2000, the rolling dynamic deflectometer (RDD) has been used in Texas to provide 100 percent coverage of existing joint conditions of concrete pavements being considered for asphalt overlays. The RDD assesses the vertical movements of each joint and identifies the weak support areas and locations where the slabs are rocking. The continuous deflection profiles produced are used to locate areas with high potential for reflective cracking due to poor load transfer and high slab movements. The overlay tester (OT) has been developed as a mix design tool to characterize the ability of an asphalt mix to resist reflective cracking. OT results have not yet been integrated with RDD results to predict the exact extent of reflective cracking that will occur. However, OT results are still good for ranking various mixtures in terms of crack performance, and some guidelines based on OT and RDD results have been developed.

This paper presents a series of case studies illustrating the relationship between the RDD deflection profiles, the OT results of the asphalt mixes, and the resulting field performance. On IH-20 experimental sections in northeast Texas, the RDD identified many locations that have high potential for reflective cracking. The mix used on this project was found to have poor crack resistance and failed the overlay test quickly (2 cycles). Major reflection cracking problems were encountered on this project. At another project, SH-12 in the Beaumont District, no visible cracks have been observed after 2 years of service, despite significant movement detected by RDD. The main reason for the good performance on SH-12 is believed to be due to the thick and flexible overlay mix (the mix lasted more than 900 cycles in the OT). On a section of US-96 in the Beaumont District, the RDD determined that the pavement had good load transfer efficiency across the cracks, and consequently was at low risk of reflection cracking, even though the surface condition was poor, with severe transverse cracks and spalling. A stone matrix asphalt mix with OT life exceeding 700 cycles was placed 5 years ago, and performance to date has been excellent. Based on these case studies, TxDOT has developed criteria for interpreting the RDD deflection data and for defining the required properties of asphalt overlays to provide good performance.

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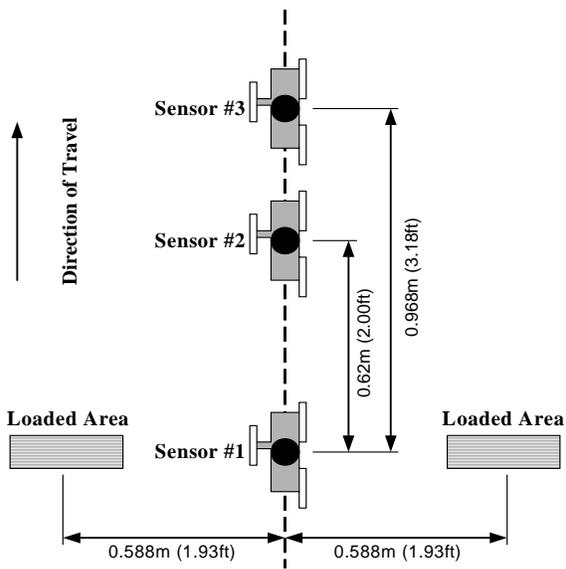
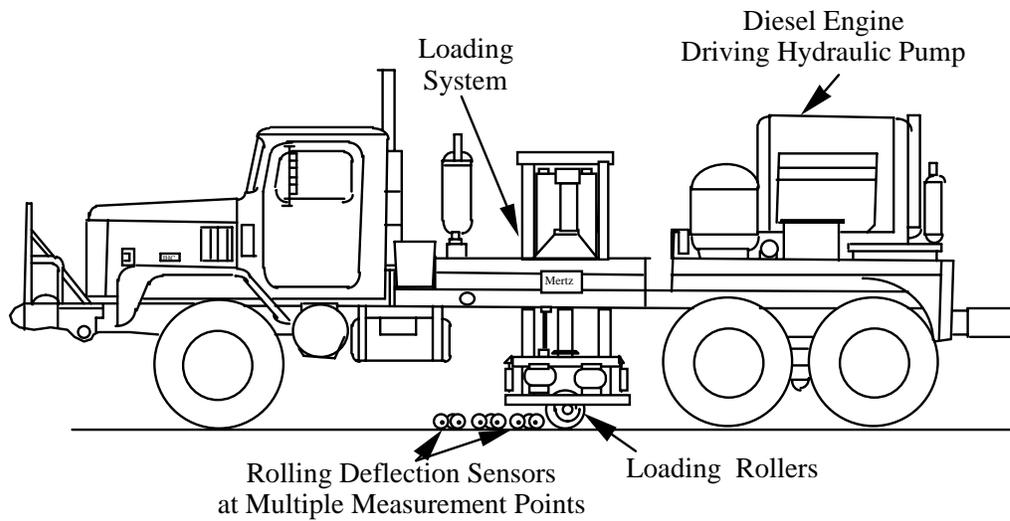
## INTRODUCTION

Rehabilitation of aged jointed concrete pavements (JCP) with an asphalt concrete (AC) overlay is problematic, and historically such overlays have been prone to reflective cracking. Although reflective cracking has been studied for a long time, it is still occurring and is responsible for millions of dollars per year in damage. Reflective cracking is defined as the propagation of an existing crack or joint upward to the new pavement surface. Reflective cracking is one of the most common causes of deterioration in overlay systems (Al-Qadi and Baek 2006). The main mechanisms leading to the development of reflective cracking are the differential vertical movements between concrete slabs and the brittleness of the overlay mix, which lacks the ability to resist the differential vertical movements. To minimize the potential for reflective cracking and maximize the success of a JCP overlay project, knowledge of the existing pavement condition and overlay material is vital. Thus, engineers need tools to quantify the severity of the cracks (or joints) to determine the potential for reflective cracking and to measure the resistance of the overlay material to underlying crack or joint movements.

Continuous deflection measurement provides valuable information for determining joint and crack conditions of an existing roadway. Research results (Scullion 2005; Chen et al. 2007; Chen 2008) have clearly demonstrated the capabilities and benefits of the rolling dynamic deflectometer (RDD) in providing continuous deflection data over entire projects. Figure 1 shows a schematic view of the RDD and sensor arrangement.

The RDD is the only operational project-level rolling deflection device in the world that is able to identify problematic areas and to provide joint and crack conditions for rehab strategy optimization. Continuous deflection profiles provide 100 percent coverage and permit pavement engineers to evaluate the entire project, locating sections where more extensive repairs are needed. Chen (2008) and Chen et al. (2007) have used the RDD to locate joints and active cracks with poor load transfer that need to be repaired before overlaying. The RDD typically runs about 1 to 2 mi/h (1.6 to 3.2 km/h), depending on the surface texture and noise transmitted to the rolling sensors. Even at this rate of collection, the RDD is far more efficient at collecting data on all of the cracks and joints than a falling-weight deflectometer (FWD) would be. The FWD machinery has a much lower overhead than the RDD and is better for collecting data at discrete points, typically 0.1 mi (0.16 km) apart. But the FWD would be much slower at collecting data comparable to RDD data, because it would have to stop for about 1 minute every 5 to 15 ft (1.5 to 4.6 m). The FWD has been used on a few of the same projects as the RDD, and the deflections are comparable when normalized to equivalent loads.

The overlay tester (OT) has been applied with success to characterize asphalt mixes' ability to resist reflective and fatigue cracking (Zhou and Scullion 2005; Zhou et al. 2007). Based on extensive laboratory and field performance observation, Zhou and Scullion (2005) found that the OT results matched very well with field cracking. When an asphalt mix lasts more than 700 cycles in the OT, it is thought to have good fatigue resistance (Zhou et al. 2007). Figure 2 shows a comparison of ability to resist reflective cracking for materials A and B after 2 years of tracking. While materials A and B were placed at the same time above the same JCP pavement structure, material A (with OT life exceeding 700 cycles) does not have reflective cracks. There are reflective cracks in the lane with material B because the mix is brittle. The OT life of material B was only 20 cycles. This example clearly demonstrates the importance of mix properties in preventing reflective cracks.



**Figure 1. Schematic and general rolling dynamic deflectometer arrangement with typical rolling sensor configuration.**



OT = overlay tester

**Figure 2. Comparison of ability to resist reflective cracking for materials A and B after 2 years of traffic.**

Finding the areas that exhibit poor load transfer and significant slab movement (areas of high potential for reflective cracking) is critical for rehabilitating concrete pavements. Otherwise, reflective cracking and localized failures occur within months of rehabilitation (Amini 2005). The relationships between the RDD, OT, and field performance data from condition surveys are critical for optimum rehabilitation strategy selection. RDD deflection thresholds for identifying areas of high risk of reflective cracking can be established, validated, and improved empirically through field performance monitoring. Also, the ability of different mixtures to resist reflective cracking can be quantified and evaluated under various moving-slab conditions. An increasing number of rehabilitation projects in Texas have been completed using data from both the RDD and OT. The field performance results to date have been very promising. In this study, three highway sections (IH-20, SH-12, US-96,) are presented to demonstrate the benefits of utilizing the RDD and OT for minimizing the reflective cracks.

## **CASE STUDIES**

### **Interstate Highway 20**

The potential of utilizing the RDD for concrete pavement rehabilitation was first realized in Texas in early 2000. The RDD was employed to provide recommendations on the location of repairs needed prior to an intended 4-in. (102-mm) overlay. RDD results indicated that hundreds of locations had significant vertical movements, and needed repairs before the AC overlay. However, due to funding limitations, only 10 locations were repaired before the 4-in. (102-mm) AC overlay was placed. After less than 1 month's trafficking, many locations had reflected cracks. After a couple years of trafficking, many patches had to be done to maintain the ride quality because of the severe reflection cracks. Efforts were made in this study to monitor and correlate the crack locations to the RDD results. The monitoring results show a very strong correlation between reflective cracks and patch locations and the high vertical movements measured by the RDD.

IH-20 is one of the busiest interstate highways in Texas. The estimated 20-year design traffic for this pavement is 87.2 million equivalent single-axle loads (based on 2004 traffic data). After years of heavy traffic, this section of highway needed to be rehabilitated. The main reason for the rehabilitation on IH-20 was the severe transverse cracking that caused poor ride quality. The original typical section consisted of 4 in. (102 mm) of AC overlay; 8 in. (203 mm) of continuously reinforced concrete pavement (CRCP); 7 in. (178 mm) of cement-treated base (CTB); 6 in. (152 mm) of CTB subbase; and 6 in. (152 mm) of select material. Although it is a CRCP, the pavement acts like a JCP in many places because the steel reinforcement was ruptured (as was evident during the slab repair). Note that before the rehabilitation, there were already many locations with full-depth repairs intended to maintain structural integrity. The rehabilitation scheme was to first mill off the 4-in. (102-mm) surface AC overlay, then do full-depth repairs to the CRCP at selected locations, followed by placing a new 4-in. (102-mm) AC overlay. The rehabilitation was completed in November 2002.

With reference to the RDD deflection sensors shown in Figure 1, research results (Chen et al. 2007; Chen 2008) have demonstrated that  $W1-W3$  is a good indicator for reflective cracking potential, especially in areas where  $W1-W3$  exceeds 6.5 mils on an AC surface without milling.  $W1-W3$  is usually at a maximum when sensors 1 and 3 are on either side of a joint. A higher  $W1-W3$  value is interpreted as a poorer load transfer across a joint or crack.  $W1-W3$  deflections for IH-20 (eastbound lane, before milling) are presented in Figure 3. The unit used on the Y-axis (and hereafter) is mils/10 kips, which represents the measured deflection in mils under 10 kips of peak dynamic force. As shown in Figure 3, numerous locations have significant spikes where the  $W1-W3$  deflection exceeds 6.5 mils. These spikes indicate locations with poor load transfer or significant relative vertical movements.

Figure 4 illustrates the pavement condition before milling, after milling, and 1 year after the 100-mm AC overlay. Even though there were significant vertical movements, no full-depth repairs were performed at this location, and cracks reflected through within 1 year. In fact, numerous reflective cracks were observed on IH-20 after only a few days. The lab results on the cores taken from IH-20 indicated that the 100-mm AC overlay was stiff and rut resistant, but had relatively poor crack resistance. Under the Hamburg Wheel test, the AC overlay had rutting of less than 12.5 mm after 20,000 cycles at 50C. The AC cores were tested with the OT, and it only took 2 cycles to fail the specimens. This means the AC overlay is very brittle and prone to crack. After traffic resumed, numerous locations like those shown in Figure 4 were observed, and many patches had to be placed in 25 months or less. The distress condition survey indicated that there were 90 locations (63 reflected cracks plus 27 patches) with distress after 25 months.

The conclusions from the IH-20 projects are as follows: (1) many areas with very poor load transfer and significant slab movements were not repaired before placing the 4-in. (102-mm) overlay; (2) a brittle mix with an OT result of 2 cycles was placed on a pavement with high potential for reflective cracking; (3) these factors led to very early and severe reflective cracks.

Two more projects were monitored to evaluate the proposed criteria for the RDD deflections and OT results. The tentative threshold values beyond which performance problems are anticipated are  $W1-W3$  deflections of 5.5 on exposed concrete surface or 6.5 mils on an asphalt surface with underlying concrete pavement; and, to minimize reflection cracking potential, the proposed overlay should last longer than 700 cycles in the OT.

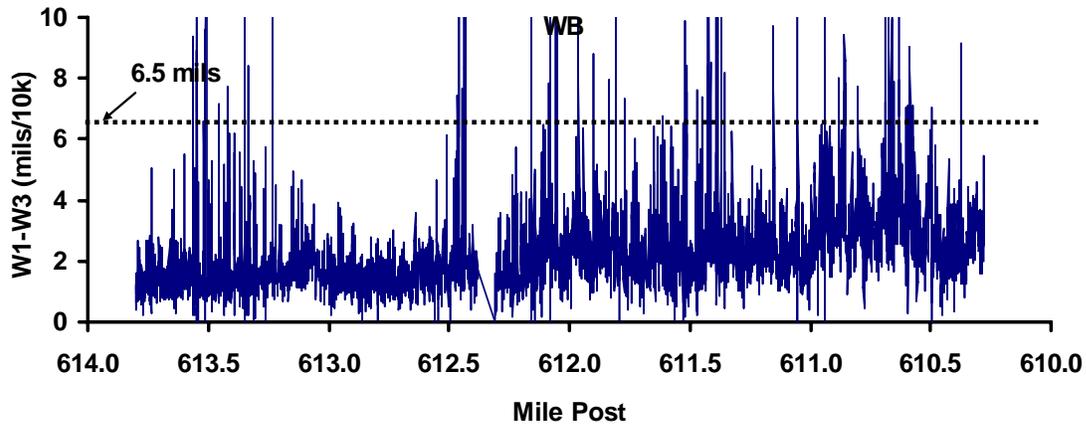


Figure 3. W1–W3 continuous deflections before milling for IH-20. Spikes indicate poor load transfer or significant vertical movements that indicate a high potential for reflective cracking.

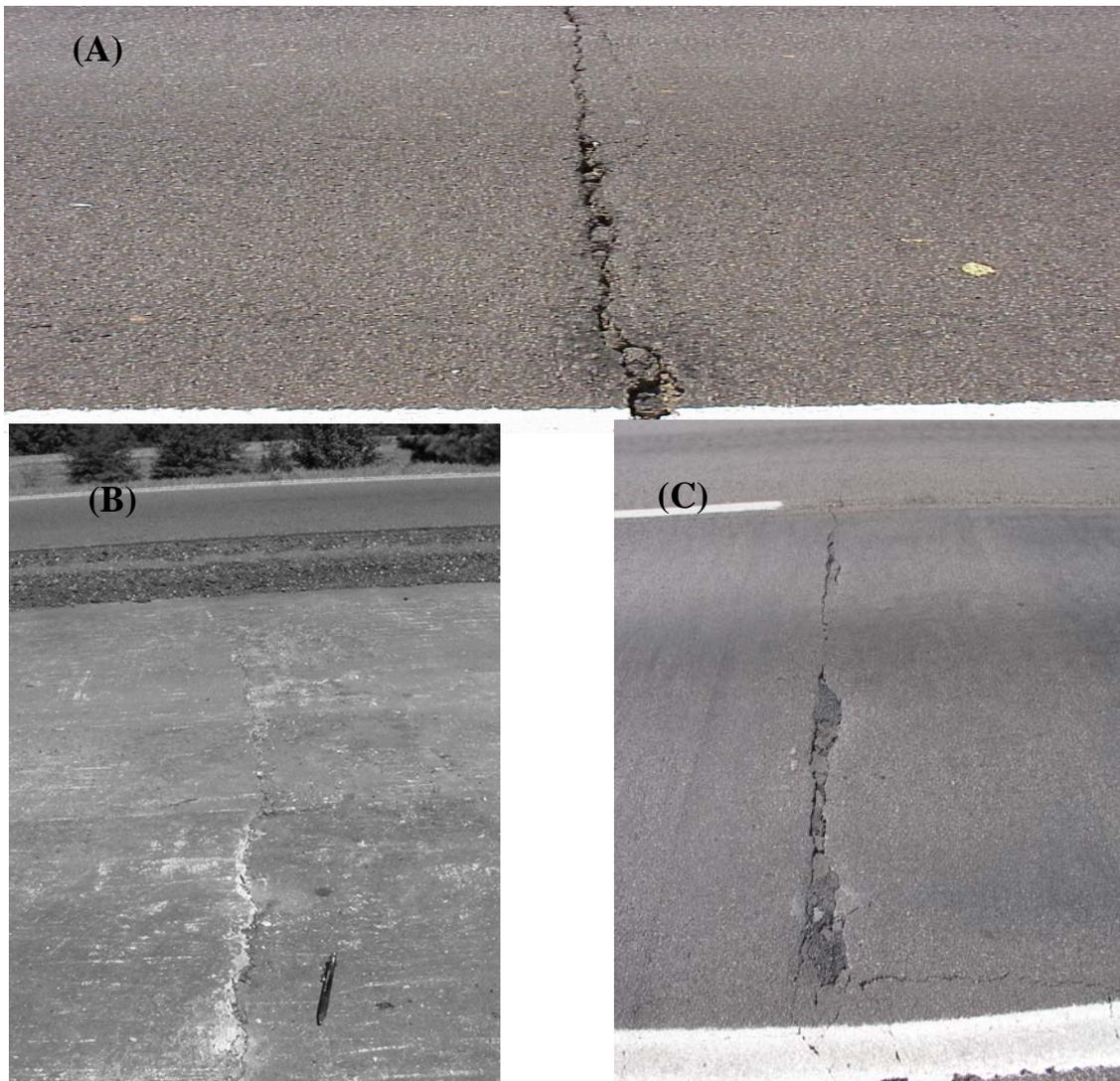


Figure 4. Pavement conditions along IH-20 (A) before milling, (B) after milling and before overlay, and (C) with reflected cracks 1 year after overlay.

## State Highway 12

The SH-12 project demonstrates a successful application of the RDD and OT to minimize the reflective cracks on a JCP that had poor load transfer and significant slab movements. The construction records indicated that the last rehabilitation (an overlay of 3 in. [76 mm] of AC) was completed in 2001. However, the AC was unable to resist the reflective cracking, as shown in Figure 5A. Due to the cracking and poor ride quality, SH-12 needed to be rehabilitated in 2006. The 2006 rehabilitation consisted of milling the existing AC overlay; repairing the JCP; and overlaying it with a 1-in. (25-mm) rich bottom layer (RBL) AC, 2 in. (51 mm) of Type D AC, and 1.5 in. (38 mm) of porous friction course (PFC) wearing surface. RBL has been tested with the OT and has a life exceeding 900 cycles, when the test was terminated. The RBL was placed in June 2006. The typical features of RBL are fine gradation, use of high quality aggregates, and high binder content. Typically the binder content for RBL ranges from 6 percent to 8.5 percent. The RBLs in Texas are designed to pass both the prevailing Hamburg wheel tracking test (for rutting and moisture susceptibility) and the OT for crack resistance. Efforts have been made to monitor the section and track the long-term performance.

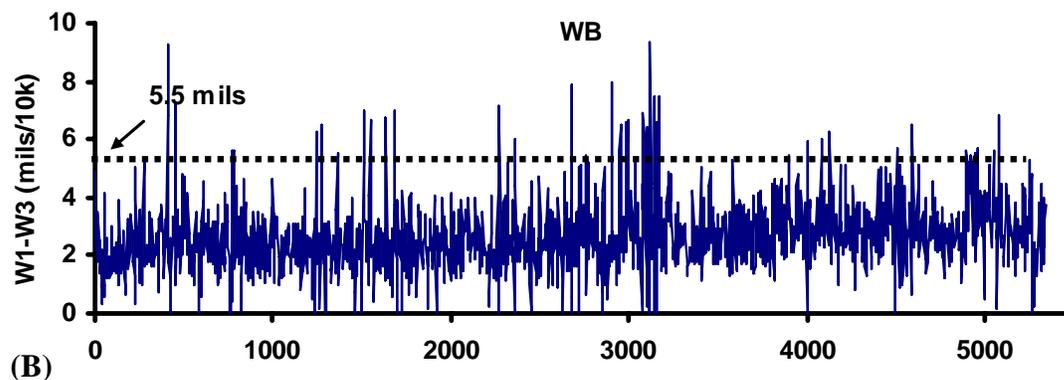
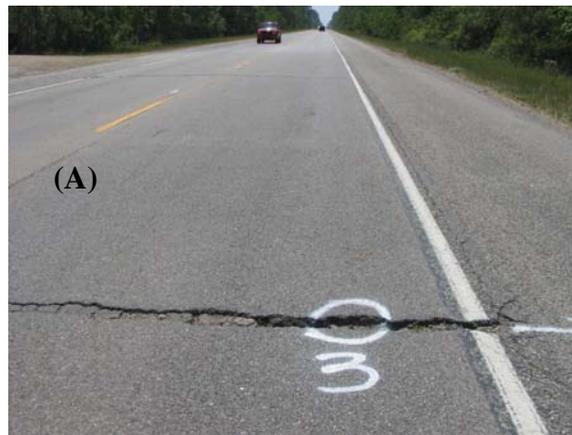


Figure 5. Pavement conditions and rolling dynamic deflectometer (RDD) measurements on SH-12: (A) pavement conditions before milling; (B) RDD continuous profiles after milling.

A typical continuous deflection profile collected on SH-12 is shown in Figure 5. RDD tests were performed on the milled JCP surface. Research results (Chen et al. 2007; Chen 2008) indicated that exposed concrete surfaces with a  $W1-W3$  deflection exceeding 5.5 mils indicates a high potential for reflective cracking.

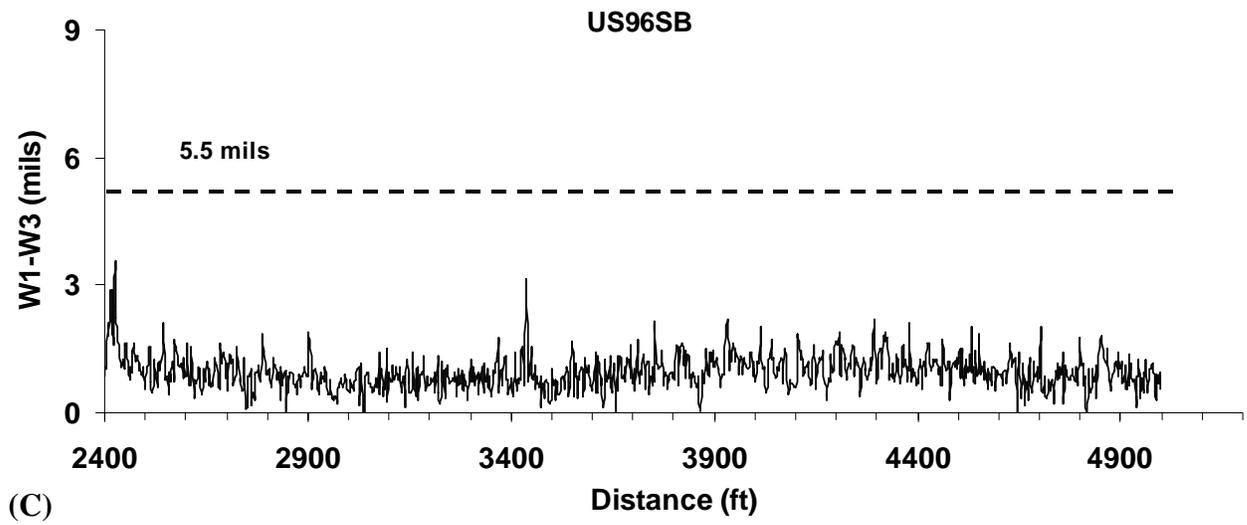
The average IRI before the 2006 rehabilitation was approximately 110 in/mi (1,760 mm/km). The 2006 rehabilitation effectively reduced the IRI to approximately 52 in/mi (832 mm/km). The IRI has remained at approximately 60 in/mi (960 mm/km) for the last 2 years. A condition survey conducted in January 2009 indicated no visible reflective cracks. This demonstrates that even with significant slab movements, the treatment of 1-in. (25-mm) RBL AC, 2-in. (51 mm) Type D AC, and 1.5-in. (38 mm) PFC was able to deter the reflective cracking. The main reasons for good performance on SH-12 are the flexible RBL mix, thicker overlay (4.5 in. [114 mm]), and the porous wearing surface mix with a high air void of approximately 20 percent. Continuous monitoring of the section will be conducted to determine when the reflective cracks appear.

### **U.S. Highway 96**

A 40-year-old CRCP on US-96 was being considered for major rehabilitation for the first time. Extensive spalling of the original CRCP near the transverse cracks was evident. The spalling was found to be limited to the top 50 mm. The pavement consists of 8 in. (203 mm) of CRCP and 4 in. (102 mm) of CTB. The roadway condition before rehabilitation is shown in Figure 6A. A candidate rehabilitation strategy proposed by the District consisted of placing an SMA overlay, 3 in. (76 mm) thick. The main concern with the placement of the overlay was the potential for reflective cracking. Since the reflective cracking is highly related to movement of the slabs at the cracks, the District requested an RDD survey to assess the support under the slabs at the cracks, as shown in Figure 6B. The continuous deflection profile collected by the RDD at this site is shown in Figure 6C. In view of Figure 6C, it was found that no cracks had  $W1-W3$  deflection exceeding 5.5 mils, indicating good load transfer and little reflective cracking potential.

The District placed the SMA overlay approximately 5 years ago, and the performance so far has been excellent with no visible cracks. SMA cores were taken and tests were conducted by the OT. The specimens yielded more than 700 cycles to failure, as the OT was terminated at 700 cycles and no crack was observed at that time.

It was believed that the success of the project on US-96 is due to the combination of the low potential of the CRCP slab for reflective cracking, as determined by the RDD, and the excellent properties of the SMA overlay for resisting reflective cracking, as determined by the OT.



**Figure 6. (A) Pavement condition of US-96 before rehabilitation with severe cracking and spalling; (B) rolling dynamic deflectometer testing (RDD); (C) RDD continuous deflection profile (on top of exposed concrete surface) for southbound direction.**

## CONCLUSIONS

The combination of the RDD and the OT was used on three concrete pavement rehabilitation projects in Texas. The RDD was used to rate the suitability of each project for an asphalt overlay, based on joint movement. The OT was applied to characterize each mix's ability to resist reflective cracking. The threshold values (a)  $WI-W3 = 6.5$  mils for a composite pavement with an old AC overlay, (b)  $WI-W3 = 5.5$  mils for exposed concrete pavements, and (c) 700 cycles from the OT were employed to evaluate three field rehabilitation projects (IH-20, SH-12, US-96). Conclusions and observations are given as follows:

1. On IH- 20 there were many locations that had  $WI-W3$  deflections exceeding 6.5 mils. After 25 months of field monitoring, there were 63 reflected cracks (many severe) and 27 patches. The RDD data collected prior to the overlay identified the areas of high reflective cracking potential. The mix used on IH-20 was very brittle, as core samples failed in 2 cycles when they were tested by the OT. Brittle mix on pavement with high reflective cracking potential leads to premature failure. This demonstrates the ability of the RDD and OT to characterize the existing pavement condition and the overlay materials.
2. No visible cracks have been observed on the SH-12 project after 2 years of service, despite the significant movement ( $WI-W3$  deflection  $> 5.5$  mils) detected by the RDD. The main reasons for the good performance on SH-12 are the flexible RBL mix, the thicker overlay (4.5 in. [115 mm]), and the porous wearing surface mix with a high air void of approximately 20 percent.
3. Although there were severe transverse cracks and spalling on US-96, all  $WI-W3$  deflections were less than 5.5 mils (at low risk for reflective cracking). SMA was placed 5 years ago, and the performance is excellent. SMA was tested with the OT, and its life exceeded 700 cycles. It was believed that the success of the US-96 project due to the combination of the CRCP's low potential for reflective cracking and the SMA's excellent properties for resisting reflective cracks.
4. Through case studies, the combination of RDD and OT testing, and the use of three threshold values show excellent potential to address problem of reflective cracking.

The RDD is now being widely used on most concrete pavement rehabilitation projects in Texas. The data are used by designers to identify entire problem areas where corrective action is needed before placement of the overlay. In other projects, localized problems (loose joints) have been identified and rehabilitated with retrofitted dowels before the overlay. In another recently completed project, the RDD data is one input into a new TxDOT overlay design program that is currently under evaluation (Zhou 2008). The overlay tester is now a standard test in Texas (Tex Method 248E), and the overlay tester criteria have now been incorporated into standard mix design specifications (CAM SS 3155, 2008).

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## ACKNOWLEDGMENTS

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# **Impact of Existing Pavement on Jointed Plain Concrete Overlay Design and Performance**

Michael I. Darter, Jag Mallela, and Leslie Titus-Glover<sup>1</sup>

## **ABSTRACT**

Concrete overlays are increasingly being constructed over deteriorated existing asphalt and concrete pavements. Designers struggle to consider the extent of deterioration of the existing pavement in the design of the concrete overlay. This paper addresses the impact of the level of condition of the existing pavement on the performance of the concrete overlay. Use is made of the new AASHTO Interim Mechanistic-Empirical Pavement Design Guide to simulate two case studies over a range of conditions and designs. Significant findings were obtained to help guide designers to better consider the condition of the existing pavement in their design.

## **1.0 INTRODUCTION**

Existing pavements that are candidates for concrete overlays vary widely in design and condition. Many engineers believe that these factors have an effect on the subsequent performance of concrete overlays. This paper presents some results using the new AASHTO Interim Mechanistic-Empirical Pavement Design Guide (MEPDG) on how different designs and conditions affect the performance and consequently the design of concrete overlays. Existing designs include various types of asphalt pavements, concrete pavements, and composite pavements. These pavements can range in condition from fair to very poor or severe.

Thus, there is a wide matrix of designs and conditions facing designers of concrete overlays. In addition, site conditions including climate, subgrade, and traffic level also contribute to the challenge of providing an economical and reliable concrete overlay design over a future design period. This paper develops a matrix of designs and conditions of existing pavements and then demonstrates the impact of the existing pavement conditions on the performance of jointed plain concrete pavement (JPCP) overlays using the AASHTO Interim (MEPDG) models. Based on the results obtained, recommendations are prepared for assisting designers to provide more reliable concrete overlay designs for widely varying existing pavement conditions.

This paper first describes the capabilities of the MEPDG to model concrete overlays, outlines the various design considerations for concrete overlays, and describes two case studies: one for JPCP overlay of an existing hot-mix asphalt (HMA) pavement and another for JPCP overlay over an existing JPCP. The paper concludes with a summary of findings and recommendations for design based on the MEPDG results.

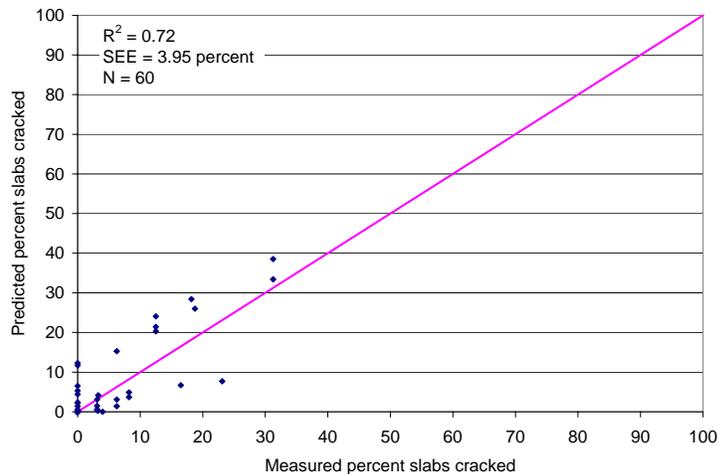
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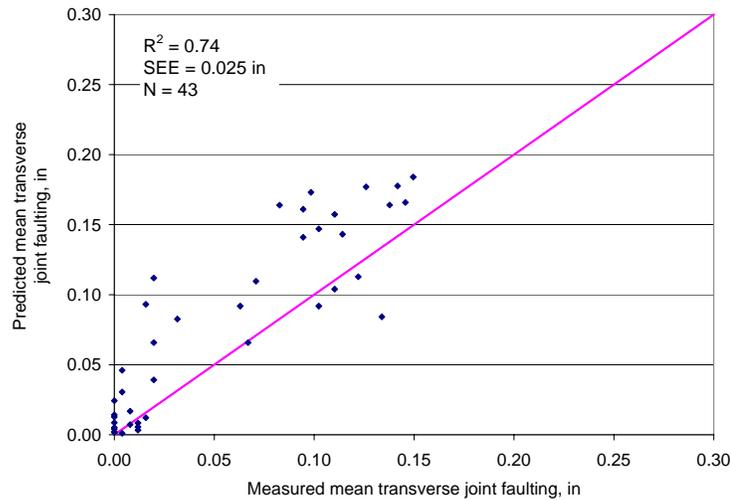
## 2.0 CAPABILITIES OF THE AASHTO INTERIM MEPDG TO MODEL CONCRETE OVERLAYS

The MEPDG was developed to model JPCP and continuously reinforced concrete pavement (CRCP) overlays of existing concrete pavements, asphalt pavements, and asphalt over concrete pavements. The ISLAB2000 finite element structural model includes layers for the concrete overlay, a separation layer, the existing HMA layer or a concrete slab, existing base and sub-base layers of any type, the embankment and or subgrade, and finally bedrock. The MEPDG models the condition of the existing concrete or HMA pavement through its effective “modulus,” which has been adjusted to reflect existing crack damage. The concrete overlay can directly consider all of the normal features of a new concrete JPCP or CRCP including such features as thickness, transverse joint spacing, slab width, joint load transfer, tied shoulders, portland cement concrete (PCC) material properties, friction between slab and HMA, and many other factors. The key distress types that are predicted by ME-based models include joint faulting, slab transverse cracking, and International Roughness Index (IRI) for JPCP. For CRCP, crack width, crack load transfer efficiency (LTE), punchouts, and IRI are predicted.

The calibration of the MEPDG for both JPCP and CRCP included a number of field sections across North America. The global cracking and faulting calibration models completed under NCHRP 1-40D are shown in Figures 1 and 2. The calibration shows very good prediction of these key performance indicators for a wide variety of concrete overlays. The mechanistic basis and the field validation give some confidence that the MEPDG can reasonably model concrete overlays that are 6-in. (152 mm) or thicker.



**Figure 1. National calibration plot for transverse cracking of unbonded JPCP overlays.**



**Figure 2. National calibration plot for joint faulting of unbonded JPCP overlays.**

### **3.0 DESIGN CONSIDERATIONS RELATED TO THE EXISTING PAVEMENT**

Design considerations include many different factors that are included in the design and construction specifications of concrete overlays. A brief summary of these considerations is provided.

- Design features of the existing pavement. The type of pavement, the layer thicknesses, other design aspects such as joints and shoulders are critical factors that must be considered. The appropriate modulus and thickness of each layer must be estimated for overlay design.
- Material types and their appropriate modulus for each layer in the existing pavement.
- Treatment to the existing surface for unevenness, if any, is another consideration. Significant unevenness of this surface must be corrected prior to concrete paving through milling or a level-up HMA layer. Correcting an uneven surface should not be left to the paver to adjust thickness of the concrete overlay.
- Friction/Bonding of the concrete overlay to the underlying HMA layer. Full friction is beneficial to provide for joint formation, reduction in erosion, and improved structural capacity. The existing HMA surface may need to be milled to provide long-term friction.

### **4.0 DESIGN CONSIDERATION OF THE CONCRETE OVERLAYS**

There are several critical design features that must be specified to have a successful construction and performance of a JPCP overlay. This section summarizes these factors, most of which are considered directly in the MEPDG.

- Dimensions of the overlay slab. Required dimensions include thickness, transverse joint spacing, longitudinal joint spacing, and shoulder edge support (tied concrete, widened slab, non-supportive).

- Joint design of overlay: load transfer details (bar size, spacing, embedment) and tie bar spacing and size.
- Overlay slab materials. Strength, modulus of elasticity, coefficient of thermal expansion, and shrinkage of the concrete are critical.
- Interface friction between the slab and HMA layer below the slab is critical.
- Climate and subdrainage considerations.
- Subgrade support.
- Traffic loadings that the overlay must support over the design period. There are many aspects of traffic related to volume of each type of truck, growth rate, lane distribution, axle types, axle load distribution, and other factors.
- Construction, including month during which the concrete overlay will be placed, time between placement and opening to traffic, and curing of the concrete to avoid built in temperature gradient.

## **5.0 CASE STUDY A: ILLUSTRATION OF JPCP CONCRETE OVERLAY OVER EXISTING HMA PAVEMENT CONDITION**

Case Study A is located near Topeka, Kansas, on I-44 where a many times overlaid HMA pavement was considered for a concrete overlay. The following are defined for the existing pavement condition and thickness, concrete overlay design, and friction between the concrete overlay and HMA layer.

- Existing HMA pavement: milled HMA layer thickness (4 in. [102 mm] after milling), 10-in. (254-mm) granular base course, 10-in. (254-mm) gravel sand subbase.
- Friction between the concrete slab and existing HMA: full friction (milled surface) over the 30-year analysis period due to the milling of the HMA.
- Existing HMA condition: “Fair” (15-35 percent), “Poor” (35-50 percent), “Very Poor” (>50 percent) alligator cracking for all severity levels. As fatigue cracking increases, the dynamic modulus ( $E^*$ ) of the HMA layer decreases resulting in a decrease in structural capacity of the overlaid pavement. The mean  $E^*$  computed by the MEPDG over the entire year for each level of fatigue damage is given in Table 1.
- Overlay design: thickness and joint spacing (6.5 in. [165 mm] with 12 ft [3.7 m] and 8.0 in. [203 mm] with 15 ft [4.6 m]), slab width (12-ft [3.7-m] slab), transverse joint load transfer (both dowels and no dowels).
- Site conditions: Topeka, Kansas, climate, A-6 subgrade soil, and I-44 traffic (two-way truck traffic is 1,350 with a 2.3 percent linear growth rate results in 8.4 million trucks (14 million equivalent single-axle loads [ESALs]) in the design lane over 30 years). Slab placed in September and opened to traffic in 1 month after placement.

**Table 1**  
**HMA Pavement Condition and Mean E\* Over the Entire Year From MEPDG**

<b>Existing HMA Condition</b>	<b>Percent Alligator Crack- ing Lane Area</b>	<b>Mean HMA E* Over the Entire Year (lb/in<sup>2</sup>)</b>
Fair	15 to 35	1,094,000
Poor	35 to 50	537,000
Very Poor	>50	314,000

The MEPDG was run for these specific conditions over a 30-year analysis period. The predicted performance at the end of the 30-year analysis period is given in Table 2. Additional MEPDG solutions were run for other conditions including wider slab, increased HMA thickness, and reduction in friction between the slab and HMA layer, with results in Table 3.

**Table 2**  
**Impact of JPCP Overlay Design Features and HMA Pavement Condition**  
**(Fair, Poor, Very Poor) on Performance of Concrete Overlay**  
**30 Years, 4-in. HMA, Full Friction Between Slab/HMA, 12-ft-wide Lane**

<b>Existing HMA Pavement Condition</b>	<b>PCC Slab: 6.5-in., 12-ft Joint Spacing</b>		<b>PCC slab: 8-in., 15-ft Joint Spacing</b>	
	<b>Nondoweled Joint</b>	<b>Doweled Joint (1-in. diameter)</b>	<b>Nondoweled Joint</b>	<b>Doweled Joint (1-in. diameter)</b>
“Fair” Condition (15–35% alligator cracking)	C = 8 % slabs F = 0.19-in IRI = 221 in/mi	C = 8 F = 0.04 IRI = 123	C = 6 F = 0.19 IRI = 189	C = 6 F = 0.12 IRI = 154
“Poor” Condition (35–50% alligator cracking)	C = 53 F = 0.19 IRI = 260	C = 53 F = 0.04 IRI = 160	C = 29 F = 0.19 IRI = 208	C = 29 F = 0.12 IRI = 173
“Very Poor” Condition (>50% alligator cracking)	C = 78 F = 0.19 IRI = 280	C = 78 F = 0.04 IRI = 180	C = 49 F = 0.19 IRI = 225	C = 49 F = 0.12 IRI = 190

C = percent slab transverse cracks at 30 years and 8.4 million trucks

F = mean joint faulting, inches at 30 years and 8.4 million trucks

IRI = International Roughness Index, in/mile at 30 years and 8.4 million trucks

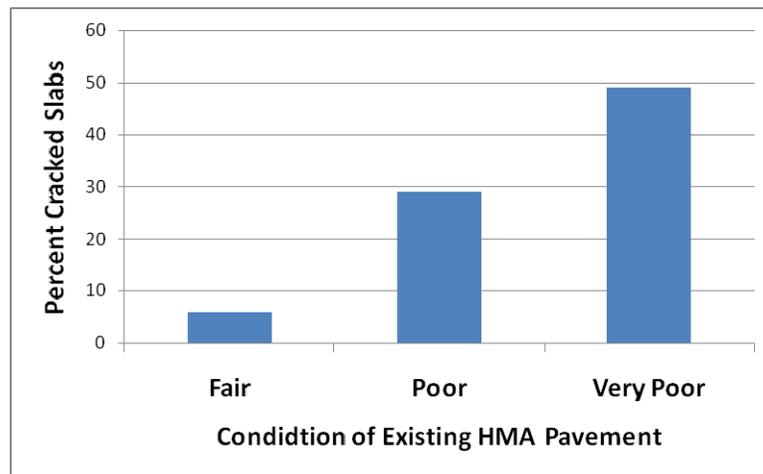
**Table 3**  
**Impact of JPCP Overlay Design Features and HMA Pavement Condition**  
**of Concrete Overlay Over 30 Years, 4-to-6-in. HMA Thickness Variation,**  
**Slab/HMA Friction Variation, 13-ft-wide Traffic Lane Slab**

<b>Existing HMA Pavement Condition</b>	<b>PCC Slab 6.5-in 12-ft Joint Spacing</b>		<b>PCC slab 8-in 15-ft Joint Spacing</b>	
	<b>Nondoweled Joint</b>	<b>Doweled Joint (1-in. diameter)</b>	<b>Nondoweled Joint</b>	<b>Doweled Joint (1-in. diameter)</b>
Special Case: Poor HMA Condition with 6-in. HMA existing thickness	C = 9 F = 0.19 IRI = 217	C = 9 F = 0.04 IRI = 121	C = 8 F = 0.18 IRI = 187	C = 8 F = 0.12 IRI = 153
Special Case: Poor HMA Condition with zero friction between JPCP and HMA existing thickness after 60 months	C = 88 F = 0.19 IRI = 288	C = 88 F = 0.04 IRI = 188	C = 58 F = 0.19 IRI = 232	C = 58 F = 0.12 IRI = 197
Special Case: Poor HMA Condition with 13-ft slab width outer lane	C = 1 F = 0.13 IRI = 178	C = 1 F = 0 IRI = 89	C = 1 F = 0.13 IRI = 161	C = 1 F = 0 IRI = 87

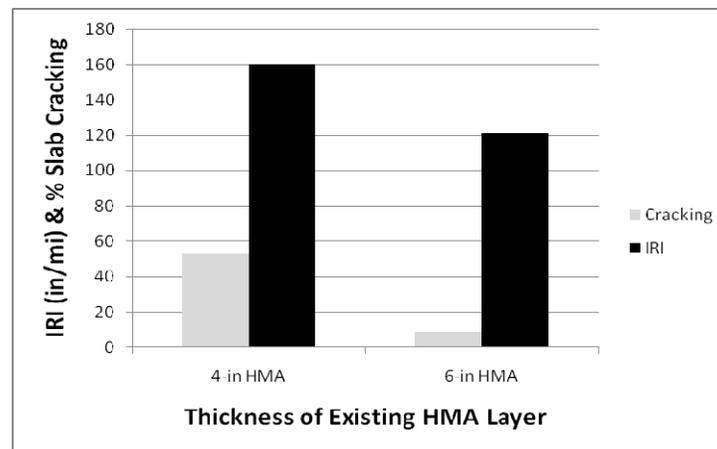
C = percent slab transverse cracks at 30 years and 8.4 million trucks  
F = mean joint faulting, inches at 30 years and 8.4 million trucks  
IRI = International Roughness Index, in/mile at 30 years and 8.4 million trucks

The following results were obtained from these analyses of Case Study A.

- As existing HMA pavement condition varies from “Fair” to “Poor” to “Very Poor,” the performance of the JPCP overlay is greatly affected as shown in Table 2 and Figure 3 for the 6.5-in. (165-mm) JPCP. A similar effect occurs for the 8-in. (203-mm) JPCP overlay. The reason for this increase in slab fatigue cracking of the overlay is the loss in structural capacity of the pavement structure that includes the existing HMA layer. The damaged E\* of the HMA layer is reduced as in Table 1. The slab and HMA layer were assumed to have full friction over the 30 years analysis period, and thus the equivalent slab would be a function of the E\* of the HMA as well as the E of the concrete.
- The thickness of the existing milled HMA layer was varied from 4 in. (102 mm) to 6 in. (152 mm) for a “Poor” HMA pavement condition and dowel bars in the overlay to see if it had a significant effect on performance. The MEPDG prediction shows a major impact on cracking and IRI for the 6.5-in. (165-mm) JPCP (cracking decreased from 53 to 9 percent slabs cracked) and for the 8-in. (203-mm) JPCP (29 to 8 percent) as shown in Figure 4. The IRI was also reduced as shown in Figure 4. This effect again shows the significant impact of the existing HMA layer on performance of the concrete overlay.

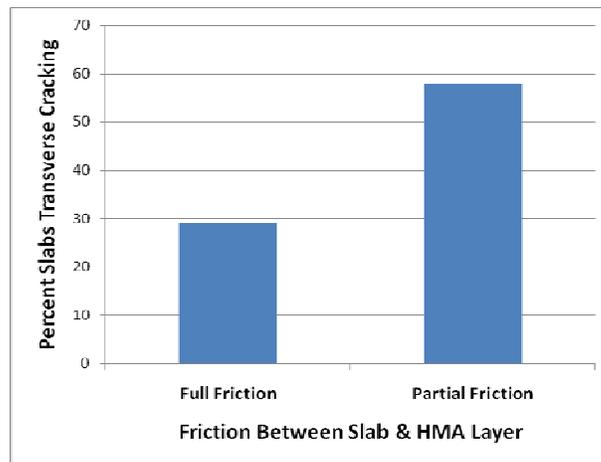


**Figure 3. Effect of condition of existing 4-in. (102-mm) HMA pavement alligator cracking on 6.5-in. (165-mm) JPCP concrete overlay slab cracking after 30 years and 8.4 million truck loadings (14 million ESALs).**



**Figure 4. Effect of “Poor” HMA layer thickness on IRI and transverse cracking of the JPCP overlay.**

- If the HMA surface is milled, it will have high friction with the JPCP slab. Friction between the concrete overlay and existing HMA would likely be full over the entire analysis period as found in the national calibration. The MEPDG was run for the 8-in. (203-mm) JPCP overlay, assuming no milling, and thus the friction between the two layers would likely be less. The friction was set to zero after 60 months of the analysis period. Figure 5 shows that with zero friction after 60 months there develops twice as much fatigue cracking in the overlay after 30 years. Again, this is caused by a decrease in structural capacity when full friction does not exist between layers.



**Figure 5. Effect of Slab/HMA friction on slab cracking (partial means no friction after 60 months).**

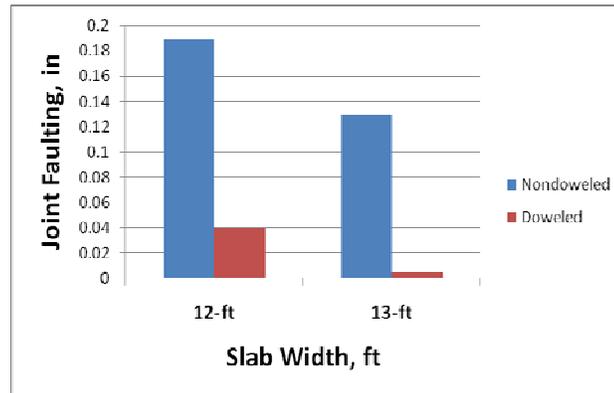
- The thickness and joint spacing of the JPCP overlay are known to have significant effects when changed individually. This analysis was conducted with both changing together (joint spacing increases with increasing slab thickness), which would be typical of practice, but the results are more complex to understand. Table 4 shows the average cracking, faulting, and IRI for “Poor” condition of the existing HMA pavement. The 6.5-in. (165-mm) JPCP overlay with 12-ft (3.7-m) joint spacing does show greater cracking than the 8.0-in. (203-mm) JPCP overlay with 15-ft (4.6-m) joint spacing. However, the longer joint spacing has more effect on slightly increasing faulting due to increased joint opening. The IRI is lower for the slab 8 in. (203 mm) thick with 15-ft (4.6-m) joint spacing.

**Table 4  
Performance of the 6.5-in. and 8.0-in. Slabs for “Poor” Condition  
of Existing HMA Pavement After 30 Years**

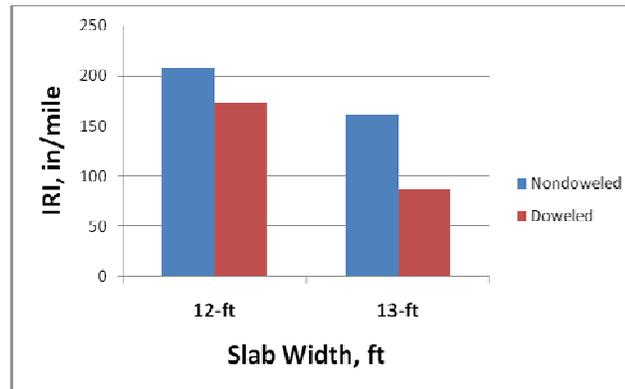
Slab Thickness & Joint Spacing	Transverse Slab Cracking (%)	Joint Faulting (in.)	IRI (in/mi)
6.5-in. & 12-ft	53	0.11	210
8.0-in. & 15-ft	29	0.15	190

- Use of dowels at transverse joints shows a major impact on joint faulting and IRI and no effect on transverse cracking. Figure 6 shows that with dowels, joint faulting is far lower than without dowels (0.04 versus 0.19-in.) over the 30-year analysis period for slabs either 12 ft (3.7 m) or 13-ft (4.0 m) wide. Note that the truck traffic in the design lane is significant at 8.4 million trucks (or 14 million ESALs), and the project is located in cold climate.
- The widening of the slab from 12 ft (3.7 m) to 13 ft (4.0 m) has a profound effect on the performance of the concrete overlay. Cracking, faulting, and IRI are all greatly reduced with a 1-ft (0.3-m) widening of the traffic lane slab that moves some of the heavy truck wheels away from the free edge. Of course, the paint stripe must be placed at the normal

12-ft (3.7-m) location. Cracking goes essentially to zero. Figure 6 shows the impact on faulting, and Figure 7 the impact on IRI.



**Figure 6. Impact of concrete overlay slab width on doweled and nondoweled joint faulting (for 8-in. JPCP) using the MEPDG.**



**Figure 7. Impact of concrete overlay slab width on doweled and nondoweled IRI (for 8-in. JPCP) using the MEPDG.**

## **6.0 CASE STUDY B: ILLUSTRATION OF JPCP CONCRETE OVERLAY OVER EXISTING CONCRETE PAVEMENT**

Case Study B is located near Aurora, Illinois (west of Chicago), on I-88 where an old JPCP with HMA overlay exists. The following conditions were defined for the existing pavement condition and thickness, concrete overlay design, and separation layer.

- Existing HMA overlay: A 4-in. (102-mm) HMA overlay exists that is badly cracked and weathered. It will be removed through milling to minimize the added height of the concrete overlay.
- Existing JPCP: Slab thickness 10 in. (254 mm), 12-in. (305-mm) granular base course, 6-in. (152-mm) gravel sand subbase.

- Existing JPCP Condition: the condition will be varied from “Good” (10 percent slabs cracked), “Moderate” (20 percent), and “Severe” (50 percent) transverse cracking of all severity levels to illustrate the impact on the overlay performance. The effective modulus of the existing slab ( $E_{BASE/DESIGN}$ ) is calculated as follows:

$$E_{BASE/DESIGN} = C_{BD} * E_{TEST}$$

Where:  $E_{BASE/DESIGN}$  = Design modulus of elasticity of existing slab, lbf/in<sup>2</sup>

$C_{BD}$  = Coefficient reduction factor

0.42 to 0.75 existing pavement in “Good” condition

0.22 to 0.42 existing pavement in “Moderate” condition

0.042 to 0.22 existing pavement in “Severe” condition

$E_{TEST}$  = Modulus of the existing uncracked concrete slab, lbf/in<sup>2</sup>.

The modulus of the existing concrete slab (sound material) was estimated by testing of cores to be 5.6 million lbf/in<sup>2</sup> (38,611 MPa)(ASTM C469). The  $E_{BASE/DESIGN}$  for each condition that is input to the MEPDG is computed in Table 5. In addition, the alternative of rubblizing the existing JPCP was evaluated.

**Table 5**  
**Existing JPCP Condition and Estimated Effective Modulus of Elasticity**

Condition	% Slabs Transverse Cracked	Effective Modulus of Existing Slab (lbf/in <sup>2</sup> )
		$E_{BASE/DESIGN}$
Good	10 %, 0.5 = $C_{BD}$	2,800,000 (2 to 5 million)*
Moderate	20 %, 0.3 = $C_{BD}$	1,680,000 (1.0 to 1.6 million)*
Severe	50 %, 0.1 = $C_{BD}$	580,000 (160,000 to 935,000)*
Rubblized	NA	50,000

\*Range of existing slab moduli obtained from the national calibration effort.

- Separation layer: A 1-in. (25-mm) HMA interlayer is placed to separate the existing and new concrete layers. The new HMA layer will have friction with the new concrete overlay to provide for proper joint formation and to minimize erosion.
- Overlay design: Thickness will vary from 8 in. to 9 in. (203 mm to 229 mm) with a constant joint spacing of 15 ft (4.6 m), slab width of 12 ft (3.7 m), and doweled joint load transfer. Traffic is too high to not include dowels, as high faulting is predicted.
- Site conditions: Aurora, Illinois, climate, A-6 subgrade soil, and I-88 traffic (two-way truck traffic is 10,000 with a 3 percent linear growth rate results in 55 million trucks (93 million ESALs) in the design lane over 30 years). Slab placed in June, opening to traffic in 1 month after placement.

The MEPDG was run for these specific conditions over a 30-year analysis period. The predicted performance at the end of the 30-year analysis period is given in Table 6. Additional MEPDG solutions were run for other conditions, including wider slab and increased HMA separation layer thickness, as shown in Table 7.

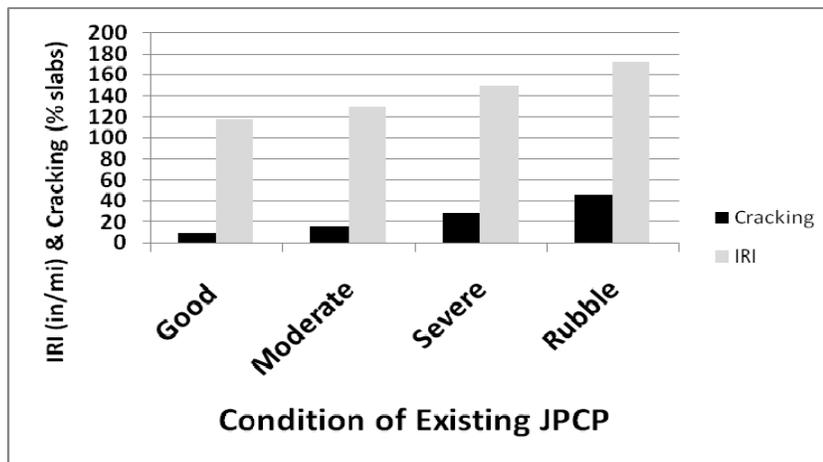
**Table 6**  
**Impact of JPCP Overlay Design Features and Existing JPCP Condition on Performance of Concrete Overlay Over 30 Years and 12-ft-wide Lanes (Case Study B) From MEPDG**

<b>Existing JPCP Condition</b>	<b>PCC Slab 8-in. Doweled Joint</b>	<b>PCC Slab 9 in. Doweled Joint</b>
“Good” Condition (10% slab transverse cracking)	C = 10% slabs F = 0.04-in IRI = 118-in/mi	C = 1 F = 0.05 IRI = 116
“Moderate” Condition (10-50% slab transverse cracking)	C = 16 F = 0.05 IRI = 129	C = 2 F = 0.06 IRI = 122
“Severe” Condition (>50% slab transverse cracking)	C = 29 F = 0.07 IRI = 150	C = 5 F = 0.08 IRI = 134

**Table 7**  
**Impact of Special JPCP Overlay Design Features and Existing JPCP Condition on Performance of Concrete Overlay Over 30 Years (Case Study B) From MEPDG**

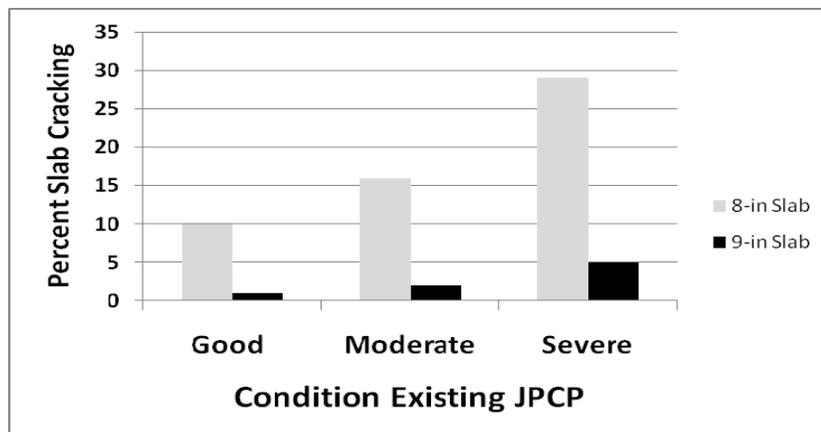
<b>Existing JPCP Condition</b>	<b>PCC Slab 8-in. Doweled Joint</b>
Special Case: Rubblize Existing JPCP	C = 46 F = 0.09 IRI = 172
Special Case: Severe Existing JPCP Condition with 13-ft slab width outer lane	C = 2 F = 0.001 IRI = 90
Special Case: Severe Existing JPCP Condition with Increased HMA separation layer from 1 to 3-in.	C = 23 F = 0.07 IRI = 145

- As existing JPCP condition varies from “Good” to “Moderate” to “Severe,” the performance of the JPCP overlay is significantly reduced, as shown in Figure 8. The JPCP overlay over the existing JPCP in “Severe” condition shows more cracking over time than the existing JPCP in “Good” condition. This is due to the composite slab effect when the existing JPCP is combined with that of the JPCP overlay to form a composite modulus.



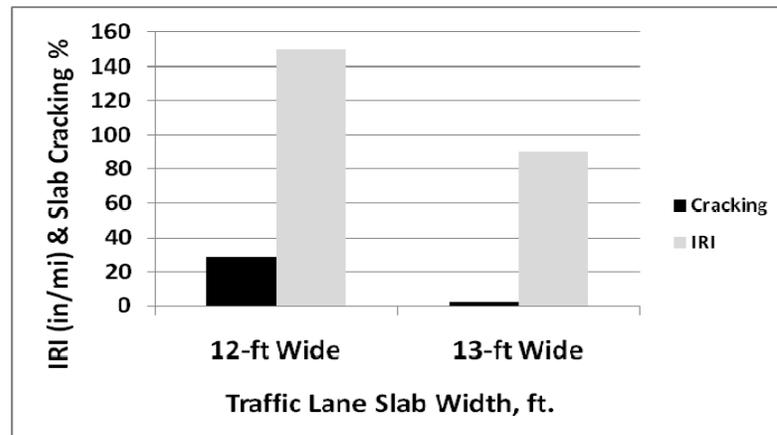
**Figure 8. Effect of condition of existing JPCP on IRI and transverse cracking of JPCP overlay after 30 years and 55 million trucks (93 million ESALs).**

- The thickness of the JPCP overlay has a significant effect on performance. The 8-in. (203-mm) JPCP overlay shows much greater cracking and IRI than the 9.0-in. (229-mm) JPCP overlay after 30 years and 55 million trucks. Figure 9 shows the trends.



**Figure 9. Effects of condition of existing JPCP and slab thickness on percentage of slab cracking after 30 years and 55 million trucks.**

- The thickness of the HMA interlayer was varied from 1 in. (25 mm) to 3 in. (76 mm) for a “Severe” pavement condition to see if it had a significant effect on performance. The prediction shown in Table 7 indicates that percent slab cracking decreases only moderately from 29 to 23 percent and IRI decreases from 150 to 145 in/mi (2.4 to 2.3 m/km).
- The 13-ft (4.0-m) traffic lane slab has a significant effect on reduction of slab cracking and IRI as compared to the 12-ft (3.7-m) conventional lane, as shown in Table 7 and Figure 10. There is also a very significant difference in joint faulting, reducing the mean faulting from 0.07 in. (1.8 mm) to zero.



**Figure 10. Effect of traffic lane slab width on slab cracking and IRI after 30 years and 55 million trucks.**

- Joint faulting is also affected by the condition of the existing pavement. Figure 11 shows the results for joint faulting after 30 years and 55 million trucks for the different levels of pavement condition including rubblizing the existing JPCP. As the effective modulus of the existing pavement decreases, the amount of joint faulting increases.



**Figure 11. Effect of existing JPCP condition on transverse mean joint faulting after 30 years and 55 million trucks.**

## **7.0 IMPACT OF EXISTING CONDITION ON JPCP OVERLAY THICKNESS**

The previous analyses focused on showing the impact of the condition of the existing HMA or JPCP on cracking, faulting, and IRI of the concrete overlay over time. This section briefly shows the impact on thickness of the overlay when the condition of the existing pavement is varied from good to very poor or severe. Table 8 shows the summary of results achieved from this analysis using the MEPDG and typical design criteria, including 0.12-in. (3.0 mm) joint faulting, 15 percent transverse cracking, and 170 in/mi (2.7 m/km) IRI all at 90 percent design reliability.

Case Study A JPCP/HMA results show the following:

- Slab thickness required increases from 7 in. (178 mm) to 8 in. (203 mm) as HMA pavement condition changes from “Fair” to “Very Poor.”
- Dowel bar diameter required increases from 1.00 in. (25 mm) to 1.25 in. (32 mm) as HMA pavement condition changes from “Fair” to “Very Poor.”

Case Study B JPCP/HMA/JPCP results show the following:

- Slab thickness required increases from 9 in. (229 mm) to 10 in. (254 mm) as the existing JPCP condition changes from “Good” to “Severe.”
- Dowel bar diameter required increases from 1.25 in. (32 mm) to 1.50 in. (38 mm) as JPCP condition changes from “Good” to “Severe.”

**Table 8**  
**Effect of Existing Pavement Condition on JPCP Overlay Design (MEPDG: Designs Meet Specified Distress, IRI, and Reliability Criteria Over 30 Years With Full Friction Assumed)**

Condition of Existing Pavement	Case Study A: HMA Existing Pavement (30 years, 8 million trucks)	Case Study B: JPCP Existing Pavement (30 years, 55 million trucks)
Fair (HMA) or Good (JPCP)	7.0-in., 1.00-in. dowels, 12-ft joint space	9-in., 1.25-in. dowels, 15-ft joint space
Poor (HMA) or Moderate (JPCP)	7.5-in., 1.25-in. dowels, 12-ft joint space	9.5-in., 1.25-in. dowels, 15-ft joint space
Very Poor (HMA) or Severe (JPCP)	8-in., 1.25-in. dowels, 12-ft joint space	10-in., 1.50-in. dowels, 15-ft joint space

## 8.0 FINDINGS AND RECOMMENDATIONS FOR CONSIDERATION OF EXISTING PAVEMENT IN CONCRETE OVERLAY DESIGN

The following findings and recommendations are based on the previous analyses of JPCP overlays over existing HMA and JPCP pavements using the MEPDG to predict performance for two case studies.

- The amount of deterioration of the existing HMA and JPCP pavements affects the performance of the JPCP overlay. This is caused by the loss of structural capacity of the existing pavement (concrete’s modulus of elasticity and HMA dynamic modulus) on the resulting composite structure of overlay and existing pavement.
- The increased slab thickness required for both existing HMA and JPCP case studies was 1 in. (25 mm) when the existing pavement ranged from “Good” to “Poor” condition. The increased dowel bar diameter required for both case studies was 0.25 in. (6 mm) over the same condition range.
- Thus, the condition of the existing pavement must be quantified and used in overlay design. The MEPDG provides practical definitions of the condition of the existing pave-

ment in terms of percent area of alligator cracking for HMA pavement and percent slabs transverse cracked for JPCP.

- The existing pavement can be repaired through full-depth repair or slab replacement. The amount of repair would affect the percentage of alligator cracked area for HMA and the percentage of transverse cracked slabs for JPCP. The required thickness of the JPCP overlay would decrease with increased repair of the existing pavement. The MEPDG provides the data to conduct a cost comparison between more repair and a thinner JPCP overlay with larger dowel bars to establish an optimum balance.
- The thickness of the existing HMA layer beneath the concrete overlay contributes to the performance of the concrete overlay. The thicker this HMA layer the lower cracking and IRI of the overlay. This will affect the thickness design.
- The HMA separation layer thickness change (from 1 in. [25 mm] to 3 in. [76 mm]) has only a moderate effect in mitigating the impact of cracking of the existing JPCP. The impact on overlay slab thickness is small.
- Rubblizing the existing concrete pavement results in a much reduced elastic modulus of the pavement. This reduction will require at least an additional 2-in. (51-mm) concrete overlay thickness over that required for the “Good” condition as illustrated in Case Study B.

The following results were obtained based on MEPDG predictions related to the effect of design features of the JPCP overlay.

- As JPCP overlay slab thickness increases, the impact of the condition of the existing pavement decreases for both HMA and JPCP existing pavement.
- The transverse joint spacing of the JPCP overlay has a major effect on the performance of the overlay. The shorter the joint spacing the lower faulting and cracking. However, jointing costs increase and this must be balanced with thickness requirements. Typically, for slabs from 6 in. (152 mm) to 7 in. (178 mm) thick, the joint spacing should be about 12 ft (3.7 m). For slabs 8 in. (203 mm) or thicker, the joint spacing should be about 15 ft (4.6 m).
- The widening of the slab by 1 ft (0.3 m) has a very beneficial effect on reducing joint faulting and transverse cracking. Only 1 ft (0.3 m) is needed; thus a slab 12 ft (3.7 m) wide would be built 13 ft (4.0 m) wide with the paint strip placed at 12 ft (3.7 m). This feature will also likely require a thinner overlay slab in the order of 1 in. (25 mm).
- Dowel bars in the transverse joints are not always needed and depend in particular on the climate and the level of traffic. Cold climates and high truck traffic loadings were demonstrated for Case Study A and Case Study B overlays. Both required dowels to control joint faulting. Lower traffic and warmer climates may not require dowels. The need for dowels can be determined using the MEPDG for specific projects.

These results shown in this paper are only as good as the MEPDG models’ predictions. How accurately can the MEPDG predict the performance of JPCP overlays? The national calibration

for slab cracking and joint faulting provided in Figures 1 and 2 indicate reasonably good predictions. The sensitivity results shown in this paper appear to agree with the general trends of the performance of JPCP overlays.

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# Concrete Pavement Patching—Simpler Can Be Better

Jim Grove,<sup>1</sup> Jim Cable<sup>2</sup>, and Peter Taylor<sup>3</sup>

## INTRODUCTION

Repair of concrete pavements includes both partial- and full-depth patching of distressed areas of the slab. Full-depth patching is a small subset of concrete paving and has a number of unique features. This is the focus of this paper. Unfortunately, many agencies approach patch design and construction from the mindset of conventional concrete paving, resulting in a number of unique properties of a patch repair being overlooked. Too often, patches do not perform well, and often the cause of failure can be traced back to procedures that are not appropriate for the patching operation.

In this paper, several of those paving paradigms are discussed with reasons why they may be working against the long-term durability of the patch. Also, those aspects that are unique to patching are discussed and despite the best of intentions, may be the reason why patches are not performing as intended.

This is a practical, application paper based on over 50 years of combined experience by the authors in the area of concrete pavement construction. Research is cited to support the ideas presented. The goals of the recommendations are to provide a highway agency with a method for patching concrete pavement that reduces the time required, allows lower opening strength, reduces cost, and results in a patch with a long life.

## WHY DO WE PATCH?

There are many reasons that pavement damage can occur. A pavement is designed and built with all the components—subgrade, subbase, base, and pavement—being optimized to perform together to provide adequate serviceability throughout the design life. Unfortunately we do not live in a perfect world. Most concrete pavements continue to provide service to the driving public long after they have reached the end of their design life. However, sometimes there are small sections that do not perform as well as the rest. When a pavement section starts to crack, pump, or become unstable, it is time to repair this area with a concrete patch.

Most patching repair falls into two general categories of causes. One is loss of base support, and the other is significant cracking of the pavement due to heavy loads or materials and construction issues. (Spalling can necessitate partial-depth repair but the focus of this paper is full-depth repair.)

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## **Loss of Subbase Support**

Loss of subgrade support can occur from a loss of base, subbase, or subgrade material under the pavement, and the pavement eventually cracks under traffic loading. Such of support is normally related to moisture that has weakened the underlying material, which is then lost through pumping. Water can collect under a pavement by a number of means: it can go down through joints, commonly when joint sealant fails, or it can come up under the pavement when the ground is saturated. This happens when a “bathtub” design has been used, and water has been trapped by the shoulder material.

## **Heavy Loads**

Cracking of the pavement can occur due to loading. In rural areas, heavy agricultural loads are carried on pavements in farm wagons with few axles. A single occurrence can crack the pavement. Heavy or repeated loadings can also cause failure if base support is insufficient or if loading occurs near the edges of slabs where it cannot be transferred to a neighboring element.

## **Construction Issues**

Damage to concrete pavements may be due to non-uniformity of the paving material at the time of construction. If a single truckload of concrete was batched incorrectly, or water was inappropriately added to a load, then that relatively small area of the pavement affected may fail early, while the remainder of the pavement is satisfactory.

Another common problem is cracking due to non-uniformity of base material. This can occur when localized base material is not properly consolidated, when a trench across the roadway is incorrectly backfilled before paving, or when there is a localized area of saturated base material.

Poor management of concrete volume changes due to temperature variations can also cause cracking: high temperature gradients due to combination of ambient temperature and heat due to cement hydration, cold front, late sawing, or built-in curl.

Also failures may be due to non-uniform curing. The curing compound may have been applied to a given area poorly, very late, or not at all.

## **HOW LONG MUST THE PATCH LAST?**

State highway departments are constantly looking for ways to reduce the traffic delay caused by patching operations, reduce construction costs, and improve the performance of patches. One of the first questions the designers should ask themselves is, “What is the purpose of this patch and how long must it provide satisfactory performance?” Too often that question is never asked, and the designers assume the patch must “outlast” the existing pavement. They think in terms of 20, 30, or more years as they would for a new pavement.

Patches are placed in a pavement to replace a small area that is not performing to the level of the pavement surrounding it. We place the patch rather than replace the entire pavement because we believe there is remaining life in the surrounding pavement. In other words, the patch needs to provide the same service life as the remaining service life of the original

pavement, however the agency defines pavement life. The AASHTO pavement design guides (AASHTO 2008) provide multiple ways to determine the remaining life in the surrounding pavement from knowledge of the original pavement design parameters, actual traffic loadings, and nondestructive testing etc. The patch should only be designed in terms of materials and construction methods to extend the life of this spot in the pavement to equal that of the pavement surrounding it. Designing for longer patch life is a waste of public funds.

Alternatively, the patch can be designed for a short period of time (1–5 years) to allow the highway agency to complete the design of a major reconstruction or rehabilitation of the entire pavement. This is the art of extending the life of the pavement until the entire pavement can be replaced.

### **WHAT MAKES A LONG-LIFE PATCH?**

A number of factors contribute to the success of a full-depth patch. The opening strength of the concrete should be sufficient to carry the immediate loading when traffic is applied. Over time, strength should increase to resist cyclic loading. This is discussed further in the following section. In general, a lower opening strength limit can generally be used for full-depth concrete patches and still achieve long-life performance.

Another material property of the mixture is the modulus of elasticity. It is critical that this be similar to, or less than, that of the existing pavement, otherwise when loads are applied, the new patch may act as a stress concentration point, leading to further damage in the existing pavement near the patch. Likewise, the coefficient of thermal expansion of the patch mixture should be similar to, or less than, that of the existing concrete to prevent stress being set up because of differential movements in the patch with respect to the remaining pavement.

The thickness of a patch should be sufficient to carry the loads applied at the time of opening, and as discussed below, there is benefit to be gained from using a thicker patch than the original pavement thickness.

### **WHAT OPENING STRENGTH IS NEEDED?**

Most States have minimum opening strength limits for concrete pavements. These empirically established limits have served the public well, but they may be more conservative than necessary for concrete pavement maintenance and repair applications like full-depth patches.

First of all, many minimum opening strength limits were established during a time when the only way to measure pavement strength was to cast and test a sample specimen. But specimen strength is only an estimate of pavement strength: The specimen does not have the mass of the pavement, so even when it is placed on top of the patch, under blankets, it does not gain strength in the same way as the patch concrete. The time required to remove specimens from under the blanket, strip the forms, deliver specimens to the laboratory, and test them also contributes to the difference between the tested strength of the specimen and the actual strength of the pavement concrete. To account for the various factors that contribute to differences between specimen strength and in-place concrete strength, a large factor of safety was built in to meet minimum opening strength limits. Now, however, in-place testing methods, like maturity testing, can more closely estimate concrete's in-place strength. A large factor of safety is no longer necessary.

Second, a pavement's minimum opening strength is generally based on the ultimate strength needed to carry expected traffic and loadings over the pavement's lifetime. By the time a full-depth patch is being considered, a portion of the traffic volumes and loads used in the original pavement design has passed over the road. Therefore, as described earlier, less strength is required of the patch to provide a service life corresponding to the remaining life of the pavement.

Finally, a full-depth patch need not be the same thickness as the original pavement. If a thicker patch is constructed, it will require less strength at opening.

Research by Okamoto et al. (1994) and Davis and Darter (1989) determined that the necessary strength needed for opening concrete pavement to traffic is significantly lower than the value commonly used by most State agencies. Field research by Cable et al. (2004) confirmed that good performance can be realized even when traffic is allowed on a patch before it has achieved commonly accepted opening strength values. ACPA Technical Bulletin TB002-02P also offers opening-to-traffic minimum strengths for patches much lower than conventional paving. More detailed discussion of these publications is provided below.

Each State must choose a strength value that balances the need for early opening with the need for a durable patch. Okamoto's research indicates that the limit may not need to be much above final set. Intuitively that seems low, but that is because our point of reference is traditional specifications based on new construction. These low values are such that under certain conditions, they may not always be achieved and tire marks in the patch are observed. When this happens, the sky is not falling! Many examples in Iowa, Georgia, and other States can be found where sections of pavement with tire marks in the concrete have performed as well as the surrounding pavement. An example is a section of a city street that runs in front of one of the authors' home. It is over 25 years old, and tracks from a car driving on the fresh concrete are visible today. There is no sign of damage caused by this early loading.

Construction sequence is another factor that should be considered when determining the necessary opening strength for a full-depth concrete patch. Normally the length of roadway section that will be closed for a patching operation is based on the time needed to complete the work within a day's working period. Often the operation proceeds from one end to the other. On larger patching projects, there may be a significant number of patches that can be completed in that one closure.

The critical patch is the last one to be placed. All others have more time to cure and therefore more time to gain strength than the last one. In most cases it would be impractical to have different opening criteria for various patches within the day's work zone; therefore all the patches except the last will automatically have an additional factor of safety.

In addition, the moment that opening strength has been achieved, there is a time lag before the first vehicle passes over it. Curing protection and then traffic control have to be removed. All this takes time, and a number of hours can easily pass before live traffic starts passing over this section of the roadway. Curing continues during that lag time and the concrete will be stronger than it was when the opening strength determination was made.

The time of the year, that is, the weather conditions, the concrete mixture, and curing methods, will determine the time needed to reach opening strength. If a 6-to-8-hour window is needed for

curing and time to remove traffic control, then patching on a two-lane roadway, where the road must be open before dark, will dictate that the last patch must be placed by shortly after midday. Admittedly this could shorten the work day somewhat, but if the proposed procedure is quicker than the traditional approach, the number of patches placed per day may not be reduced.

### **HIGH EARLY STRENGTH—ARE WE SHOOTING OURSELVES IN THE FOOT?**

Sometimes agencies focus on accelerating the development of sufficient concrete strength for opening by using calcium chloride, heated water, and/or Type III cement in the mixture. One characteristic of concrete, however, is that the faster it gains strength, the sooner strength gain will slow or stop. In other words, accelerating early strength gain can sacrifice long-term strength.

Calcium chloride is an inexpensive admixture used to accelerate early strength gain of concrete. Its effect is to accelerate early hydration, leading to elevated strengths in the first few hours and days. The side effect is to reduce later age hydration, meaning that there is little to no strength development after the first few weeks. This has the effect of reducing the safety margin in that no additional strength is developed over time to help carry long-term cyclic loads. Therefore using a mixture that will not gain much additional strength poses more risk than a mixture that will, over time, achieve strengths in excess of the opening strength. In the few hours when the strength is lower, very few loadings occur compared to those of the life of the patch, and therefore long-term strength is important for long-term performance.

Mixtures containing calcium chloride will normally generate significant amounts of heat, leading to the concrete mixture's setting at relatively high temperatures. High stresses may be incurred when the mixture cools. When the blankets are removed when low ambient temperatures are present (such as in the fall), thermal shock can initiate cracking.

Calcium chloride will also significantly increase the risk of corrosion of any embedded steel, whether dowels or tie bars. Another side effect is that the risk of efflorescence and discoloration is increased.

Another technique often used to increase early strength gain is to add extra cement. Experience shows that adding extra cement may not provide extra strength, but it will increase drying shrinkage, heat, cracking risk (Buch et al. 2008), and the cost of the mixture.

Since we have determined that lower values for opening strength can provide adequate concrete patch strength at the time of opening, we can design for increased long-term patch strength rather than accelerated strength. There will always be exceptions: When patching must be performed during weather conditions that are cool or cold (in general below 60 °F [16 °C]), for example, methods to accelerate hydration may be necessary. The use of heated water or other means to raise the concrete temperature may need to be employed, although if at all possible, calcium chloride should still be avoided. It may be necessary to extend the curing period under these conditions. A time-to-opening specification may need to be modified when cold temperatures are encountered.

## **EXTRA THICKNESS—LONGER LIFE PATCH AT NO EXTRA COST?**

Extra concrete thickness provides several advantages. First, it results in lower stresses in the pavement. Thicker pavements are built on the Interstate system than on residential streets because they need to withstand heavier loads. Second, thicker full-depth concrete patches need less strength before opening to traffic.

Third, when considering the whole patching process, extra concrete thickness may be added for almost no extra cost. The real cost of patch repair is in the labor cost of the hand work required and the traffic control needed for the operation. The material cost is minimal in comparison. Most patching specifications require excavation of unsuitable material and replacement with compacted granular material. That granular material is delivered in a dump truck, but an individual patch only needs far less. Therefore the cost of the granular material is high because of the small amount that is used. The cost of the hand work to place the material and the cost to hand compact it is considerable, and hand work adds time to the whole operation. The alternative is to simply fill the hole with concrete. The ready-mix truck is already on site. The only added cost for extra concrete thickness is the concrete itself. When the cost of the conventional approach is weighed against simply using more concrete, the thicker patch offers the advantages listed above and may not incur any additional cost.

Fourth, the number of patches that can be placed in a day may be increased, since the backfill, placement, and compaction operations are eliminated.

One concern about replacing a pavement section with a thicker patch is the deeper pavement blocking the drainage. The climate conditions in each region of the country must be considered with this issue. When longitudinal subdrains are in place and working properly, they should be able to provide needed drainage, since the transverse slope away from the high point of the crown will still direct water to the edge of the pavement.

In some regions there may be a drainage issue when no subdrains are present. That is often the issue that initiated the failed pavement; therefore the problem has to be addressed whatever the method of repair used.

Some agencies may have a concern about possible heave from the use of a thicker patch. This is a phenomenon related to moisture. A small change in pavement thickness will not change those conditions. As discussed above, steps should be taken to address excess moisture if that is the cause of the original failure or if it is a concern because of the climatic and soil conditions. Additionally, the dowels that resist the downward movement from traffic loadings will also resist upward movement from any heaving forces.

## **WHAT DO THE RESEARCH AND TECHNICAL BULLETINS SAY?**

A number of research studies support the recommendations of this paper, including an ACPA technical publication that gives guidance on full depth patching.

- A Federal Highway Administration study by Okamoto et al. (1994) investigated how much pavement life is consumed by early loading. Edge loading was the critical location, but most traffic will not drive on the edge. Also, cones or other traffic devices can be used to keep traffic away from the edge until higher strengths are achieved. This research found

that for a 10-in. (254-mm) pavement, compressive strength of 1,041 lbf/in<sup>2</sup> (7.18 MPa) (note: this would be less than 200 lbf/in<sup>2</sup> [1.38 MPa] flexural strength, depending on the conversion utilized), under interior loading, the pavement would experience no fatigue damage over 10,000 cycles). The report states that “Interior loading produced virtually no fatigue damage for 10- and 12-in. slabs, even if loaded when the compressive strength was 260 psi.”

Okamoto goes on to state that the bearing stresses produced by the dowels are often the controlling factor. He concludes that “Larger diameter bars were very effective in reducing the magnitude of the dowel-bearing stress.” This is another design element that is normally not reconsidered for patching, with new-pavement dowel sizes normally being used. The cost to use larger dowels would be negligible with respect to the cost of the patch, but could significantly improve the early performance.

- In research conducted by Davis and Darter (1989), it was determined that the thicker the patch, the lower the opening strength needed to be. Table 1 lists the results from that research:

**Table 1**  
**Opening Strength Required for Varying Pavement Thickness**

Slab Thickness, in.	Minimum Slab Flexural Strength, lbf/in <sup>2</sup>
7	370
8	335
9	275
10	200

- Cable et al. (2004) conducted research to meet the following objectives:
  - Investigate the relationship between pavement patch thickness and allowable opening time for concrete pavements.
  - Investigate the relationship between patch thickness and early traffic load capacity of concrete pavements.
  - Investigate the use of the maturity method being suitable for determining concrete strength and pavement early opening to traffic timing.

The experiment consisted of various combinations of concrete mix design, pavement patch thicknesses, and times of opening to traffic. The concrete mixes utilized in the experiment included a conventional paving mix and a patching mix. The existing roadway on this project consisted of a two-lane pavement, 24 ft (7.3 m) wide and 9 in. (229 mm) thick, with doweled transverse joints. The 9-in. (229-mm) depth served as the default thickness. Additional patch depths of 11, 13, and 15 in. (279, 330, and 381 mm) were tested to determine the effect of patch thickness on performance. The thickness also represents the impact of removal of unstable base and replacing it with concrete rather than special backfill and extra compaction effort.

Traffic opening criteria consisted of allowing traffic to begin to drive over the patches at 3, 5, and 7 hours. An additional set of patches was designed for opening at 350 lbf/in<sup>2</sup> (2.41 MPa) flexural strength based on maturity measurements. The overall layout of the test patches resulted in 16 separate patches utilizing the patching mix and an additional 16 patches employing the paving mix.

The study produced the following results:

- Increased patch depth enhanced the concrete strength gain associated with the heat of hydration determined by maturity testing.
  - Performance measures indicated no differences due to concrete mix (standard mixes or those with calcium chloride), opening times for traffic, or concrete patch depths.
  - Maturity testing proved to be an accurate and consistent method of estimating concrete strength gain with regard to determining traffic opening strengths.
  - Time of patching was equal to or less than conventional methods, with the substitution of over-depth concrete patch material placement versus subgrade preparation and pavement replacement.
- Buch et al (2008) investigated high early strength mixtures for patching, particularly from a durability standpoint. They reported that high-range water reducers negatively affect the air-entraining system. They found that the use of both Type III cement and high-range water reducer together often negatively influenced the air-void system and to a greater degree when calcium chloride was used.
  - Narotam and Vu (1993), Iowa Department of Transportation, conducted a study in 1993. They reported that water reducers resulted in lower early strengths. They also recommended a 6-hour cure time minimum.
  - ACPA Concrete Paving Technology—Guidelines for Full-Depth Repair recognizes opening-to-traffic criteria can be either minimum strength or minimum time. Although they recommend a strength criteria for opening, in early opening applications (24 hours or less), they also provide a minimum strength requirement of 250 lbf/in<sup>2</sup> (1.72 MPa) for patches 10 in. (254 mm) thick or more. This strength value is repeated in a later ACPA publication (ACPA 2006) and referenced in the FHWA publication “Concrete Pavement Preservation Workshop” (FHWA 2008).

## **WHAT ARE STATES DOING?**

When constructing full-depth patches, one way to remove the focus from early strength gain to long-term performance is to remove the requirement for strength to be met before the pavement can be opened. At least two States have done that. Georgia and Iowa do not have a minimum strength specification for opening to traffic. Both simply use a minimum time—or age—requirement. This can be sufficient if the mixture design demonstrates that adequate strength can be achieved, weather conditions and concrete temperatures are conducive to achieving those strengths, and proper curing and protection are applied.

In the current environment of limited inspection staff by most agencies, the need to measure strength is a work item that is often not realistic for inspection personnel. A time specification is much easier to enforce, and if equal or better performance is achieved, then this is a reasonable approach.

For a two-lane roadway patching project, where the road is to be opened by dark, the contractor is not required to have night lighting or reflective clothing for workers. What is going to happen at dusk is that the road will be opened, despite the strength of the concrete. With a strength specification, the contractor and the inspection personnel are placed in a no-win position of having to violate at least one specification, the open-by-dark or the strength specification. In most cases, the patch does not fail, but the wrangling over the specification has just begun. Neither is desirable, and neither changes the performance of the patch.

## CONCLUSIONS AND RECOMMENDATIONS

As discussed in this paper, some concrete pavement full-depth patch repair practices that have been commonly a part of most State specifications may actually lead to reducing the life of the patch. The practices discussed in this paper have the potential to shorten time to open, decrease cost, and improve the performance of full-depth patches.

Numerous variables influence how a patching operation will be conducted. A two-lane or four-lane facility, length of time allowed for closure, and weather conditions all must be considered and will affect the specifications and limitations. Although the practices recommended below may not be applicable to every project, in general they can be applied to both two-lane and four-lane roadways, where a minimum of 6 to 8 hours of curing is acceptable and when conditions are warm enough to not impede rapid strength gain, at least 65 °F to 70 °F (18 °C to 21 °C) air temperature. One variation to these proposed practices can be to utilize them for most of a day's work and then for the last patches placed, accelerated techniques may be used.

In circumstances where strength measurement is required for opening, the maturity method should be utilized. Bear in mind that using a test specimen to estimate the strength of the pavement is no less an estimate. Breaking a core or beam merely determines the sample strength, not the pavement strength.

Below are recommendations to increase performance and reduce the time needed to complete a patch repair with little change in cost.

- *Use a conventional concrete mixture with no calcium chloride.*
- *Make the patches thicker and eliminate granular backfill.*  
Remove any unsuitable material and excavate to a depth of at least 12 in. (305 mm). Then, fill the hole with only concrete.
- *Use larger dowel bars.*
- *Use age as the only criteria for opening, along with certain curing and temperature requirements.*  
Conduct laboratory testing to establish opening-to-traffic criteria based on time only. Test the strength of the mixture, using a reduced strength requirement, at temperatures

similar to anticipated field conditions. Of the three research and technical bulletins cited, for a patch 10-in. (254 mm) thick, ACPA recommends 250 lbf/in<sup>2</sup> (1.72 MPa) flexural strength; Davis and Darter recommend 200 lbf/in<sup>2</sup> (1.38 MPa); and Okamoto's recommendation is less than 200 lbf/in<sup>2</sup> (1.38 MPa). The authors believe that if a 12-in. (305-mm) minimum thickness patch is specified, 200 lbf/in<sup>2</sup> (1.38 MPa) could be used as a minimum strength for determining the opening time requirement.

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# Forensic Evaluation of Cracking in Panels Adjacent to Panel Replacements on Interstate 5 in Washington State

Linda M. Pierce,<sup>1</sup> Jeff S. Uhlmeier,<sup>2</sup> Jim Weston,<sup>3</sup> and Keith W. Anderson<sup>4</sup>

## ABSTRACT

In 2003 the Washington State Department of Transportation (WSDOT) conducted pavement rehabilitation (dowel bar retrofit, diamond grinding and panel replacements) on a 42-year-old plain jointed concrete pavement that was 9 in. (230 mm) thick. Within 5 months of construction, maintenance forces had placed temporary patches at six locations along the project length; by spring of 2005 the number of distressed locations had increased to 35 and by June 2006, construction estimates to replace the deficient panels ranged from \$3.5 to \$ 7.6 million. This paper summarizes the forensic investigation that ensued due to the rapid failure of the concrete pavement on this project. Though there appears to be no single cause of the rapid increase in panel cracking, the investigation identified a number of possible contributors that include: panel demolition/excavation methods, dowel bar drilling operations, construction equipment operating on panels supported by weak base or subgrade materials and dowel bar misalignment.

## INTRODUCTION

In 2003 the Washington State Department of Transportation (WSDOT) conducted pavement rehabilitation (dowel bar retrofit, diamond grinding, and panel replacements) on a 42-year-old plain jointed concrete pavement that was 9 in. (230 mm) thick. Within 5 months of construction, maintenance forces had placed temporary patches at six locations along the project length; by spring of 2005 the number of distressed locations had increased to 35, and by June 2006, construction estimates to replace the deficient panels ranged from \$3.5 to \$ 7.6 million (based on the replacement of either 200 or 500 cracked panels, respectively).

WSDOT has a successful history in conducting dowel bar retrofit and panel replacements, this severe and rapid increase in panel cracking has not been seen on any other concrete pavement rehabilitation project. The distress on this project was of great concern due to the rate of distress, the large number of distressed panels, and the high cost of repair and replacement, such that a forensic investigation was initiated. The following summarizes the results of the forensic investigation (1).

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## BACKGROUND

The pavement on this section of I-5 was originally constructed in 1961 with 9 in. (230 mm) of portland cement concrete pavement over 7 in. (178 mm) of crushed stone base. Currently, this pavement section carries an average daily traffic of approximately 64,000 vehicles and 9 percent trucks or approximately 17 million equivalent single-axle loads per year in the design lane.

Rehabilitation of this roadway was required due to panel cracking and faulting of approximately 0.5 to 0.75 in. (13 to 19 mm). The rehabilitation strategy selected for this section of roadway included panel replacements, approximately 270 panels (both directions of a four-lane, 3.3 mi [5.3 km] project), dowel bar retrofit of the right lane (both directions), and diamond grinding of all lanes. Construction began in April 2003 and was completed within 6 months. Panel replacements were completed during daylight hours of continuous 3- or 5-day lane closures.

## PANEL REPLACEMENT CONSTRUCTION

Standard panel replacement procedures that were conducted included perimeter sawcutting of joints around panel to be replaced, breaking of panel using a guillotine hammer, removing concrete with a backhoe equipped with a hoe point, drilling for dowel bar and tie bar placement, placing a bond breaker, and placing, finishing, and curing concrete (Figures 1 through 9).



**Figure 1. Perimeter sawcutting.**



**Figure 2. Breaking panels.**



**Figure 3. Removal with backhoe hoe point**



**Figure 4. Excavated panel.**



**Figure 5. Drilling holes for tie bars.**



**Figure 6. Drilling holes for dowel bars.**



**Figure 7. Placing concrete.**



**Figure 8. Use of vibratory screed.**



**Figure 9. Finishing concrete.**

For the most part, construction proceeded as planned with only the following noted challenges:

- During dowel bar drilling, spalling and cracking (Figure 10) between drilled holes occurred. The result of the cracking was a reduction in the concrete pavement structural section. In some instances, the contractor was instructed to delete dowel bars to avoid placement within the cracked or spalled area.
- During concrete placement, the contractor needed multiple reminders to correctly set the grade for aggregate base that was placed too high or too low.



**Figure 10. Cracking and spalling after dowel bar drilling.**

- Improper placement (too high or too low) of dowel bar cages.
- Placing concrete too high, causing the requirement for excessive grinding.
- Inconsistency of the concrete mix, resulting in placement problems.
- Contract special provisions required the placement of lightweight polyethylene sheeting or approved debonding agent along all existing concrete surfaces. During the first closure period, plastic sheeting was utilized, but for the remaining three closures, form oil (Figure 11) was applied as the debonding agent. Since there is a potential for the inconsistent application of form oil, WSDOT requires the use of plastic sheeting or roofing paper for ensuring that a bond does not take place.



**Figure 11. Use of form oil at transverse joint.**

## PAVEMENT PERFORMANCE

Within 2 weeks of project completion, severe panel cracking was observed in the concrete panels adjacent to the panel replacements. Within 5 months of project completion, WSDOT Maintenance forces were required to conduct temporary panel repairs at six locations of severely distressed panels (Figure 12). For the temporary fixes, maintenance forces chipped out the distressed concrete (sawcutting, inserting dowel or tie bars, and base preparation were not conducted) and backfilled using Five Star Hwy Patch. The total number of panels requiring temporary repairs increased to 16 by early 2007.

Since WSDOT Maintenance budget restrictions did not allow for extensive patching of concrete pavement, there was a concern that the number of distressed panels would continue to increase. The last temporary patch conducted by WSDOT Maintenance forces occurred in January 2007 and resulted in a total cost of \$4,700. At this time, the WSDOT Pavement Division was asked to conduct a forensic investigation into the causes and possible resolution of the accelerated panel cracking.



**Figure 12. Severely distressed panel.**

## DISTRESS INVESTIGATION

The first step of the forensic investigation was to conduct a field visit to quantify the amount and extent of the panel distress. The field visit identified a total of 38 panels that required replacement and 26 additional panels that contained some form of corner cracking or other distress that require repair. The field visit also identified (confirmed by WSDOT Maintenance forces) that the typical observed distress is transverse cracking, located 2 to 4 ft (0.6 to 1.2 m) from the transverse joint (Figure 13) on existing concrete panels adjacent to panel replacements.



**Figure 13. Typical panel distress.**

As part of the forensic investigation, the WSDOT Pavement Division conducted a thorough review of all project documents (plans, specifications, inspector reports, etc.). This investigation identified a number of possible contributors to the panel failures including the following:

- Dowels deleted during construction likely contributed to the smaller localized failures (Figure 14). Smaller localized failures are occurring within wheelpath locations where the pavement section was reduced as a result of spalling.



**Figure 14. Localized cracking and spalling.**

- Though it is difficult to prove that lane rental bidding incentives contributed to the panel cracking, the use of this incentive may have caused hesitancy on the part of the project inspectors to add additional work or change orders to the contract. For example, additional excavation and placement of backfill materials was needed in areas of poor base or subgrade materials, however, project plans did not clearly define these excavation areas. More than likely some panel replacements were placed over poor soil conditions, however, not all cracked slabs occurred in the areas of weak subgrade.
- The sawcutting operation did not extend to the full depth of the concrete slab (Figure 15), placing undue stresses within the concrete pavement during removal (impact breaking).



**Figure 15. Inadequate sawcut depth.**

WSDOT Standard Specifications (2) did not require full depth relief cuts (Figure 16) prior to panel demolition. Based on the experiences of northwest Washington concrete contractors, full-depth relief cuts should be made parallel to the perimeter of the panel replacement, 18 in. (457 mm) inside the longitudinal and transverse joints. Pavement removal, either by breaking concrete or lifting concrete sections, should first be restricted to the interior of the panel provided by the relief cuts. This provides a “buffer zone” that attenuates resulting forces used for pavement removal, so that damage to the adjacent panels, such as spalling, is avoided.

- Relief cuts were not used on this project, and in conjunction with insufficient sawcut depth, the force of the impact demolition using a guillotine hammer was transmitted to the existing panels and likely caused the panel cracking.
- During construction, concrete trucks backed off the existing concrete into the excavated area for the panel replacement. The subgrade may have been too weak to restrict excessive deflections at panel edges, which could have led to panel cracking.

Another possible cause of distress could be related to dowel bar misalignment. WSDOT’s current dowel bar alignment specifications (2), for an 18-in. (457-mm) dowel bar length are as follows:

- 0.5 in. (13 mm) from parallel to the centerline (horizontal alignment).
- 0.5 in. (13 mm) from parallel to the roadway surface (vertical alignment).
- 1 in. (25 mm) of the middle of the concrete slab depth (depth placement).
- 1 in. (25 mm) of being centered over the transverse joint (side shift).



**Figure 16. Typical relief cut layout used on other WSDOT projects.**

To evaluate the potential of dowel bar misalignment, arrangements were made with the Federal Highway Administration’s Concrete Pavement Technology Program to evaluate the dowel bar contained within the panel replacement using the magnetic imaging technology scan (MIT Scan). Of a total of 51 transverse joints between existing concrete and panel replacements scanned, 8 joints had less than the specified number of dowel bars. The dowel bar alignment results of the MIT scan are shown in Table 1.

At this time there does not appear to be a direct relationship between misaligned dowels bars and pavement distress; however, with additional traffic and climatic loading, a relationship may develop. One challenge is the uncertainty of how far out of alignment a dowel bar needs to be to initiate cracking; the other challenge would be whether or not the WSDOT specification is appropriate to minimize distress due to misaligned dowel bars. Currently, the National Cooperative Highway Research Program, Project 10-69, Guidelines for Dowel Alignment in Concrete Pavements, is evaluating dowel bar placement tolerances, which hopefully will provide guidelines for dowel bar alignment.

Finally, more to rule out the possibility, an investigation into the presence of alkali–silica reactivity (ASR) was conducted on this project. A total of nine cores were obtained within the project length and sent to Construction Technologies Laboratories (CTL) for a detailed analysis. CTL described the submitted core samples as fairly dense, hard, and strong, which is indicative of a concrete that was placed with a moderately low water–cementitious materials ratio. The CTL analysis concluded that there was no evidence of an ASR problem.

**Table 1  
Summary MIT Scan Results**

<b>Specification</b>	<b>Percent Exceeding Specification</b>
Horizontal (> ½ in.)	39.0
Vertical (> ½ in.)	16.0
Side shift (> 1 in.)	19.0
Depth (> 1 in.)	3.7

## **SUMMARY**

While many possible failure scenarios have been discussed, the failure mechanism on this project cannot be specifically attributed to a single factor; however, the accelerated panel cracking appears to be construction related. WSDOT has conducted panel replacements on over 250 lane-mi (402 lane-km) of concrete pavements; failure of any type, particularly panel cracking from panel replacements is rare.

The possible contributors to the panel cracking based on a thorough review of all project records include:

- Panel demolition/excavation.
- Drilling operations.
- Construction equipment operating on panels supported with weak base or subgrade materials.
- Dowel bar misalignment.

The cause of the panel cracking is most likely due to the use of a guillotine pavement breaker to break the panels to be removed. Since relief cuts were not used on this project, excessive forces were transmitted to adjacent panels during the breaking process. While cracking was not evident during construction, cracking became evident after continued loading from traffic.

## **PROJECT REHABILITATION RECOMMENDATIONS**

Since October 2003, WSDOT Maintenance has spent nearly \$70,000 to repair the growing number of cracked panels. By 2007, approximately 16 panels had been repaired. Initially, both WSDOT Maintenance and the Pavement Division estimated that these repairs would be temporary and last maybe a year, but the resulting performance of these repairs has been very good (Figure 17). With each repair, expertise has been gained, and it appears the repairs will last much longer, perhaps 5 years. WSDOT Maintenance estimated that within the next year, eight panels will need repair at a cost of \$40,000. Repairs to smaller areas (cracking over dowel, spalls, etc.) will increase this cost to \$50,000.

Due to restricted budgets, the repair scenario that makes the most sense, at this time, is to have WSDOT Maintenance continue to repair the cracked panels on an as-needed basis. Recently, there has been discussion that additional work will likely occur in this corridor, possibly within the next 5 years. It may be more economical to program additional panel replacement work on these future contracts. In 2006, construction estimates to correct the deficient panels throughout the project limits were estimated and included two options: (1) Replace severely distressed panels (estimated to be 210 panels) at a cost of \$3.5 million, or (2) replace all cracked panels (estimated to be 510 panels) at a cost of \$7.6 million.



**Figure 17. Performance of Maintenance repair.**

## **RECOMMENDATIONS**

This forensic investigation illustrated the importance of sound construction practices and the realization that construction procedures can ultimately affect pavement performance. Specific considerations for future panel replacement work include the following:

*Drilling Operations.* If excessive spalling occurs during dowel bar installation, the panel replacement limits should be extended to sound concrete.

*Panel Excavation Techniques.* If pavement breaking using a guillotine hammer is performed, project inspectors must ensure that full-depth perimeter sawcuts are provided. Additionally, full-depth relief cuts should be placed around all abutting concrete 12 to 18 in. (305 to 457 mm) inward of the full-depth perimeter sawcut edge. WSDOT inspectors need to enforce Section 5-01.3(4) of the Standard Specifications, which specifies that the removal process shall not damage adjacent slabs that will remain in place.

*Operation of Construction Equipment.* Panels supported by weak base or subgrade can essentially become cantilevers and allow excessive deflection in the existing panels. Under these conditions, backing excavation equipment or concrete trucks off existing panels to areas where concrete will be poured should be avoided. When poor soil conditions are present, restricting the use of construction equipment on the pavement or grade or, in some cases, performing work from the shoulders may be necessary.

*Dowel Bar Alignment.* While it is not conclusive that poor dowel bar alignment contributed to joint lockup and ultimately panel cracking, care should be given to place dowel bars according to WSDOT specifications.

*Panel Replacement Procedures.* Best practices should always be followed as specified in WSDOT Standard Specifications. Simple steps, such as sawcutting panels to full depth or using polyethylene sheeting around the perimeter of a pour-back area, need to be followed to provide panel replacements that will perform as intended.

*Concrete Rehabilitation Training.* The WSDOT Pavement Division, Construction Office, and Regional Training Offices, have been providing short courses for inspectors on dowel bar retrofit, concrete rehabilitation, and new concrete construction. These training sessions have been very well received and have provided a valuable forum for inspectors to ask questions and address concerns. These resources are readily available to project offices when concerns or problems arise with concrete pavement design or construction. A collaborative approach between the project office and these resources can lead to a reduction of errors during both the design phase and the construction phase of projects. More proactive coordination is needed between the various offices on the availability and benefit of these types of training courses.

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## **Part 2**

# **Concrete Pavement Preservation, Repair, and Rehabilitation**



# Restoration of New PCC Pavement With Uncontrolled Cracking in Missouri

John P. Donahue<sup>1</sup>

## INTRODUCTION

A new, properly designed jointed plain concrete pavement (JPCP) is intended to provide long-lasting performance with minimal maintenance and infrequent, if any, rehabilitation. The time required for quality control over construction techniques, mix materials, and environmental conditions inhabits a minute portion of the pavement's potential design life, yet renders so much influence over the probability of achieving that design life. Since every aspect of quality control cannot realistically be fully attained during construction, State specifications usually contain a safety net allowing the project engineer to reject the finished product if any defects become visually apparent.

One such defect that randomly occurs is uncontrolled cracking. Cracking has its root in various causes, including but not limited to late sawing, insufficient sawing depth, thermal shock, and high water-to-cementitious materials (w/c) ratios. Whatever the causes are, the result is the same, a panel with an indeterminate reduction in service life because it is no longer a monolith. Typical State specification language universally proclaims, "Remove and replace." Although the specifications often allow some room for alternative mitigation, the project engineer representing the State agency, not necessarily trained in the nuances of judging relatively harmless versus crippling concrete fractures, generally errs on the side of conservatism and requires the full-depth repair. Unfortunately, the replacement panel, besides being a costly fix, can never duplicate the aggregate interlock properties of the original slab.

For the past decade or so the American Concrete Pavement Association (ACPA)<sup>1</sup> and the Federal Highway Administration (FHWA)<sup>2</sup> have placed greater emphasis on using alternative and less severe repair options for cracked panels such as cross-stitching, dowel bar retrofit, under-sealing, and partial-depth repair. The Missouri Department of Transportation (MoDOT) has made a concerted effort in the past 5 years to employ these strategies. This paper presents the details of four new JPCP projects that had uncontrolled cracks form soon after construction, and the procedures taken to evaluate and correct the deficiencies.

## US-36, MACON COUNTY, MISSOURI

### Description

In 2003, a new dual-lane facility on US-36 was built approximately 6 mi (9.7 km) west of Macon in north-central Missouri. Incorporating the existing two-lane roadway into the new alignment required building new roadway in both the eastbound and westbound directions. The pavement was 12 in. (300 mm) of jointed plain concrete (JPC) with 15-ft (4.6-m) joint spacing

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on 4 in. (100 mm) of crushed stone base. The widened outside 14-ft (4.3-m) lane, 12-ft (3.7-m) inside lane, and full-depth, 4-ft (1.2-m) inside shoulder were paved together monolithically in one pass.

### Observed Distress

That summer, spalling and longitudinal cracking had developed in several panels. Spalling occurred at the centerline joint / transverse joint intersection in three panels. It was soon found during the repair of these spalls that the longitudinal tie bars were lying across the transverse dowel bars. Subsequent inquiry revealed that the contractor’s automated tie bar inserter on the concrete paver had not been working properly.

Later that year more problems were discovered. Pavement in the eastbound direction, which had been open to traffic for several months, was closed per the construction staging plan. A close inspection revealed what was difficult to ascertain under traffic: longitudinal cracking was developing parallel to the centerline joint. Crack appearance was random with some cracks occurring in single panels while others stretched across multiple panels. Most were on the median side of the joint in the eastbound lanes, but a few were in the westbound lanes, which were open to traffic

### Field Investigation

In the spalled area, a steel locator was used to determine the extent of misplaced or misaligned tie bars. The testing length extended as far as the tie bar inserter was thought to have malfunctioned. Based on the locator findings, 138 out of 668 panels failed the minimum tie-bar placement criteria (at least 3 in. [76 mm] across the joint).

The cracked panels in the eastbound direction were surveyed in August. A total of ten 4-in. cores were drilled at three separate locations to determine the extent of cracking. The cracked panels in the westbound lanes were not surveyed because of traffic.

The thickness and sawcut depths of four consecutive cores are summarized in Table 1. Pictures of the cores and a diagram of their locations are shown in Figure 1.

**Table 1**  
**Core Data from US-36**

<b>Core No.</b>	<b>Core Thickness (in. / mm)</b>	<b>Sawcut Depth (in. / mm)</b>
7	12 <sup>7</sup> / <sub>8</sub> / 327	3 <sup>1</sup> / <sub>2</sub> / 89
8	13 / 330	3 <sup>3</sup> / <sub>8</sub> / 86
9	13 <sup>1</sup> / <sub>8</sub> / 333	None
10	13 <sup>1</sup> / <sub>4</sub> / 337	3 <sup>1</sup> / <sub>2</sub> / 89

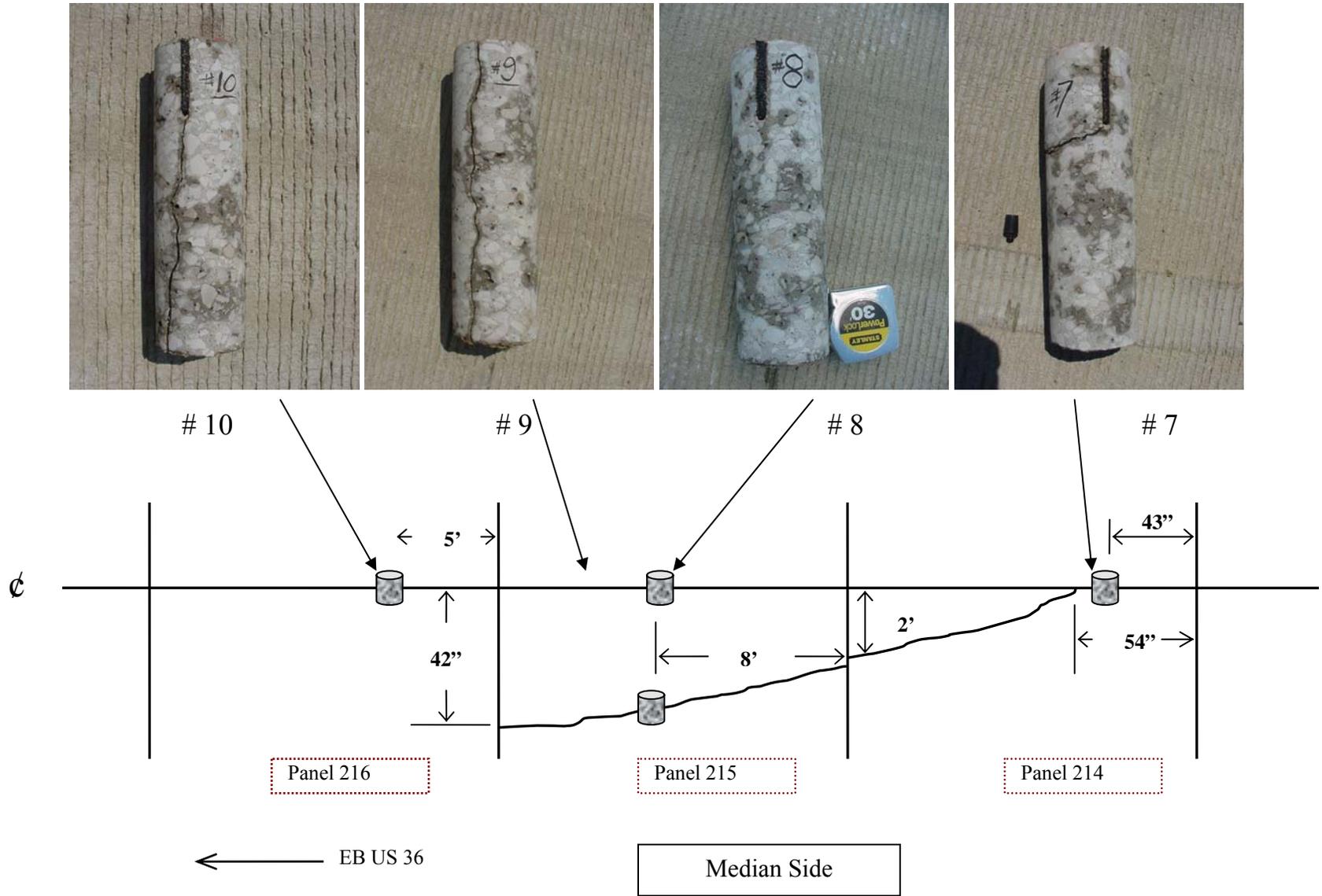


Figure 1. Core locations on US-36.

The trend observed in the crack and cores in Figure 1 typified what was seen in the others. The sawed centerline joint would crack through (Core No. 10) at a point parallel to the start of the uncontrolled crack, but farther along it would not crack (Core No. 8). Meanwhile, the uncontrolled crack would usually, but not always, veer into the sawed joint. The uncontrolled crack's influence would still be felt beyond this point, however; as evidenced by the shallow angular fracture planes under the sawed joint in Core No. 7, which were believed to be prone to spalling or breaking off.

Another finding was that 75 percent of the cracked panels (in the eastbound lanes) were within 1.5 ft (0.46 m) of the longitudinal joint on the median side. The median side of the sawed joint had the greater potential monolithic width at 16 ft (4.9 m) (combined inside shoulder and inside lane) versus 14 ft (4.3 m) on the widened outside lane, thus indicating a propensity for the uncontrolled crack to form near the center of the 30-ft-wide (9.1 m) monolith.

Finally, the ratio of the average sawed depth to field core thickness was 26 percent, very close to the T/4 (one-quarter the thickness) specification.

### **Distress Mitigation**

The 138 panels that failed the minimum tie bar placement criteria were subsequently cross-stitched across the sawed centerline joint by the contractor.

In the other area of the project, where uncontrolled cracks ran parallel to the centerline joint in both eastbound and westbound lanes, panels with continuous cracks were cross-stitched with No. 8 bars on 2-ft (0.61-m) centers. Panels with a confluence between a sawed joint and an uncontrolled crack were replaced because of concern over the shallow angular fractures eventually spalling.

As a result of the uncontrolled cracking on this project, the standard specification for longitudinal sawcut depth on all other projects was increased from T/4 to T/3.

## **ROUTE 94 IN ST. CHARLES COUNTY**

### **Description**

Among the many work items included in the reconstruction of the US-40 / Route 94 interchange in St. Charles County was the addition of a right-turn lane in the southbound direction on Route 94 at the Weldon Spring Road intersection during May 2005. The lane was constructed with a tied outside shoulder, and a concrete barrier was doweled to the shoulder edge. The 9-in. (230-mm) JPCP pavement rested on a fill area stabilized by a retaining wall (Figure 2). The old fill was left in place next to the new fill under the turn lane; however, the rest of the pavement was removed and a 3.5-ft (1.07-m) cold-millings layer was compacted across the entire roadbed prior to paving. The turn lane led to a residential and light industry zone.

## Observed Distress

Twenty-one of 22 panels in the turning lane developed some form of cracking. The cracking pattern is detailed in Figure 3: 15 panels had a single longitudinal crack running from transverse joint to transverse joint; 4 panels had multiple cracks emanating in a Y-shape; 2 panels had corner cracks. The crack widths remained fairly tight, yet some displayed obvious faulting, which became the primary concern of the field investigation. Project office personnel said that the crack initiated at the southern end and propagated northward along the entire length.



**Figure 2. Right turn lane with pinned concrete barrier on retaining wall.**

Load transfer testing was also conducted across three transverse joints and two cracks. The load transfer values were 87 percent or higher, but the values were judged artificially high because of the warm ambient temperatures during testing.

### ***Faulting***

Faulting was measured across the cracks in 1/16-in. (1.6-mm) increments. Three evenly distributed measurements were made in each full-length panel crack. Faulting in this case is the drop in elevation of the outside (barrier side) cracked slab panel with respect to the inside (traffic side) one. The average faulting of each panel is shown in Figure 4. The greatest faulting occurred at the southern end of the turn lane. The depth of faulting in a longitudinal crack on a new pavement, particularly one that had experienced very light traffic, strongly suggested settlement problems. A half-year after the initial field investigation, faulting increased by an average 1/16-in. (1.6 mm), indicating further settlement of the outside pavement.

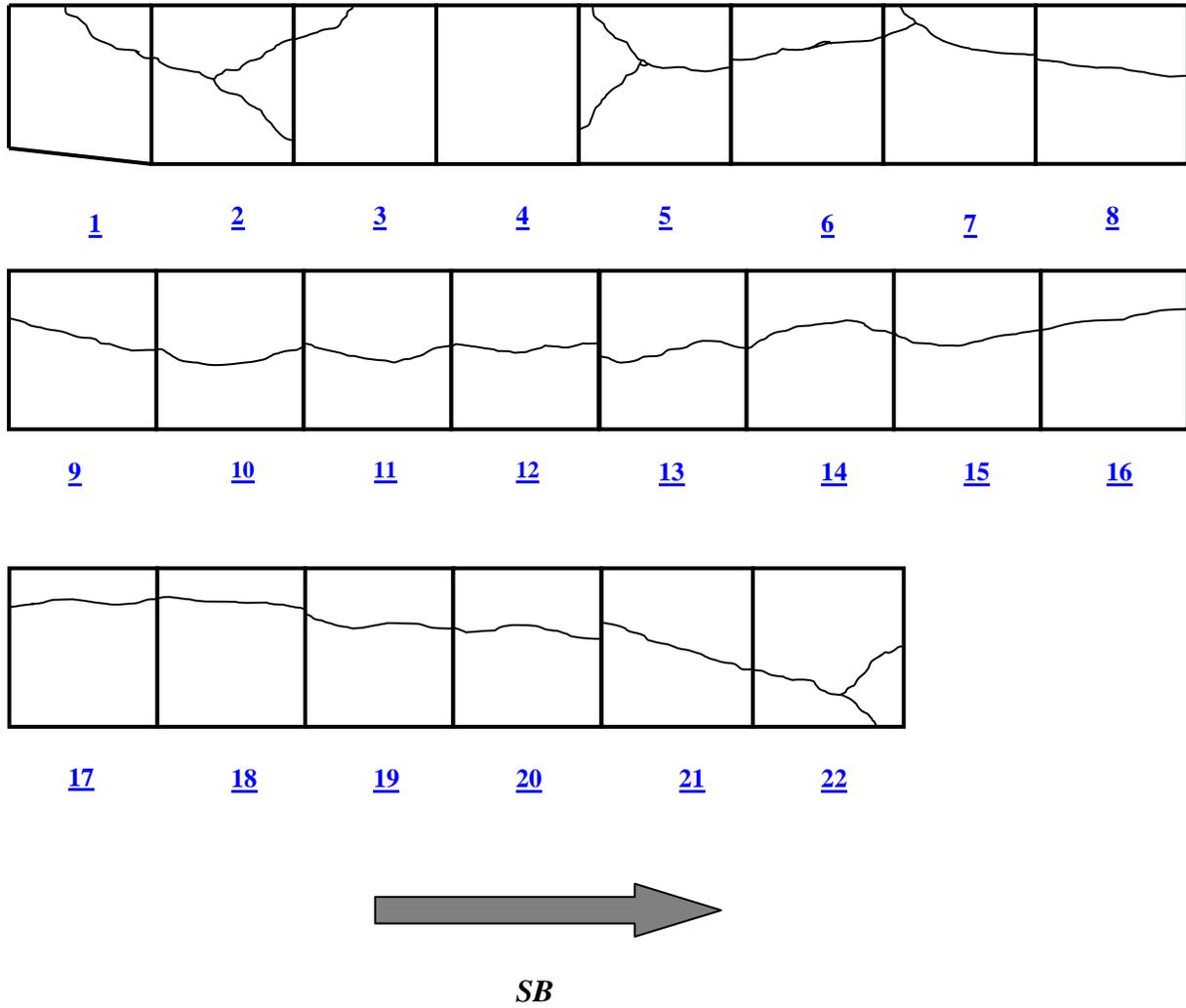
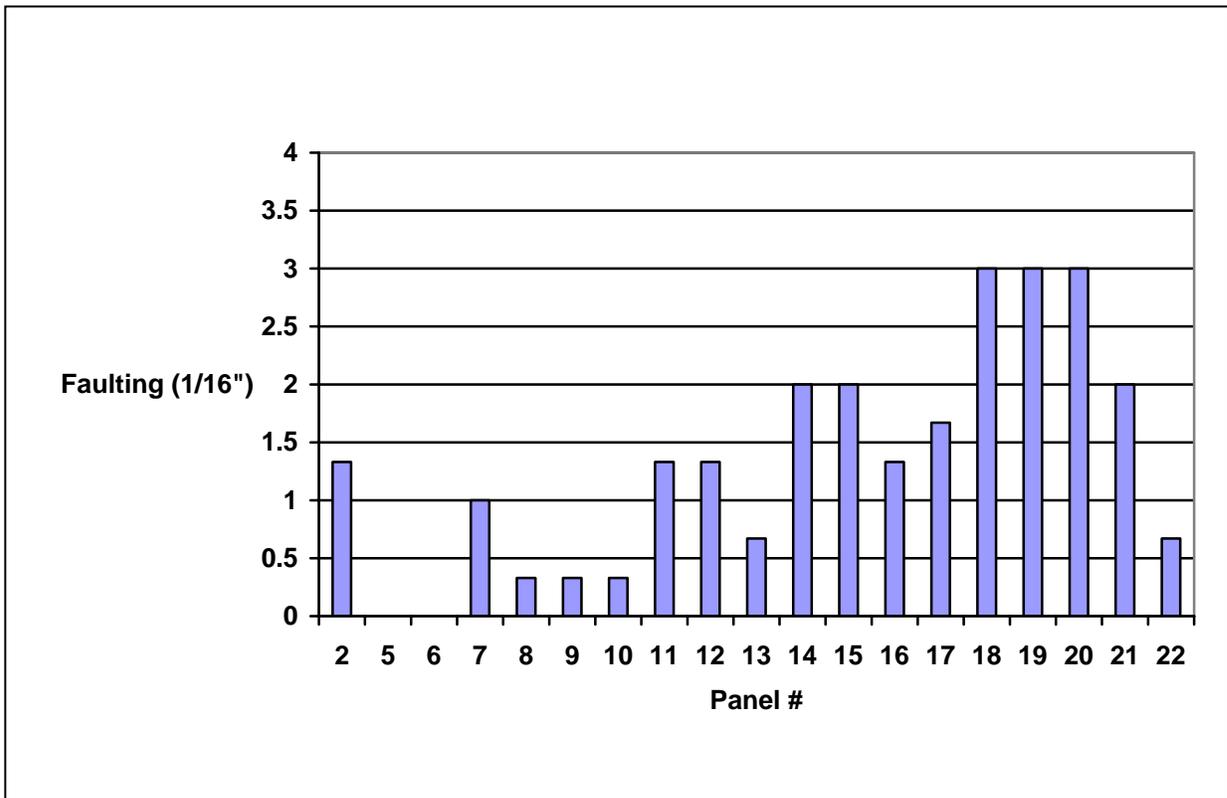


Figure 3. Route 94 turn-lane cracked panel diagram.



**Figure 4. Faulting across longitudinal cracks.**

#### *DCP testing*

DCP testing was performed after removal of slabs with multiple-fractures in December 2005. Four tests were situated in one panel area—one directly on and one directly below the 4-in. (100-mm) crushed stone base, and two below the 3.5-ft (1.07-m) cold-millings layer. Of the two below the cold millings, one was on undisturbed (by this project) subgrade below where the original outside shoulder had been and the other was on the newly compacted fill under the widened pavement. Neither the widened fill subgrade nor the original subgrade exhibited uniformly stiff layers, but on average the latter was stiffer and somewhat more consistent than the former.

Differential settlement in the widened fill was the only probable explanation for the slabs cracking and faulting. This problem was not unlike the settlement occurring at many bridge approaches where the high fill continues to consolidate and lose volume, while the adjacent structure on one side and the pavement on shallow fill on the other side have virtually no relative settlement. A thorough geotechnical investigation would have been required to substantiate this hypothesis and verify the exact nature and location of the settlement, but the low overall importance of this piece of pavement precluded that level of activity.

#### **Distress Mitigation**

Despite the apparent deep-seated cause of the cracking and longitudinal faulting, reconstruction of the fill and pavement was not considered a viable option. Rather, the panels had to be re-

paired in a way that corresponded to the significance of this particular lane. Since most of the longitudinal cracks appeared to maintain tight aggregate interlock, cross-stitching with No. 6 bars was employed on 17 of the panels. Only panels 2, 5, 7, and 22 were deemed damaged beyond conventional restoration techniques and were replaced. Three years later, the cross-stitched panels have not shown any visible increase in distress, and the roadway is functioning normally under traffic.

## US-412 IN DUNKLIN COUNTY

### Description

Multiple projects were constructed in the 2000s to complete the dual-lane expansion of US 412 from I-55 to Kennett in southeast Missouri. One such project was paved in the eastbound direction between Dunklin and Pemiscot counties during 2004 and 2005. A 12-in. (305-mm) JPCP was constructed on 4 in. (100 mm) of compacted granular base. The section of interest to this report was paved in three sections on November 5–7, 2004. Temperatures during paving ranged from the high seventies to low forties (°F). As with nearly all other concrete projects during Missouri’s era of full-depth shoulders, the widened outside 14-ft (4.3 m) lane, 12-ft (3.7 m) inside lane, and full-depth 4-ft (1.2-m) inside shoulder were paved together monolithically in one pass.

Soon after paving, early entry sawing was used to form the transverse and longitudinal joints. The joints were sealed on November 17. Temperatures during the day of sealing ranged from 50 °F to 70 °F (10 °C to 21 °C). The pavement section was not opened to public traffic prior to spring 2005, when MoDOT inspectors encountered cracks in the outside lanes emanating from the transverse joint within 1–2 ft (0.3–0.6 m) of the shoulder longitudinal joint. Some of the cracks spalled and began to heave the corners up to 0.1875 in. (4.8 mm) (Figure 5). The cracks were always accompanied by severe joint sealant extrusion (Figure 6). In three locations the corner crack spread across multiple panels (Figure 7).



**Figure 5. Corner cracking and spalling.**



**Figure 6. Joint sealant extrusion.**



**Figure 7. Crack across multiple panels.**

## **FIELD INVESTIGATION**

The cracking distresses were investigated in summer 2005. Testing consisted primarily of taking cores to examine cracking and joint saw dimensions. Joint sealant material was extracted from a couple of cores and tested in the Central Lab chemical section.

Cores were drilled out at various joint locations, but primarily at transverse joints near the outside shoulder longitudinal joints. Table 2 includes core data from one distressed section in particular to represent what visibly occurred in several other locations.

Coring revealed a pattern. Corner cracking and/or spalling occurred every five to seven slabs where full-depth cracks formed through the sawed joints, as at Joints 67, 72, and 77. In addition to the extreme surface extrusion, these joints also had moderate to severe sealant infiltration, in some cases through nearly the full thickness of the core (figures 8 and 9). Sealant-filled cracks were up to 0.2 in. (5 mm) wide. Interim transverse joints between the cracked joints had either no or light partial-depth cracking, therefore no sealant was able to penetrate into these joints.

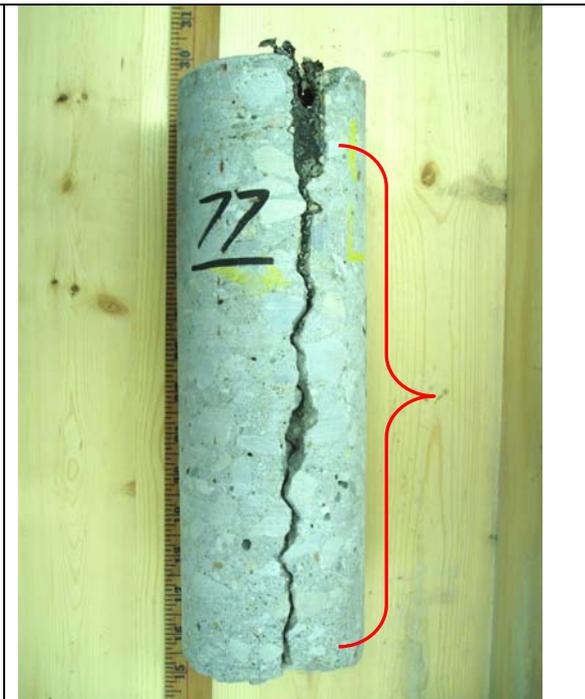
Minimum saw depth for early entry sawing is one-eighth the thickness, or in the case of a 12-in. (300 mm) pavement, 1.5 in. (38 mm). The saw depth was more than adequate at all cored locations, often exceeding 2 in. (50 mm); however, it was known from talking to project personnel that the time of sawing had substantially exceeded the 12-hour limit required for early entry methods. The total thickness ranged from 12 to 13 in. (300 to 325 mm).

**Table 2**  
**US-412 Core Data**

Joint No.	Lane	Core Thickness (in. / mm)	Saw Depth (in. / mm)	Joint Crack	
				Formation	Width (in. / mm)
67	Driving	12.50 / 318	2¼ / 57	Full depth	0.12 / 3
68	Driving	12.40 / 315	2¼ / 57	No	N/A
69	Driving	12.75 / 324	2¼ / 57	Partial	N/A
70	Driving	13.00 / 330	2⅛ / 54	Partial	N/A
71	Driving	13.00 / 330	2⅜ / 60	No	N/A
72	Driving	13.00 / 330	2¼ / 57	Full depth	0.04 / 1
73	Driving	12.75 / 324	2¼ / 57	No	N/A
75	Driving	12.50 / 318	2⅜ / 60	No	N/A
76	Shoulder	12.10 / 307	2⅛ / 54	No	N/A
76	Driving	12.25 / 311	2¼ / 57	No	N/A
77	Shoulder	12.75 / 324	2⅛ / 54	Full depth	0.16 / 4
77	Driving	12.25 / 311	2½ / 54	Full depth	0.16 / 4



**Figure 8. Joint 67.**



**Figure 9. Joint 77.**

Faulting was measured at every joint. Virtually no faulting existed other than the cracked corner locations that had spalled and heaved upward.

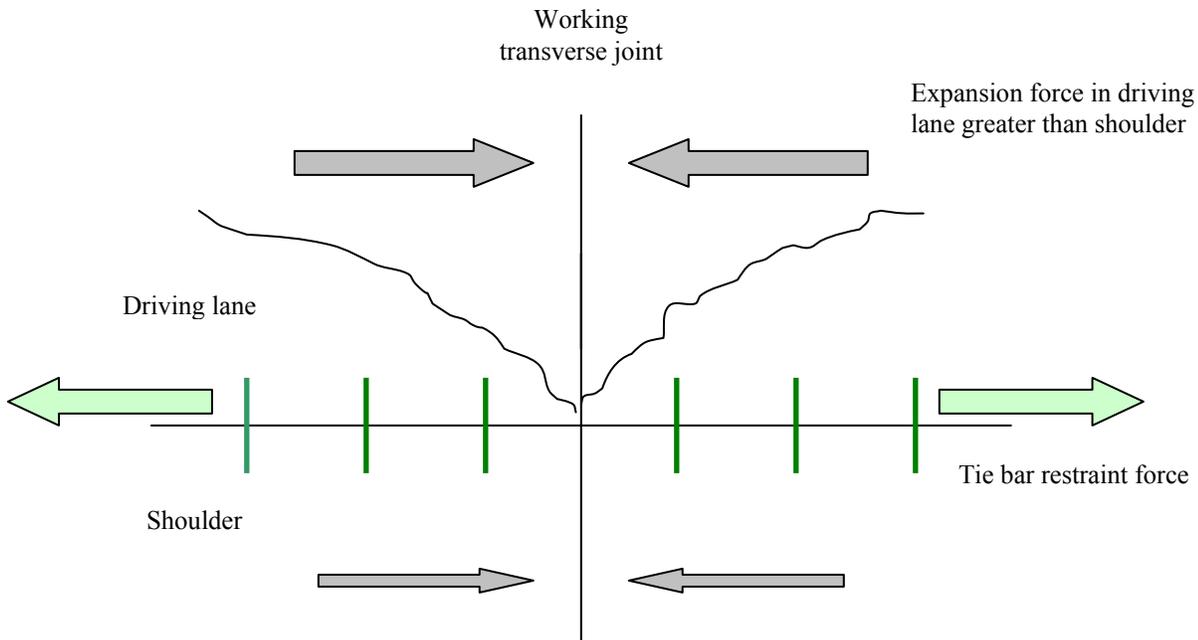
Over 600 g (21 oz) of the sealant, a hot-pour type typically used on concrete pavements, was extracted from a couple of joint cores for performance testing. Table 3 contains the minimum MoDOT requirements, which are based on the ASTM 6690 specification; the manufacturer’s specifications for its product; and the actual MoDOT chemical lab test results for the sample.

**Table 3**  
**Joint Sealant Test Results**

<b>Specification</b>	<b>Maximum Penetration (mm/10)</b>	<b>Maximum Flow (cm)</b>	<b>Minimum Resilience (%)</b>
ASTM 6690	90	3.0 (in 5 hr)	60
Manufacturer specs	80	0.1 (in 5 hr)	63
MoDOT lab results	138	> 3.0 (in 1 hr)	0

The results clearly show that the sealant failed to meet ASTM 6690 criteria by a wide margin and fell even farther below the manufacturer’s standards. This led to an early hypothesis that the uncontrolled pavement cracking and corner heaving could be attributed to sealant infiltration. The circumstances that would have allowed this to happen included the combination of several factors: a difference between ambient and concrete temperature during paving of 30 °F to 40 °F (1 °C to 4 °C), causing cracks to develop every five to seven slabs; late and insufficient saw depth at the other joints necessary to initiate cracking at interim panel joints; hot pour mixed with fine aggregate and cement slurry infiltrating the deep crack openings; and finally, hot summer temperatures the following year, causing expansion and subsequent shear failures.

However, since it is doubtful that the sealant / slurry mixture could have had enough viscosity to act as an incompressible, the more logical explanation, or at least the apparent major contributor, was tie bar restraint. The 8-ft (2.4 m) shoulder was paved at a later date during significantly colder temperatures than the adjacent monolith, therefore it experienced less thermal contraction in the winter months. In June, as temperatures approached 100° F (38° C), the effective “super slabs” in the driving lanes had much farther to expand at the working joints than did the adjacent outside shoulders. The difference in expansion forces activated a shear stress by the No. 6 tie bars, embedded 15 in. (381 mm) on either side, which in turn caused a shear failure near the corners (Figure 10). Had the shoulder expanded as much as the driving lane, the joint interface would have experienced uniformly high compressive stresses and the tie bars could not have been in shear. This theory was verified by thinner joint crack openings measured in the portion of driving lane that had been “freed from bondage” to the remaining section still attached to the shoulder.



**Figure 10. Joint restraint failure.**

### **Distress Mitigation**

A few panels had multiple fractures, and there remained little choice but to replace them. However, the majority of the panels were salvaged with the following combination of CPR treatments:

- Tie bars were severed (with a saw) along the length of two panels on either side of “super slab” working joints. This eliminated the tie bars’ ability to restrain the differential thermal movement of the driving lane and shoulder and propagate more cracking.
- Partial-depth repairs were performed at corner crack locations where the cracks had not grown more than a few feet.
- Cracks that extended across multiple panels were cross-stitched in the interim panels where the crack did not compromise corners with potential spalls.

A survey was conducted in 2008, more than 3 years after the repairs had been made, and no additional cracks or failures were observed. Cores taken during this survey verified that the interim joint locations had finally cracked through at the sawed joints, allowing the pavement to behave in discrete 15-ft (4.6 m) panels, as designed. The roadway is open to traffic and functioning normally.

Since the construction of this project MoDOT has eliminated tie bars between driving lanes and shoulders.

## MO-21 IN JEFFERSON COUNTY

### Description

MO-21 in Jefferson County, a notoriously dangerous, curvy, two-lane road, was realigned as a divided four-lane facility with vastly improved geometrics through a series of projects spanning the past 10 years. The two 8.5-in. (216 mm) JPCP driving lanes on 4-in. (100 mm) aggregate base from the latest project were paved in separate passes during fall 2008.

### Observed Distresses

Within a week of its scheduled opening, a series of unexplained cracks were suddenly appearing in the pavement. The cracks, varying in length from less than a foot to several feet, ran roughly parallel to and within several inches of the transverse joints near the centerline (figures 11 and 12).



Figure 11. MO-21 cracking.



Figure 12. Crack close-up.

### Field Investigation

Cores were drilled at locations centered on and slightly beyond cracks as well as on adjacent sawed transverse joints. The intent was to determine the extent and shape of the crack and whether or not a crack had also formed through the sawed joint (Figure 13). The cores revealed that the cracks were uncontrolled nearest the centerline (Figure 14), but eventually worked their way back into the sawed transverse joints (Figure 15).



**Figure 13. Cores at transverse joint.**



**Figure 14. Uncontrolled transverse crack.**



**Figure 15.** Crack merging with sawed joint.

The cores also clearly showed that the joint saw depth tapered to nothing at the centerline joint. The sawing subcontractor, somewhat inexperienced, had apparently tried to avoid overcutting into the adjacent paved lane. By doing so, however; uncontrolled cracks developed and continued parallel to the sawed joint until its depth (and influence) forced the crack into a confluence with the sawed joint some distance away from the centerline.

### **Distress Mitigation**

Since the majority of the uncontrolled cracks had not resulted in corner spalling and were well within the influence of the 18-in. (457-mm) dowel bars, the sawed joints next to the working cracks were sealed with an epoxy, thus reducing the possibility of future spalling. Compressible board blockouts were inserted near the joint / uncontrolled crack confluence to prevent the epoxy from infiltrating the working sawed joint. The few locations where spalls had already occurred were corrected with partial-depth repairs.

At all interim panels with no apparent distress, the joints were sawed through to their required depth for the full width of the panel to prevent the possibility of uncontrolled cracks occurring.

### **CONCLUSIONS**

The case studies presented in this paper were simply intended to provide real-life examples of early concrete pavement distress analysis and mitigation. Sometimes the solutions led to specification or design changes, but not always, because the roots of the problems were speculated and could not be thoroughly proven. Occasionally, lack of inspection by either the contractor or MoDOT might have played a role in allowing the situation to manifest, however; the occurrences were only prevalent enough to reinforce existing oversight procedures, not create new ones. The important result was always restoring a new pavement with the least intrusive repair techniques possible to functionally and structurally acceptable condition.

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# California's Perspective on Concrete Pavement Preservation

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## ABSTRACT

The California Department of Transportation (Caltrans) has established a strong pavement preservation program to preserve existing pavements and delay rehabilitation. To implement this effort, Caltrans developed a 5-year pavement preservation plan with dedicated funding and established the Pavement Preservation Task Group (PPTG) consisting of over 22 sub-groups, several of which apply to concrete pavement preservation specifically. The PPTG works as an advisory body to the California Pavement Preservation (CP2) Center located at the California State University in Chico.

This paper describes the major pavement preservation activities underway on pavement preservation of concrete pavements in California. In particular, it covers the following activities:

1. The development of the Maintenance Technical Advisory Guide (MTAG – 1) for Rigid Pavement Preservation.
2. The performance of diamond-grinding projects and the benefits of diamond grinding in life extension, ride quality improvement, and noise reduction.
3. The performance of dowel bar retrofits and the lessons learned from several projects.
4. The performance of full-depth slab repair, particularly with regard to the use of rapid strength concrete (RSC).

Caltrans plans for implementing pavement preservation on a more widespread basis within the State are also discussed, for both pavements in general and rigid pavements in particular.

## THE MAINTENANCE TECHNICAL ADVISORY GUIDE FOR CONCRETE PAVEMENTS

The Maintenance Technical Advisory Guide (MTAG) – Volume II was developed for use by Caltrans and other pavement professionals by the PPTG and the CP2 Center. The first edition was developed in July 2006, followed by the development of the second edition in March 2008 (1).

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Both editions focus on pavement preservation strategies for rigid pavements. The eight chapters of the second edition are as follows:

1. Introduction
2. Surface characteristics
3. Strategy selection
4. Joint resealing and crack sealing
5. Diamond grinding and grooving
6. Dowel bar retrofits
7. Isolated partial-depth repair
8. Full-depth repair including slab replacement

Training modules for each of the chapters were also developed, along with a separate module on distresses in concrete pavements.

Chapter 1 presents information on the pavement preservation concept, benefits of pavement preservation, and information on design and materials. Chapter 2 presents information on the most important surface characteristics (ride, surface texture and safety, noise, durability, and aesthetics) and why these factors are important. It also describes how each characteristic can be evaluated in the field. Chapter 3 describes the strategy selection process that should be followed to select the appropriate treatment for a given pavement with specific types of distress.

The remaining five chapters describe the concrete pavement preservation treatments most widely used in California. For each treatment, the purpose of the treatment as well as information on project selection, design, materials, specifications, and construction and inspection are presented. Limitations and troubleshooting guides are also included throughout.

A similar document exists for flexible pavement preservation as well.

For more detailed information on MTAG – Volume II, a complete copy of the rigid pavement document can be found on the Caltrans pavement maintenance Web site at [www.dot.ca.gov/hq/maint/MTA\\_GuideVolume2Ridgid.html](http://www.dot.ca.gov/hq/maint/MTA_GuideVolume2Ridgid.html) or on the CP2 Center’s website at [www.cp2info.org/center](http://www.cp2info.org/center).

## **DIAMOND GRINDING**

Diamond grinding of concrete pavements in California, when properly timed and applied, has been found to be a very effective and low-cost pavement preservation technique. Arguably, it may well be the most cost-effective technique currently available for pavement preservation of jointed concrete surfaced pavements.

Diamond grinding of concrete pavements has been shown to result in the following benefits:

1. *Smoother pavement surfaces*—Diamond grinding is generally triggered by an unacceptably high level of roughness (International Roughness Index or IRI), oftentimes accompanied by some joint faulting, that has developed over the years. As applied in California, diamond grinding generally results in appreciably improved levels of pavement smoothness.

2. *Longevity and life extension*—While the longevity of diamond grinding has been quantified, more effort is needed to determine the exact extended life benefits of grinding.
3. *Better skid resistance*—Diamond grinding generally restores an excellent level of skid resistance to the concrete pavement surface, resulting in increased safety for the traveling public.
4. *Improved noise levels*—Diamond grinding has also been shown to audibly reduce pavement–tire interface noise levels, as measured by on-board sound intensity (OBSI) noise measuring equipment.
5. *Lower agency costs*—Diamond grinding is one of the lowest cost—and oftentimes the most cost-effective—pavement preservation alternatives for concrete pavements.
6. *Lower user costs*—Diamond grinding improves fuel economy associated with driving on a smoother (i.e., lower IRI) pavement surface.

The following subsections quantify and explain the benefits of diamond grinding in more detail.

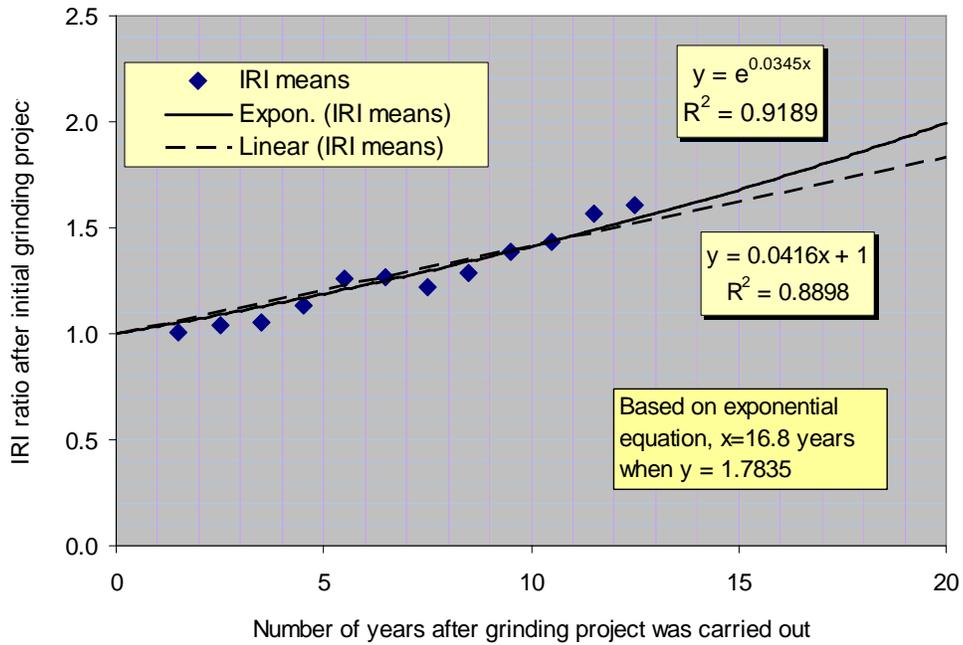
### **Smoothness and Longevity**

A study carried out by Caltrans (2) found that the average roughness in terms of IRI before a planned PCC grinding project by Caltrans was 165 in/mi (2.61 m/km) before grinding and 93 in/mi (1.46 m/km) after grinding. Thus the average before / after IRI grinding ratio in California is  $165.3 / 92.7 = 1.78$ . Moreover, this level of improvement (limited to California—mostly frost-free) results in an average treatment life of nearly 17 years before further rehabilitation or preservation (e.g., another grinding project, if feasible) is needed. These figures compare favorably with the reported national average longevity of a given grinding project of approximately 14 years (3). Moreover, since diamond grinding is almost always carried out on structurally sound (albeit having poor ride quality) concrete pavements in California in a timely manner, it is possible that the life extension of the pavement could be nearly as long as the treatment life itself.

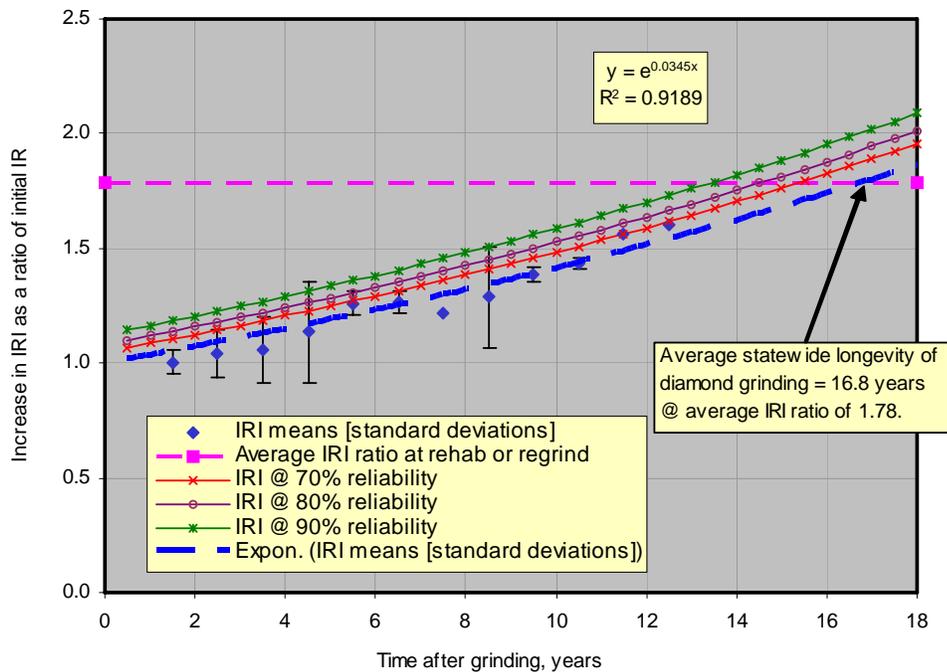
Figure 1 shows the average decrease in IRI after diamond grinding based on 29 statewide projects within California. Figure 2 shows the various confidence levels associated with the statistics for these same 29 projects. Two forms of regression equations were fit to these data. While both provide good  $R^2$  values, the exponential fit (Figure 1 and Figure 2) is recommended for use since it is unlikely that the progression of IRI will remain constant from cradle to grave, as the linear regression equations would imply.

For planning and design purposes, furthermore, it is recommended that the selected treatment life reliability level should be around 80 percent, which translates to about 14.5 years (see Figure 2)—perhaps slightly more for low- to medium-volume traffic roadways or slightly less for high-volume traffic roadways.

### Regression Equations for All IRI Data from California



**Figure 1. Change in the ratio of IRI over time as a function of the initial IRI after grinding (2).**



**Figure 2. Reliability levels for the expected survivability of California diamond-grinding projects (2).**

## Skid Resistance and Noise Properties

It is a well-known fact that grinding of a concrete pavement generally improves skid resistance, in many instances substantially. For example, changes in skid resistance due to diamond grinding of longitudinally tined concrete test sections were monitored and reported by the Arizona Department of Transportation (4). Table 1 shows the results of a diamond grinding study carried out in the Phoenix area (the percentage improvement for any given test section due to grinding is shown in parentheses in the After Grinding columns).

Friction is typically denoted by the Friction Value or the Coefficient of Friction using a Standard Test Method (ASTM or other), and is defined by:

$$\text{Friction Value} = F/L$$

where: F = frictional resistance to motion in the plane of the pavement–tire interface;

L = tire load force perpendicular to the pavement–tire interface.

**Table 1**  
**Friction Values and Percentage Improvement in Skid Resistance**  
**Due to Diamond Grinding (4)**

Test Section	Friction Values (% Improvement)					
	Before Grinding			After Grinding		
	Lane 1	Lane 2	Lane 3	Lane 1	Lane 2	Lane 3
1	0.52	0.55	*	0.65 (+25%)	0.63 (+15%)	*
2	*	*	0.56	*	*	0.66 (+18%)
3	0.49	0.51	*	0.69 (+41%)	0.69 (+35%)	*
4	*	*	0.53	*	*	0.67 (+26%)

\* Denotes an empty cell.

The test sections shown in Table 1 were all new (with longitudinal tining—the same surface texture used in California) before grinding. Therefore, the improvement in skid resistance due to diamond grinding is likely to be even better for older PCC pavements, many of which exhibit polished and exposed aggregate. As indicated in Table 1, the increase in friction values and, therefore, skid resistance ranged between 15 percent and 41 percent, with an overall average improvement for the six sections tested of 27 percent.

Another study carried out by Caltrans (5) showed similar benefits for sound or noise emissions from the tire–pavement interface. Such emissions are the primary source of audible noise emanating from high-speed roadways.

A measuring technique has been developed to accurately measure tire-pavement noise levels, called on-board sound intensity or OBSI. A typical setup using dual microphones to measure OBSI pavement–tire interface noise level is shown in Figure 3.



**Figure 3. OBSI microphone setup—two-probe system.**

Currently, there is no Standard Test Method for measuring OBSI. However, methods are currently under development at both ASTM and AASHTO using the two-microphone setup shown in Figure 3.

Typically, decibels (dB) are stated in terms of a weighted average value, called dB(A) or dBA, which covers the so-called *A-weighted* range of audible frequencies to the human ear. Audible sources of typical noise using two methods of measuring sound levels are shown in Figure 4.

Differences between before- and after-grinding OBSI levels on various freeways in California are shown in Table 2. These differences have been shown to be noticeable, meaning easily perceptible to the human ear, whether one is riding in a vehicle or is not moving but is within ear-shot of the roadway.

Note that OBSI levels are measured only some 5 in. (127 mm) from the source of the noise, i.e., from the tire–pavement interface as shown in Figure 3, and are thus wildly exaggerated compared to what a person would hear inside a typical vehicle or in the proximity of a high-speed roadway. Accordingly, the OBSI values shown in Table 2 should be used for comparative purposes only, not to ascertain the true dBA level heard by the traveling (or nearby) public. The main point to be made is that diamond grinding significantly reduces noise levels resulting from high-speed concrete roads, such as freeways and tollways.

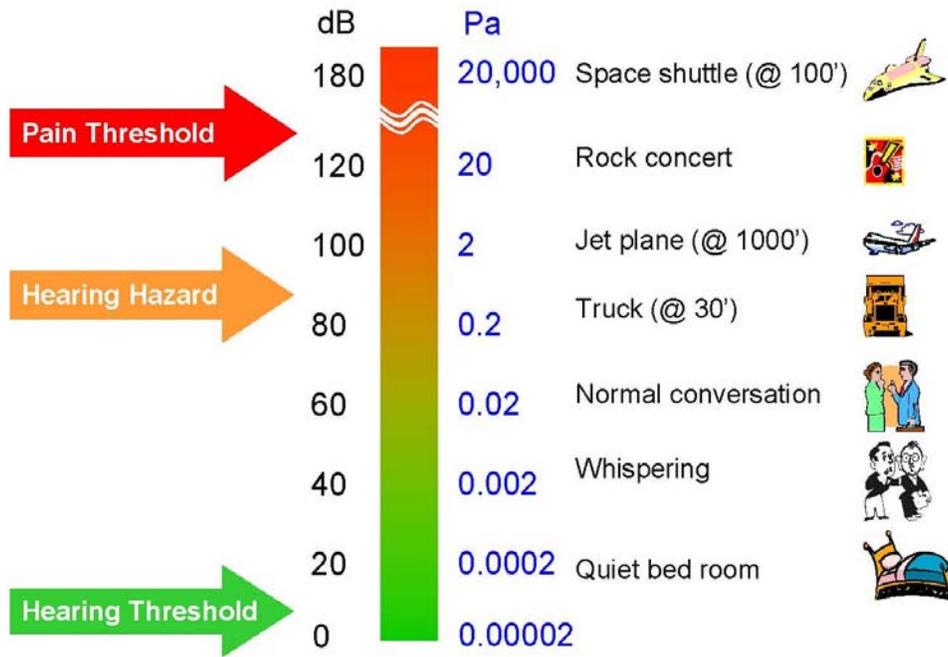


Figure 4. Conversion from sound pressure to perceptible noise levels and typical examples of dB(A) levels at normal distances from various sound sources (6).

Table 2  
Pre- and Post-Grind OBSI Values From Six Projects in California (5)

Site No.	Route Designation	County	Pre-Grind OBSI (dBA)	Post-Grind OBSI (dBA)	Post-Grind Reduction (dBA)
1	SR-60	San Bernardino	105.1	103.9	1.2
2	I-15	Riverside	103.9	101.8	2.1
3	I-5	Orange	104.0	101.3	2.6
4	I-405	Orange	104.4	102.0	2.5
5	I-5	Kern	103.2	100.0	3.2
6	I-5	Sacramento	104.7	100.3	4.4
<b>Average</b>			<b>104.2</b>	<b>101.5</b>	<b>2.7</b>

### Agency and User Costs

While agency and/or user costs of maintaining and using a pavement are difficult to quantify precisely, it is obvious that the longer the time between treatments, the lower the agency costs are to maintain the roadway. Furthermore, diamond grinding is a comparatively low-cost pavement preservation technique, with an expected life cycle of—on average—nearly 17 years in California (2).

Even more importantly from a roadway user perspective, it has been shown that pavements with a lower (smoother) IRI contribute to lower user costs. Vehicles use less fuel when driving

on smoother pavements, plus vehicle maintenance costs are reduced since a smooth pavement has a tendency to cause automobile and truck parts to last longer (eg. brakes, suspension, steering, tires, etc.).

Annual fuel cost savings have been estimated to be as much as \$25,000 per lane-mile, for trucks only, for the average ground PCC freeway pavement in California (7). Keep in mind that these cost savings are based on 2002 fuel costs and do not include other user costs, vehicle delay costs during construction, and passenger vehicle costs at all. Thus the actual user cost savings will be considerably higher than the \$25,000 per lane-mile indicated in the referenced 2002 Caltrans report (7).

Recalling that a diamond grinding project generally lasts 17 years or so in California, the *current* approximate cost of diamond (\$30,000 per lane-mile) pales in comparison to the savings achieved in fuel costs of more than \$25,000 per year during the first half of the 17-year design life, or at least \$210,000 over the expected life of the treatment.

It can be concluded that diamond grinding is a cost effective and easy-to-carry-out method of pavement preservation for concrete pavements—as long as the pavement programmed for diamond grinding qualifies as a candidate for this procedure.

### **Determining If a Concrete Pavement Is a Candidate for Diamond Grinding—or Not**

There are certain pavement distresses or conditions that should *preclude* the use of diamond grinding as a rehabilitation alternative, including the following:

1. Lack of structural integrity (e.g., voids under joints from pumping, excessive slab cracking, or progressive cracking over time).
2. Poor load transfer across transverse joints as indicated by excessive faulting, voids, or large differential deflections.
3. Spalling or other damage due to alkali–silica reactivity.
4. Freeze–thaw damage, including D-cracking (generally confined to freeze–thaw zones, which exist but are relatively uncommon in California).
5. Soft aggregates in the PCC slab, such as limestone, that are prone to polish (this may be overcome by widening the spacing of the grinding blades).

None of these distresses would be remedied by diamond grinding alone, and the problem would likely continue to cause pavement failure a short time after grinding. While such precluding factors are certainly considered under present Caltrans guidelines, this consideration is not yet based on any truly objective criteria or measurable threshold values to indicate the most optimum timing to carry out diamond grinding.

Still, even without a formal process for selecting a project for diamond grinding, it is seen that Caltrans achieves an average 17-year life cycle for diamond-grinding projects. Based on this, future pavement preservation initiatives in California will certainly utilize this very effective pavement preservation strategy to an even greater extent in the future.

## DOWEL BAR RETROFITS

Poor load transfer across transverse joints was listed above as one of the factors that should preclude the use of diamond grinding. Dowel bar retrofit (DBR) rectifies poor load transfer and can be used in combination with diamond grinding. DBR (see Figure 5)—if properly carried out—can be a very effective pavement preservation strategy. This approach can be used to remedy poor joint load transfer, thus rendering a diamond grinding project feasible—barring any other precluding conditions in the above list.



**Figure 5. Dowel bar retrofits at a transverse joint (8).**

This strategy can be very successful when properly designed and constructed. For example, an extensive project in 1999 along the I-10 freeway in Los Angeles County, California, proved to be a very successful DBR project prior to grinding (see Figure 6). Excellent materials and workmanship, together with an adequate time window before opening to traffic, were achieved throughout the project. By 2008, a mere 2 percent of the 22,000 dowel bars (11 per joint) have shown some signs of distress—some 9 years after the fact. Many other successful DBR projects have also been constructed in California since the early 2000s.

On the other hand, there have been reports of dowel bar retrofit distresses in some projects in California. These distresses appear to be related to materials or workmanship in the rather demanding process of dowel bar retrofitting. An example of one of these distressed projects is on I-5 in Orange County. In this project, it appeared that the grouting material was not properly consolidated (vibrated) between the edge of the slot and the dowel bar (see Figure 7), and there were many locations of misaligned dowels.



Figure 6. Successful dowel bar retrofit project on I-10 in Pomona, CA (9).

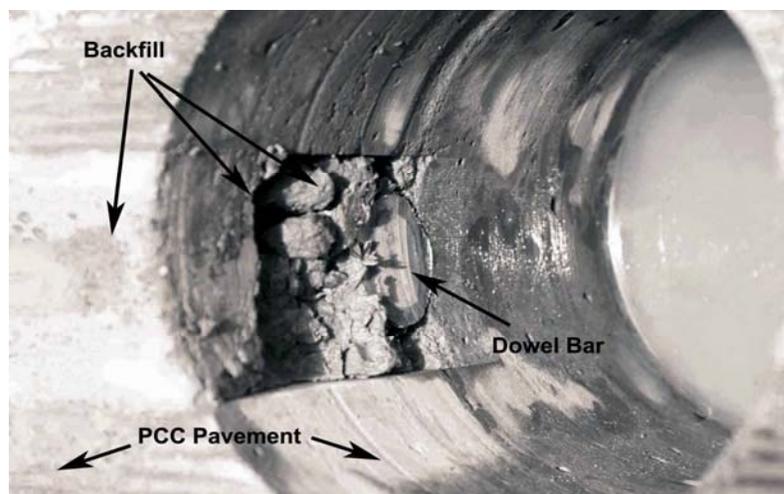


Figure 7. A poorly performing dowel bar retrofit on I-5 in Orange County (10).

In other instances of premature failure, it has been found that the dowel bar slots were not thoroughly cleaned prior to placing the backfill material, thus resulting in an inadequate bond between the backfill and the original concrete. There were instances where the dowels were misaligned because inexperienced contractors did not use gang-saws and had a hard time retaining the correct dowel bar alignment.

Dowel bar retrofits are best suited for pavements that are structurally sound but still exhibit low load transfer at joints and/or cracks. Pavements with little remaining structural life, as evidence by extensive cracking (more than 10 percent stage 3 cracking) or with high-severity joint defects are not good candidates for DBR.

In summary, there are five major factors to consider when evaluating a potential project for DBR (1):

- Structural condition of the slabs [should be good or better].
- Structural condition of the base [should exhibit low falling-weight deflectometer (FWD) deflections].
- Measured load transfer efficiency [ $< 60$  percent as measured by an FWD].
- Magnitude of faulting [ $> 0.10$  and  $< 0.5$  in. (2.5–13 mm)].
- Condition of joints and/or cracks [moderate severity spalling or better].

Finally, and most importantly, good materials and workmanship and adequate time allowed before opening to traffic are mandatory. DBR retrofits, when warranted, are generally followed by grinding to restore smoothness properties.

## **FULL-DEPTH SLAB REPAIR**

Full-depth slab repairs, or replacements, are used when isolated slab distresses are too severe to warrant other forms of pavement maintenance and preservation. Full-depth repairs may be used with or without followup diamond grinding, depending on the overall pavement condition and the remaining life expectancy of the untreated pavement. A typical photo of a removed slab prior to a slab replacement panel is shown in Figure 8.

Full depth slab repair involves full-depth and -width slab removal followed by cast-in-place replacement of slabs (or partial-length slabs) in an existing pavement. Typically the minimum length requirement is 6 ft (1.8 m); however, when repair areas are closely located it may be more cost effective to substitute a larger area—usually the full length of the original slab. Guidelines associated with full-depth slab repairs, such as reestablishing joints, etc., are detailed in the Caltrans *Slab Replacement Guidelines* document (11).



**Figure 8. A full-depth concrete slab replacement project (11).**

Full depth slab repair can address a wide variety of distresses, including transverse cracks, longitudinal cracks, joint spalling, and blowups. According to the FHWA (12), Table 3 shows the typical distress types and severity levels where full-depth repairs should be applied.

**Table 3  
Typical Distresses Addressed by Full-Depth Repairs  
for Jointed Concrete Pavement (12)**

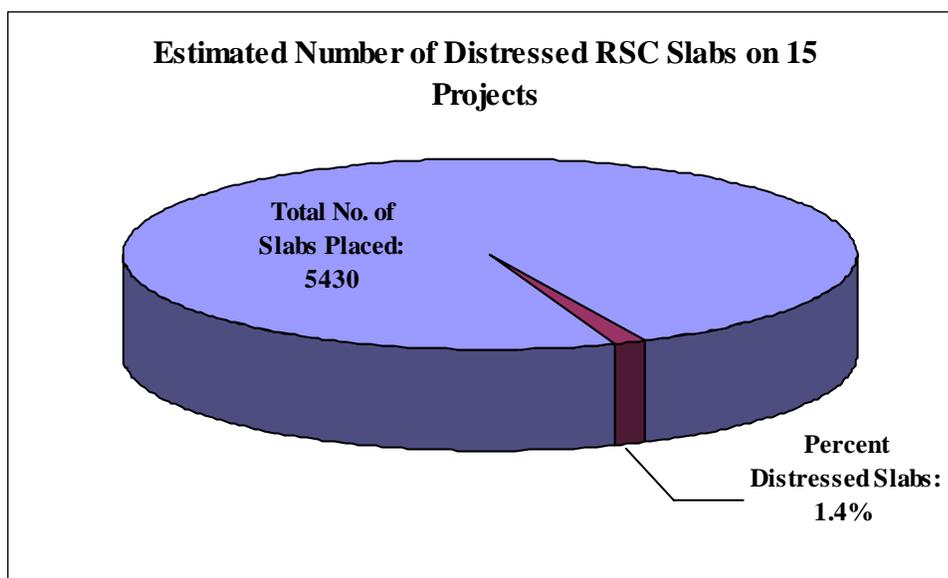
<b>Distress Type</b>	<b>Severity Levels That Require Full-Depth Repair</b>
Transverse cracking	Medium, High
Longitudinal cracking	Medium, High
Corner break	Low, Medium, High
Spalling of joints	Medium <sup>1</sup> , High
Blowup	Low, Medium, High
Reactive aggregate spalling	Medium <sup>1</sup> , High
Deterioration adjacent to existing repair	Medium <sup>1</sup> , High
Deterioration of existing repairs	Medium <sup>1</sup> , High

<sup>1</sup> Partial-depth repairs can be optionally used if the deterioration is limited to the upper one-third of the pavement slab.

A wide variety of materials are available for full-depth slab repairs. The selection of a suitable material will depend on the project's environmental, design, and funding requirements along with desired performance. While these slab repair materials include conventional portland cement concrete (PCC) mixtures and bituminous materials, rapid-strength concrete (RSC—also known as fast-setting hydraulic cement) is most often used in California due to the need for early opening to traffic, most commonly by the early hours of the following morning.

Slabs constructed using RSC mixtures are generally opened to traffic in less than 4 hours after casting. RSC mixtures are designed to develop a flexural strength in excess of 400 lbf/in<sup>2</sup> (2.75 MPa) within 4 hours. This requirement was developed using finite element analysis and is based on the fact that if the slab is subjected to traffic prior to obtaining this minimum strength, the durability and life expectancy of the slab will likely be compromised.

RSC cement mixtures have performed very well in California, indicating both excellent concrete design and construction practices (13). A total of 15 statewide projects were surveyed by Caltrans recently for premature distresses on RSC slab replacements. The results were excellent (see Figure 9), with less than 2 percent of the surveyed slabs showing any significant surface distresses. Where distresses were observed, shrinkage cracks were the most prevalent distress type.



**Figure 9. Percent distressed slabs on surveyed RSC projects (13).**

Detailed construction guidelines to assist contractors in the proper use of RSC were published by Caltrans in January 2004 (11).

## **PERTINENT PAVEMENT PRESERVATION ISSUES AND RECOMMENDATIONS**

The preceding sections have outlined Caltrans experience with various rigid pavement preservation strategies. It should be emphasized that the concept of “preserving rigid pavements as rigid” should be the motto of concrete pavement preservation. There is a need to refine and determine the appropriate pavement preservation tools to determine the right fix at the right time.

Examples of issues that have been implemented and will need to be explored further:

- Determine the optimum timing and utilize the remaining life concept in pavement preservation. There is an optimum time for each strategy depending on appropriate threshold values and the remaining life of the existing pavement. For example, DBRs should not be used on a project near the end of its performance life.
- Determine the structural integrity of cement treated bases. For example, several attempts were made on the 710 Freeway in the Los Angeles basin to determine the integrity of the base to help determine the most appropriate design strategy using FWD and backcalculation techniques. This proved to be difficult to determine in advance without the use of more costly, destructive test methods.
- Ensure that asphalt concrete as an alternative slab replacement strategy is used as a temporary measure only until a viable rigid pavement strategy can be implemented. The use of asphalt concrete to replace distressed slabs is detrimental to the adjacent slabs because it creates a free edge effect and can cause accelerated deterioration of the adjacent slabs.
- Determine the economic benefits of rigid pavement preservation. One of the major issues for decision makers is quantifying the benefits of preservation as compared to rehabilitation alternatives.
- Use nondestructive test methods to help determine the most appropriate strategies. Non-destructive methods need to be refined for use in pavement preservation. For example, there is a need to determine the support conditions under the slab and detect voids more reliably so that preservation treatments can be applied more successfully.
- Use of RSC in California has become a standard strategy and will continue to be used as such. However, more effort will be needed to address shrinkage.
- The use of precast and prestressed concrete panels will become a standard strategy in urban areas. This technique has been shown to be successful when properly constructed.
- Use a treatment design life of approximately 14 years for most diamond grinding projects is recommended, not the 17-year, statewide average, treatment life.
- Investigate the use of second- and third-generation concrete grinding to mitigate noise, skid resistance, and loss in ride quality.
- Another technique that needs to be investigated further is crack and joint sealing and the proper techniques to achieve the best performance.

- Investigate the use of polyester concrete as a backfill material in DBRs. Laboratory tests have shown a stronger bond with the existing concrete results from the use of polyester concrete.
- Use analytical methods to determine the benefits of pavement preservation to complement actual performance data. This is needed to bridge the existing gap between theory and practice as well as the current lack of adequate performance data.

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#### **DISCLAIMER**

This report solely reflects the technical views of the authors. The contents do not necessarily reflect the official views or policies of California Department of Transportation or the State of California.

# **New Applications for Thin Concrete Overlays: Three Case Studies**

Shiraz Tayabji,<sup>1</sup> Andrew Gisi,<sup>2</sup> Jason Blomberg,<sup>3</sup> and Dan DeGraaf<sup>4</sup>

## **ABSTRACT**

The need for optimizing preservation and rehabilitation strategies used to maintain the Nation's highway pavements has never been greater. Concrete overlays have a long history of use to preserve and rehabilitate concrete and asphalt pavements, and many of the practices are well established. However, of recent origin are techniques that use thinner concrete overlays with shorter joint spacing. Field experience over more than 15 years with the thinner concrete overlays under a range of traffic and site conditions has demonstrated their viability as a cost-effective solution to extend the service life of deteriorated asphalt and concrete pavements.

Concrete overlays can be designed for a range of traffic loadings to provide long performance lives, 15 to 40+ years, and to meet specific needs. Well-designed and well-constructed concrete overlays require low maintenance and can have low life-cycle costs. Thin concrete overlay applications include bonded and unbonded overlays over existing asphalt, concrete, and composite pavements.

This paper provides a review of thin concrete overlays applied as bonded or unbonded overlays. In addition, three recent case studies are presented that illustrate the wide range of applications of thin concrete overlays.

## **INTRODUCTION**

The need for optimizing preservation and rehabilitation strategies used to maintain the Nation's highway pavements has never been greater. Concrete overlays have a long history of use to preserve and rehabilitate concrete and asphalt pavements, and many of the practices are well established. However, of recent origin are techniques that use thinner concrete overlays with shorter joint spacing. Field experience over more than 15 years with the thinner concrete overlays under a range of traffic and site conditions has demonstrated their viability as a cost-effective solution to extend the service life of deteriorated asphalt and concrete pavements.

The Federal Highway Administration (FHWA) has initiated several activities to support technology transfer related to concrete overlays. These activities include reviews, on a regional or statewide basis, of current applications of concrete overlays, identification of gaps in technology, and assistance in developing a program—jointly with State departments of transportation (DOTs) and industry—for technology transfer and demonstration projects. This paper provides a review of thin concrete overlays applied as bonded or unbonded overlays. In addition, three

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new applications are presented that illustrate the wide range of applications of thin concrete overlays to rehabilitate existing distressed pavements.

## **OVERVIEW OF CONCRETE OVERLAYS**

Concrete overlays offer a broad range of applications for preserving and rehabilitating asphalt, concrete, and composite pavements. Concrete overlays can be designed for a range of traffic loading to provide long performance lives, 15 to 40+ years, and to meet specific needs. Concrete overlays can be constructed rapidly and with effective construction traffic management, and well-designed and well-constructed concrete overlays require low maintenance and can have low life-cycle costs. An important benefit of concrete overlays is that concrete overlays can be applied to a wide variety of existing pavements exhibiting a range of performance issues. Applications include the following:

- Over existing asphalt pavements
  - Bonded overlay of asphalt pavements
  - Unbonded (directly placed) overlay of asphalt pavements
- Over existing concrete pavements
  - Bonded overlay of concrete pavements
  - Unbonded (separated) overlay of concrete pavements
- Over existing composite pavements
  - Bonded overlay of composite pavements
  - Unbonded (directly placed) overlay of composite pavements

The family of concrete overlays is illustrated in Figure 1. The applicability of concrete overlays is summarized in Figure 2.

# System of Concrete Overlays

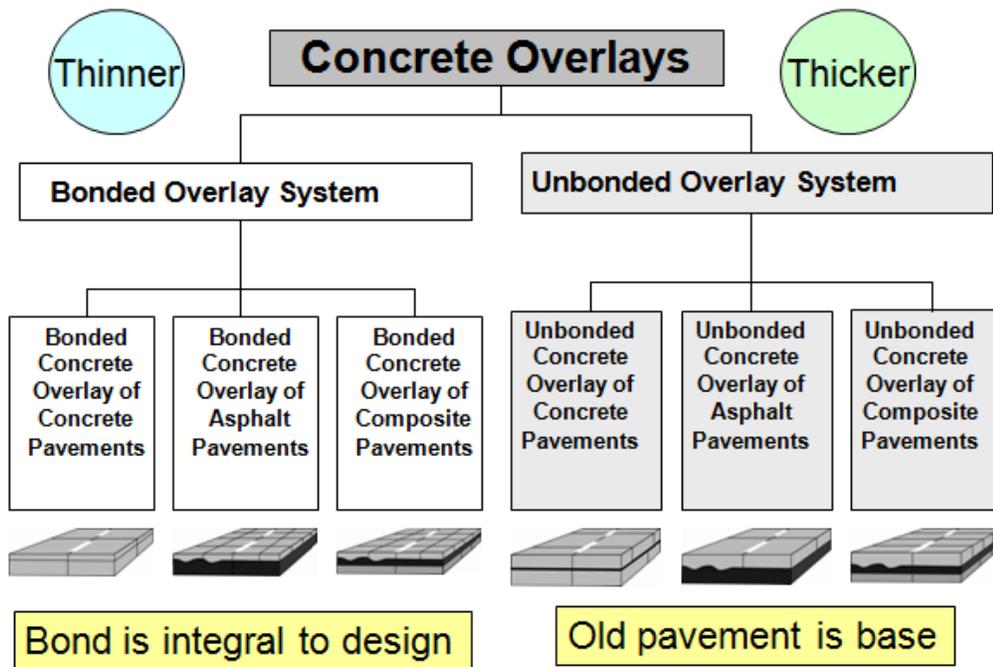


Figure 1. Family of Concrete Overlays (NCPTC 2008).

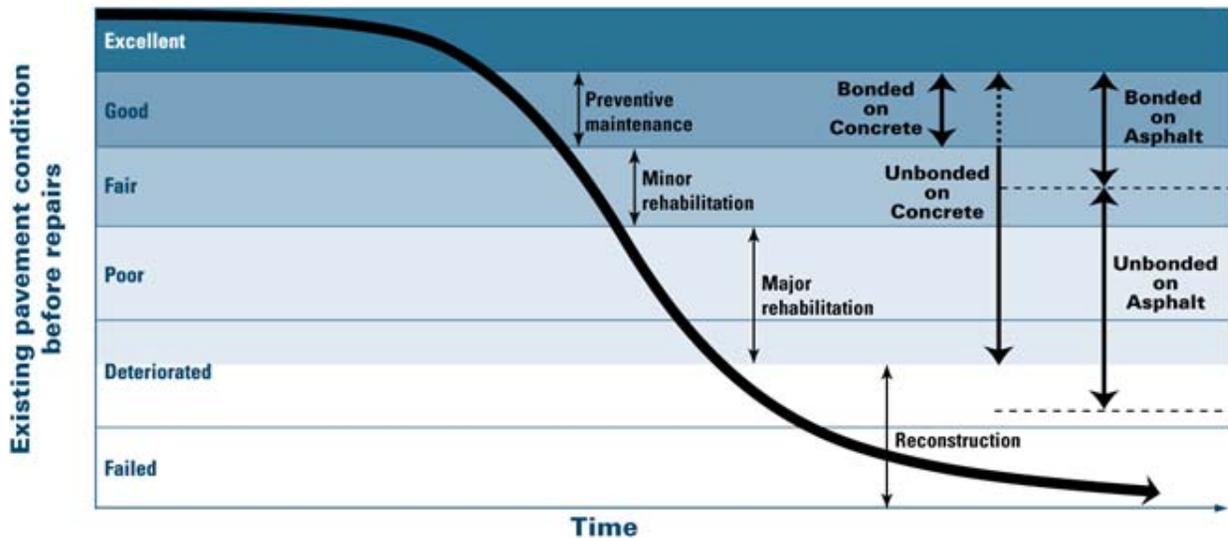


Figure 2. Applicability of Concrete Overlays (NCPTC 2008).

## Bonded Overlays

Bonded overlays are typically thin, 2 to 6 in. (50 to 150 mm) in thickness. When bonded to a milled asphalt surface, the overlay panels are typically 6 by 6 ft (1.8 by 1.8 m) or less in dimension. When bonded to a prepared concrete surface, the overlay jointing pattern matches the jointing pattern of the existing jointed concrete pavement. In the case of continuously reinforced concrete pavement, transverse jointing is not provided.

Thin bonded overlays of asphalt concrete (AC) pavement are of recent origin. Even though some early trial installations were constructed during the early 1990s, it was only after the late 1990s that the technique became more widely accepted. As a result, the typical in-service experience with thin bonded overlays over AC pavement is less than 15 years. However, performance of the many miles of thin bonded overlays in many States indicate that properly designed and constructed thin bonded overlays can provide service life of at least 15 years and possibly greater than 20 years.

The use of thin bonded overlays over a composite pavement needs to be carefully assessed. A high potential exists for development of reflection cracking in the overlay if there is a severe level of reflection cracking in the composite pavement and truck traffic volume is high. If the reflection cracking is not extensive, then a bonded overlay may still be considered by using deformed steel bars over the cracking.

The experience with thin bonded overlays of existing concrete pavements has been mixed. For highway applications, these overlays range from 2 to 4 in. (50 to 100 mm) in thickness and are used to eliminate any surface defects in an existing concrete pavement, increase structural capacity; and improve surface friction, noise, and rideability. Typically, bonded overlays of existing concrete pavements are used to increase the structural capacity of existing concrete pavements while the existing pavements are still in good structural condition with only some surface distress. When constructed well, these overlays provide good service, typically 15 to 20 years. However, because these overlays are very sensitive to existing pavement surface preparation and curing and joint sawing operations, these overlays can exhibit early failures, typically joint corner delamination leading to cracking in the overlay. One of the three case studies presented in this paper details an innovative inlay type of application of a thin bonded concrete overlay over a distressed concrete pavement to restore the condition of the existing pavement.

## Unbonded Overlays

Unbonded overlays, referred to also as directly placed overlays, are of two types:

- *Conventionally thick overlays*, 6 in. (150 mm) or thicker, are full-width and have transverse joint spacing of 12 to 15 ft. (3.7 to 4.6 m).
- *Thinner overlays* are 4 to 6 in. (100 to 150 mm) thick, and the overlay panels are typically 6 by 6 ft (1.8 by 1.8 m) or less in dimensions.

Irrespective of thickness, unbonded overlays are typically placed over an AC surface—either an asphalt pavement or an asphalt interlayer/resurfacing placed over a concrete pavement. When used over existing concrete pavements, an interlayer is required for good performance.

The role of the interlayer is to:

- Isolate overlay from existing pavement.
  - Prevent reflection cracking.
  - Prevent bonding/mechanical interlocking.
- Provide level surface for overlay construction.
- Provide a softer interlayer—less curling stress.

The interlayer typically used is a dense-graded, hot-mix asphalt 1 to 2 in. (25 to 50 mm) thick. As discussed in this paper, studies are underway to evaluate the use of thicker geotextile fabric as the interlayer material.

Conventionally thick unbonded concrete overlays of AC and concrete pavements are routine in application and are widely used to rehabilitate existing AC and concrete pavements that exhibit a higher level of distress. Unbonded overlays require very little pre-overlay repair. Of recent origin are the thin unbonded concrete overlays of AC and composite pavements. This application is a derivative of the thin bonded concrete overlays of AC pavements. Thin unbonded overlays are typically 4 to 6 in. (100 to 150 mm) thick, and the panels are typically 6 ft by 6 ft (1.8 by 1.8 m). This technique was developed in Michigan where it has been used successfully on many projects. One of the three case studies presented in this paper details such an application in Michigan.

An innovative thin unbonded concrete overlay design incorporates thick geo-fabric as the separator layer. The established practice in the US for a separation layer for unbonded overlays has been to use a 1 to 2 in. (25 to 50 mm) hot mix AC layer. A standard German practice is to use a geo-fabric, 0.2 in. (5 mm) thick, as a separation layer between a cement-treated base and concrete surface for new construction. U.S. investigators are applying this concept to thin unbonded concrete overlays. A project was constructed in Missouri during 2008 that used a thin geo-fabric as a separation layer for a thin unbonded overlay. This project is one of the three case studies presented in this paper.

### **Thin Concrete Overlay Design and Construction Considerations**

The following are key design and construction considerations for use of thin concrete overlays:

1. Overlay type: Plain concrete.
2. Jointing:
  - a. No dowel bars at transverse joints (for overlays less than 8 in. [200 mm] thick).
  - b. Tie bars may be used along exterior longitudinal joints. Colorado DOT requires use of tie bars along longitudinal joints. The Michigan practice is not to use tie bars.
3. Concrete mixture: similar to conventional concrete paving; use of smaller maximum aggregate size for thin overlays; rapid-setting concrete may be used if needed.

4. Concrete placement.
  - a. Similar to conventional concrete paving: slipform or fixed-form construction may be used.
  - b. Curing and joint sawing very critical for thin overlays.
5. Surface requirements (ride and texture): similar to conventional concrete pavement.

### **Concrete Overlays Guide**

With support from FHWA, the National Concrete Pavement Technology Center (NCPTC) at Iowa State University has developed a best-practices *Guide to Concrete Overlay Solutions* (NCPTC 2008). Prepared by a joint industry/State DOT Task Force on Concrete Overlays, the guide presents an overview of concrete overlay systems for resurfacing or rehabilitating pavements and includes detailed guidelines for overlay use:

- Evaluating existing pavements to determine whether they are good candidates for concrete overlays.
- Selecting the appropriate overlay system for a specific pavement condition.
- Managing concrete overlay construction work zones under traffic.
- Accelerating construction of concrete overlays when appropriate.

### **RECENT INNOVATIVE THIN CONCRETE OVERLAY PROJECTS**

As discussed, thicker unbonded concrete overlays have been routinely used to extend the service life of existing concrete and asphalt pavements. Although thinner bonded concrete overlay of concrete pavements has a long history, the technique has been typically used to strengthen pavements that are in good condition. The thin bonded overlays of AC pavements are establishing good service records, and the design and construction procedures are becoming more established. Of recent practice is the use of thin unbonded overlays over existing concrete and composite pavements, especially the technique that replaces the standard AC interlayer with a thick geotextile fabric. Case studies of three innovative projects are presented next:

1. Thin bonded inlay of a deteriorated concrete pavement in Kansas.
2. Thin unbonded overlay of a deteriorated concrete pavement in Missouri. This project used a geotextile interlayer in lieu of a standard AC interlayer.
3. Thin unbonded overlay of a deteriorated composite pavement in Michigan.

#### **Thin Bonded Concrete Inlay—Kansas**

A thin bonded inlay was constructed during 2008 along a section of I-35 in Johnson County, Kansas. The existing concrete pavement was constructed in 1985 and consisted of a jointed reinforced concrete pavement (JRCP) 9 in. (225 mm) thick over a cement-treated base 4 in. (100 mm) thick. The JRCP joint spacing was 30 ft (9.1 m).

The JRCP was exhibiting joint distress in the form of surface spalls several inches wide and about 2 in. (20 mm) deep. Visual observation of the spalls shows that the sand/cement mortar in the concrete was deteriorating because of a poor entrained air system. The condition of the existing pavement is shown in Figure 3. The coarse limestone aggregate was found to be intact, and only an occasional limestone aggregate appeared to be affected by “D-cracking.” The JRCP was reinforced using wire mesh placed about 2.25 to 4.5 in. (57 to 114 mm) from the surface. The concrete flexural strength was estimated to be about 680 lbf/in<sup>2</sup> (4.7 MPa).

The construction process consisted of full-depth patching of some joints, milling 2 in. (50 mm) of the surface, shot blasting of the milled surface, application of a cement slurry (3 parts water and 1 part cement), then application of a plain concrete inlay 2 in. (50 mm) thick. The slurry application and concrete placement are shown in Figure 4. Joints were sawed into the inlay over the existing joints. Sawcut depth was full-depth plus 0.5 in. (12.5 mm). The contractor had 10 weeks to complete the entire project. Four weekends were allowed for the inlay placement. The construction specification allowed the pavement to be opened to construction traffic at 340 lbf/in<sup>2</sup> (2.3 MPa) flexural strength or 1,800 lbf/in<sup>2</sup> (12.4 MPa) compressive strength. The inlay was diamond ground.

Almost a year after construction, the concrete inlay is performing well. There were no early age failures. Kansas DOT estimates that the inlay will extend the service life of the existing pavement by at least 15 years.



**Figure 3. Condition of existing concrete pavement.**



**Figure 4. Slurry application and concrete placement.**

### **Thin Unbonded Overlay—Missouri**

A thin innovative unbonded concrete overlay, incorporating a geotextile fabric as an interlayer, was constructed in Missouri during 2008. The 3.13-mi (5-km) long project is located along a section of Route D in Jackson Count, between Routes 150 and 58. The existing jointed concrete pavement, 8 in. (200 mm) thick with 30 ft (9.1 m) joint spacing, was constructed in 1986. One-inch (25 mm) dowel bars were used at transverse joints. As of 2008, the pavement section was exhibiting severe D-cracking along both transverse and longitudinal joints, and it was estimated by the Missouri DOT (MoDOT) that up to 25 percent of the pavement area would need full-depth repair prior to a conventional AC overlay. The existing pavement condition is shown in Figure 5.



**Figure 5. Condition of existing concrete pavement.**

The MoDOT investigated several alternatives for rehabilitating the Route D pavement. These alternatives included new construction using AC and PCC, conventional unbonded overlay 8 in. (200 mm) thick, and rubblize with 12 in. (300 mm) AC overlay. Based on discussions between MoDOT and industry, it was decided to consider an experimental thin unbonded con-

crete overlay incorporating a nonwoven geotextile fabric as an interlayer in lieu of a conventional AC interlayer. The concrete overlay design developed required use of an unbonded concrete overlay, 5 in. (12.5 mm) thick with 6 ft by 6 ft (1.8 by 1.8 m) panels. The same design was selected for the shoulders. This technique of using a geotextile fabric as an interlayer is an extension of the German practice of using a geotextile fabric 0.2 in. (5 mm) thick to separate concrete from a cement-treated base for new concrete pavement construction. The Germans have used this application successfully to eliminate bonding between the concrete and cement-treated base and to reduce curling stresses as a result of the cushioning effect provided by the 0.2 in. (5 mm) thick fabric. Two nonwoven geotextile were utilized for this project. These were Geotex 1201 and Geotex 1601, manufactured by Propex. These geotextiles were slightly thinner than the 0.2-in. (5-mm) geotextile fabric used in Germany.

The overlay construction steps included the following:

1. Pre-overlay repairs—Some of the severely deteriorate joint areas were patched using flowable concrete.
2. Fabric placement—The geotextile material was delivered to the project in 300-ft (90-m) rolls, as shown in Figure 6. The fabric was 15 ft (4.6 m) wide. As shown in Figure 6, a telescopic forklift was used to place the fabric. Two rolls of geotextile were used to span the width of the pavement
3. Fabric fastening—A Hilti gas-powered fastening system was used to fasten the fabric onto the underlying concrete pavement.
4. Concrete placement—The geotextile material was wet prior to concrete placement. Concrete was placed using a conventional slipform paver riding directly on the fabric, as shown in Figure 7. No problems were reported with the geotextile during concrete placement.
5. Concrete curing—A curing compound was used and applied at a higher rate than is typically used for new concrete pavement construction.
6. Concrete joint sawing—Joints were sawed in a timely manner and no early-age problems were observed.

The MoDOT is satisfied with the initial project outcome and will be monitoring the performance of the overlay over the next few years. The thin unbonded overlay is performing well after about 9 months of service.



**Figure 6. Geotextile fabric placement.**

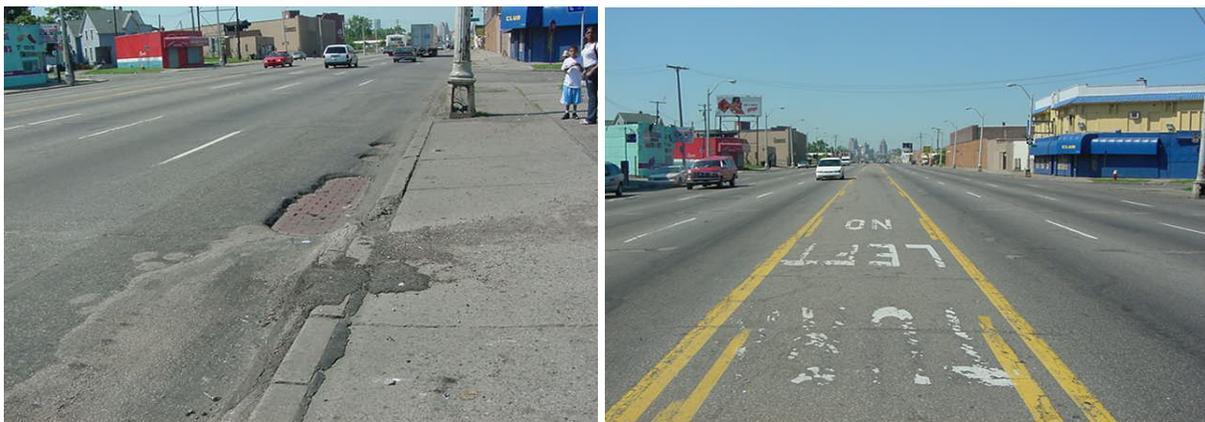


**Figure 7. Slipform paver over the fabric and completed unbonded overlay.**

### **Thin Unbonded Overlay of a Composite Pavement—Michigan**

In summer 2003, the Michigan Department of Transportation (MDOT) Metro Region decided to pursue a thin concrete overlay demonstration project on M-3 (Gratiot Avenue) between I-75 and I-94 in the city of Detroit. The length of the project was 3.17 miles, encompassing approximately 198,000 yd<sup>2</sup> (165,000 m<sup>2</sup>) of pavement. The original underlying concrete pavement dates back to the early 1900s and had been overlaid several times with hot-mix AC. The pavement consists of nine lanes; a parking lane, three through lanes in each direction, and a common center turn lane. Based on the review of the available plans, the existence of underlying brick pavers along the outside parking lane was discovered. The surface condition of the composite pavement had deteriorated significantly and rehabilitation of the roadway was considered necessary. Visual inspection of the composite pavement lane indicated localized areas of questionable base support. In general, an AC overlay will take the shape of the base below, and areas of poor support are easy to identify. Once the surface AC milling was completed, an

immediate site review was conducted to determine the extent of potential support problems. Figure 8 shows the condition of the composite pavement.



**Figure 8. Condition of existing composite pavement.**

Based on discussions between MDOT and the industry, it was decided to construct a thin unbonded concrete overlay, with smaller 6 by 6 ft (1.8 by 1.8 m) panels, over the composite pavement. Nationally, very few composite pavements have received this type of rehabilitation treatment. From October 2003 through April 2004, a cross-functional team had six working meetings to develop strategies and specifications for the application of this pavement treatment. The project was let for bid during August 2004. The lowest bid was \$ 7.17 million.

Areas of weakest base support, typically near inlets, were repaired using full-depth patches, and areas exhibiting less severe deterioration were repaired using partial-depth removal of concrete with hand-patched asphalt or concrete replacement. An asphalt milling depth of 5 in. (12.5 mm) was established to maintain curb depth and cross-slope. There were 350 manhole structures in the composite pavement, and a core drill 5-ft (1.5-m) in diameter was used to isolate manhole structures from the pavement. Prior to milling the AC surface layer, each phase of the core containing the manhole casting was removed. The castings were adjusted to the new pavement grade prior to concrete placement. After cold milling of the AC surface layer, a 4-in. (100-mm) concrete overlay was placed over a 1-in. (25-mm) asphalt separator layer. As part of this project, experimental sections with two different asphalt separator mixes and sealed versus unsealed joints were also constructed. The slab panels were 6 in. (150 mm) thick and 6 by 6 ft (1.8 by 1.8 m) (maximum) in dimension. The majority of the work was initiated in 2005 and completed in late fall 2005. Figure 9 shows the pre-overlay activities—full-depth repair and median strengthening. Figure 10 shows the AC interlayer placement to provide a smooth and uniform grade for concrete placement. Concrete was placed using slipform paving as shown in Figure 10. The completed thin unbonded concrete overlay is shown in Figure 11.

The MDOT Metro Region is satisfied with the initial project outcome and is planning additional demonstration projects. The unbonded overlay is performing well after about 3 years of service.



**Figure 9. Pre-overlay repairs.**



**Figure 10. AC interlayer and concrete placement.**



**Figure 11. Completed thin unbonded overlay.**

## **SUMMARY**

As discussed in this paper, thin concrete overlays can provide cost-effective solutions for rehabilitation of existing asphalt, concrete, and composite pavements. The three case studies presented illustrate new applications for thin concrete overlays. These applications were made possible because of sound engineering decisions and a desire to continue to explore new ways to extend existing technologies. The success of these innovative applications will provide pavement engineers with new tools that can be used with confidence to cost-effectively rehabilitate existing pavements. A key advantage of using thin concrete overlays is that it allows existing pavement to remain in place, thus significantly reducing the construction time for the rehabilitation activity.

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## **ACKNOWLEDGMENTS**

The opinions presented are those of the authors and do not necessarily reflect the views or opinions of the Missouri Department of Transportation, the Kansas Department of Transportation, or the Michigan Concrete Pavement Association.

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# Optimization of Concrete Maintenance to Extend Pavement Service Life

Craig White<sup>1</sup> and David K. Hein<sup>2</sup>

## ABSTRACT

The Highway 407 ETR concession in Ontario, Canada, is responsible for the management of a large highway network for a period of 99 years. As a part of that concession, 407 ETR manages over 600 lane-km (373 lane-mi) of exposed concrete highway. 407 ETR has a very active pavement maintenance and preservation program to maximize the life of the pavement. Also, as a private-sector concession, 407 ETR has the ability to act very quickly and actively partner with industry to promote innovation and to avoid the pitfalls of low-bid procurement.

407 ETR has an active pavement management and maintenance management system that is used for future needs planning but, more importantly, is also used to identify maintenance and rehabilitation needs early in their development so that they can be addressed using a less expensive preventive maintenance program. It is in the best interest of the concession to maximize the life of the pavement, to provide a high-quality riding surface for the paying public, and to avoid disruptions to traffic flow and revenue. To accomplish this, 407 ETR has employed many maintenance techniques including slab stitching, dowel bar retrofit, joint retrofit, diamond grinding, shot blasting, longitudinal grooving, underslab sealing and lifting, targeted slab replacement, microsurfacing, and other proprietary thin asphalt surfacings.

This paper reviews each of the concrete pavement maintenance and repair techniques used by 407 ETR over the past 10 years, discusses their performance, provides guidance on “what to do and what not to do” aspects of their use, and compares their life-cycle benefits.

## INTRODUCTION

The Highway 407 ETR concession in Ontario, Canada, is responsible for the management of a highway network for a lease period of 99 years. As a part of that concession, 407 ETR manages over 600 lane-kilometres of exposed concrete highway. 407 ETR’s pavement maintenance and preservation program is designed to maximize the life of the pavement, provide a high-quality riding surface for the paying public, and avoid disruptions to traffic flow and revenue.

The pavement management and maintenance management system is used for future needs planning, but, more importantly, it is used to identify maintenance and rehabilitation needs early in their development so that they can be addressed using a less expensive preventive maintenance program. Also, as a private sector concession, 407 ETR has the ability to act very quickly and actively partner with industry to promote innovation in pavement preservation and to avoid some of the pitfalls of required low-bid procurement. This allows 407 ETR to test new pavement preservation technologies and procedures by working with in-

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dustry to quickly and effectively determine the applicability and usefulness of these technologies.

As a result of the active pavement maintenance and preservation program, 407 ETR has been able to maintain its pavement condition rating above a value of 90 out of 100 for the past 10 years.

## PAVEMENT PRESERVATION

The key to cost-effective preservation of pavements is to have a toolbox that includes a variety of effective preventive maintenance techniques that can be effectively used to extend the service life of the pavement. Preventive maintenance treatments are applied to prevent premature deterioration of the pavement or to retard the progression of pavement defects. As a result, the pavement is maintained at a relatively high level of service. This type of approach is illustrated in Figure 1.

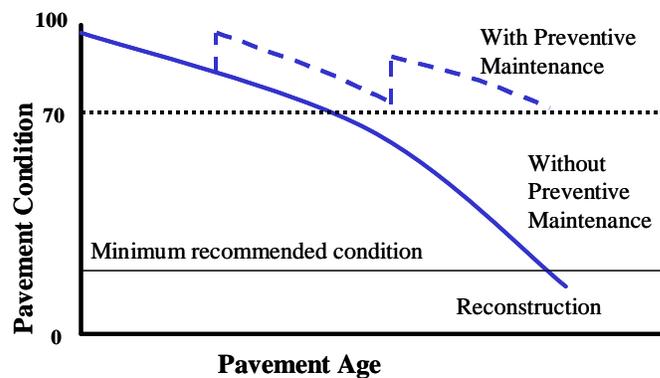


Figure 1. Pavement deterioration with and without preventive maintenance.

To accomplish this, 407 ETR has employed many maintenance techniques including slab stitching, dowel bar retrofit, joint retrofit, texturization (diamond grinding, shot blasting, longitudinal grooving), underslab sealing and lifting, targeted slab replacement, microsurfacing, and other proprietary thin asphalt surfacings.

## PAVEMENT PRESERVATION TECHNIQUES

Highlights and lessons learned from the implementation of each of the concrete pavement maintenance and repair techniques used by 407 ETR over the past 10 years including performance on “what to do and what not to do” aspects of their use are provided below.

### Crack Stitching

407 ETR has made extensive use of crack stitching over the past 10 years. Crack stitching typically includes the following steps:

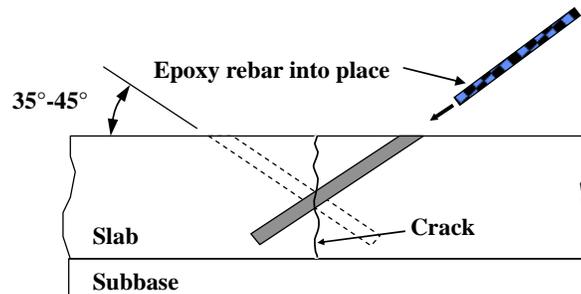
- Drilling holes at a 35-to-45° angle so that they intersect the crack at about the slab middepth (Figure 2).
- Cleaning of holes by air blasting.
- Injecting epoxy into the hole (in sufficient volume to fill all the available space after a tie bar is inserted).

- Inserting a deformed tie bar into the hole, leaving about 25 mm (1 in.) between the pavement surface and the end of the tie bar (Figure 3).
- Removing excess epoxy and finishing it flush with the pavement.
- Sealing of the crack (Figure 4).

The objective of crack stitching is to prevent widening of the cracks and to assist in providing load transfer across the cracks. Narrow cracks maintain aggregate interlock, reduce the potential for stepping (or faulting), and are easier to seal. Good candidates for crack stitching are pavements in good condition with low severity longitudinal cracks.



**Figure 2. Drilling holes for cross stitching.**



**Figure 3. Stitched longitudinal crack.**



**Figure 4. Completed crack stitch.**

407 ETR has completed cross stitching for over 350 cracks over the past 10 years. To date, the installation and performance has been excellent. The estimated service life for crack stitching is expected to be 10 or more years. Cracks stitched in 1998 still show excellent performance. The typical cost for crack stitching is in the order of US \$60 per bar (2008 dollars). This is substantially less than the cost and user inconvenience if the concrete slabs were removed and replaced. Crack stitching is a cost-effective tool in extending the service life of a concrete pavement while minimizing the impact on traffic. A key lesson learned with respect to crack stitching is the early identification of cracking and immediate stitching of the crack before any secondary cracking occurs. If secondary cracking has occurred, the effectiveness of the treatment is reduced (Figure 5). While crack stitching has been used successfully for some transverse cracks, it is best suited to longitudinal or diagonal cracks.

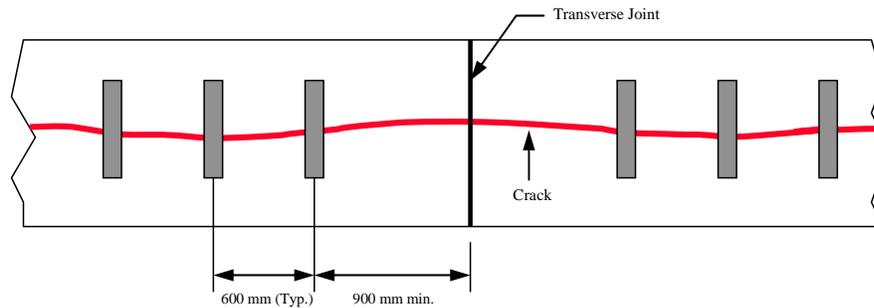


**Figure 5. Cracking at a stitch that was completed too late.**

### **Transverse Joint/Crack Load Transfer Retrofit**

Load transfer is the distribution of wheel loads across transverse joints in jointed concrete pavements. The distribution of loads across a joint (or crack) can be addressed in two ways; through aggregate interlock or through the use of mechanical devices. Poor load transfer can lead to a number of pavement deficiencies including faulting, pumping, and corner breaks. The occurrence of these distresses often leads to a reduced pavement service life. Load transfer retrofit is a procedure used to restore the load transfer efficiency of joints / cracks, which in turn improves pavement performance and ride quality.

The purpose of load transfer restoration (also called dowel bar retrofit) is to insert dowel bars across transverse cracks (Figure 6) or insert additional dowel bars across the transverse joints of jointed portland cement concrete (PCC) pavements. The objective is to increase load transfer and reduce potential for further progression of faulting (stepping of slabs), pumping (repeated deflections at transverse joints that can erode slab support), and slab cracking.



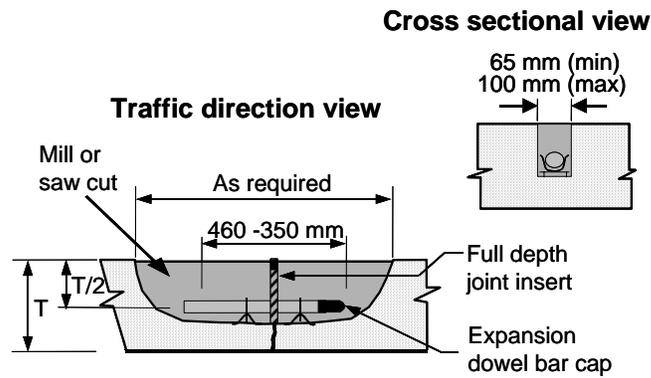
**Figure 6. Typical spacing of dowels across a transverse crack.**

Load transfer restoration is suitable for pavements with deflection load transfer of 60 percent or less in cool weather that show early signs of faulting (more than 2 mm [0.08 in.] but less than 6 mm [0.24 in.]). The pavement should have adequate PCC slab thickness. To ensure proper selection of transverse joints that would benefit from load transfer restoration, evaluation of the load transfer efficiency should be carried out using falling weight deflectometer (FWD) testing. The basic design and construction steps of load transfer restoration method include the following:

- Cutting slots perpendicular to the transverse crack / joint. The slots must be large enough to place the dowel at middepth of the slab and allow for the backfill grout to flow below and around the dowel.
- Removal of the concrete within the slot using light hammers. The slot must be properly cleaned by sand blasting followed by air blasting.
- Installation of the load transfer devices, which are typically smooth, epoxy-coated dowel bars. The size of the dowel bars depends on the slab thickness and anticipated loads. For the concrete pavement on 407 ETR, which is 280 mm (11 in.) thick, a dowel bar 32 mm (1.3 in.) in diameter and 450 mm (18 in.) long is used (Figure 7). One half of the dowel bar is coated with a bond-breaking compound (grease-based) and equipped with an expansion cap. A spacer is inserted in the middle to preserve transverse crack opening.
- Backfill of the slots using materials that develop adequate early strength gain to facilitate early opening of the area to traffic and exhibit little or no shrinkage. Polymer concretes and high-early-strength PCC materials have been used in most installations to date.

Load transfer retrofit techniques are very rapid to complete, require minimal disruption to traffic, and can cost-effectively extend the life of a concrete pavement. A completed load transfer retrofit treatment for a mid-panel transverse crack is shown in Figure 8.

To date, 407 ETR has completed over 100 load transfer retrofit installations primarily at mid-slab transverse cracks. The performance to date has been excellent and a life expectancy of at least 8 to 10 years is anticipated. The cost to install eight bars per lane is in the order of US \$1,300 (2008 dollars).



**Figure 7. Dowel bar placement for load transfer retrofit.**



**Figure 8. Load transfer retrofit for a mid-panel transverse crack.**

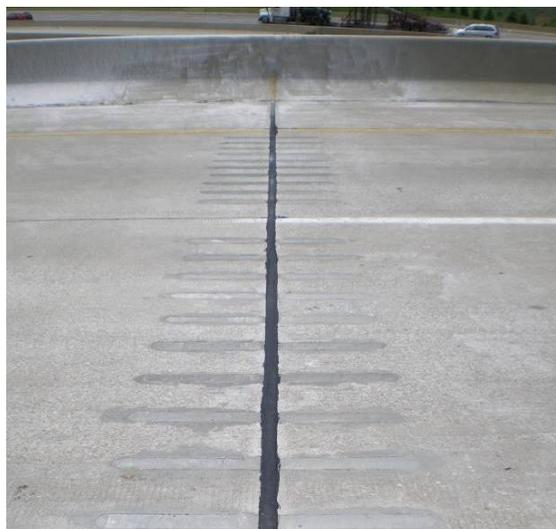
### **Expansion Joint Retrofit**

Expansion joint retrofit is an extension of the load transfer retrofit outlined above. Bridge inspections completed in 2007 indicated that there were some problems with cracking of ballast walls on some of the bridges. A detailed visual inspection indicated that there were some shear cracks in the corners of the ballast walls where the reinforcing steel forms the cleat with the wing wall.

Typically, the bridges and approach slabs are constructed in advance of the pavement. The pavement is then slipformed to within four or five slabs from the approach slab depending on jointing details and bridge skew. The remaining concrete pavement is then formed and placed with an expansion joint at the tie-in to the concrete pavement. The expansion joint typically includes compressible expansion joint material (fibre board) covered with hot-poured rubberized crack sealant. In some cases, the expansion joint is insufficient to accommodate the expansion of the concrete pavement.

In the late spring/early summer of 2008, expansion joint retrofits were installed at several bridge structures. The expansion joint retrofit is similar to the load transfer retrofit for transverse cracks except, the existing dowel bars at the joint are cut using a diamond saw. A sec-

ond cut is then completed to widen the joint to a width of between 40 and 50 mm (1.6 and 2.0 in.). Slots are cut for new dowel bars in between the existing now cut dowel bars. A fibre expansion board is then placed in the joint to the bottom of the slot cuts and dowel placed in the slots. Dowel bars with expansion caps on one end are then placed within the slots with a plastic grout retention shield on either side of the bar at each slot to ensure that the grout is in the slots and does not flow into the joint. The grout is then placed in the slots to the top of the pavement, followed by additional expansion board, which is then topped with hot-poured rubberized asphalt sealant. A photograph of a typical installation is shown in Figure 9. Shortly after the expansion joint installation, the sealant was noted to be squeezing out of the joint, indicating that pressure is being relieved at the joint and the installation is working as designed.



**Figure 9. Expansion joint retrofit at bridge approach.**

In 2008, 407 ETR installed these expansion joint retrofits at six bridge locations. It is hoped that they will protect the bridge ballast walls from further damage and have a service life of at least 15 to 20 years. The cost to install each expansion joint is in the order of US \$ 4,200 per lane (2008 dollars). Additional installations are planned for the spring of 2009. However, prior to completing the new installations, each of the existing retrofit expansion joints will be scanned using an MIT Scan (magnetic imaging tomography) device to evaluate the alignment of the dowel bars in the slots. It is critical that they be aligned properly to ensure that there is free movement across the slabs at the joint.

To maximize the efficiency of the expansion joint installation, it is critical that the joints be installed during relatively cool weather conditions ( $< 20\text{ }^{\circ}\text{C}$  [ $< 68\text{ }^{\circ}\text{F}$ ]). At higher temperatures, the stress due to the expansion of the concrete makes it very difficult to cut the slots and could result in the saw blade binding within the cut. The cut should be completed using several passes to ensure that any built-up stress in the concrete is gradually relieved. It is also important to maintain parallel cuts to ensure a consistent joint width. As shown in Figure 9, concrete barrier walls and concrete shoulders are present at the approach to many of the bridges. To facilitate free movement of the pavement, it may also be necessary to cut the concrete barrier wall. Finally, as many of the bridges have skewed approaches, it was decided to not install any expansion joints in skewed slabs but rather to move back to the first slabs that allow for the installation of the expansion joint perpendicular to the direction of

the pavement. It was felt that this would assist in accommodating any expansion while facilitating easier joint installation.

### **Texturization**

Texturization techniques include conventional milling, fine milling, micromilling, diamond grinding, longitudinal and transverse grooving, precision milling, surface abrading and other techniques that remove unevenness from the pavement surface or improve its texture, and leave an abraded surface that is used as a driving surface. While milling techniques have been used for concrete pavements in some jurisdictions, they tend to be very aggressive causing joint damage as shown in Figure 10. As such, 407 ETR has limited its use of texturization techniques to diamond grinding and grooving and surface abrading.



**Figure 10. Difference between micro-milling (left) and diamond grinding (right).**

Diamond grinding of PCC pavements is a rehabilitation technique that removes a shallow depth of pavement surface material by sawcutting closely spaced grooves into the pavement surface using diamond-tipped blades. The purpose of diamond grinding is to improve pavement smoothness and improve pavement surface friction. When used to improve pavement smoothness, diamond grinding can be used only for selected areas of the pavement, for example to remove slab stepping (faulting) at selected transverse joints. When used to improve pavement surface friction, diamond grinding is used over the entire pavement area. Diamond grinding can remove up to 20 mm (0.8 in.) from the pavement surface and can remove surface defects and irregularities such as polished or scaling surface and faulting, and restore pavement surface smoothness.

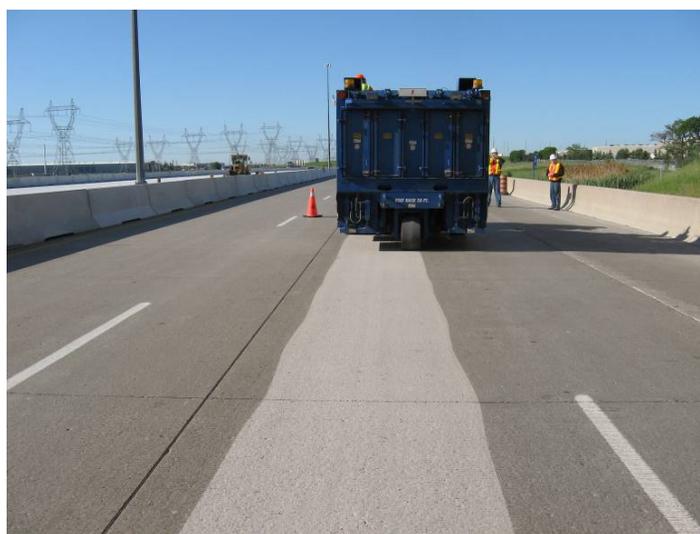
Grooving of the pavement is not intended to improve pavement smoothness but rather to provide channels for the removal of surface water thus potentially improving the frictional properties of the pavement. Longitudinal grooving is shown in Figure 11.



**Figure 11. Longitudinal grooving completed perpendicular to the transverse tines.**

Surface abrading can be completed using a number of proprietary techniques such as Blastrac and Skidabrader. An example of Blastrac texturization is shown in Figure 12.

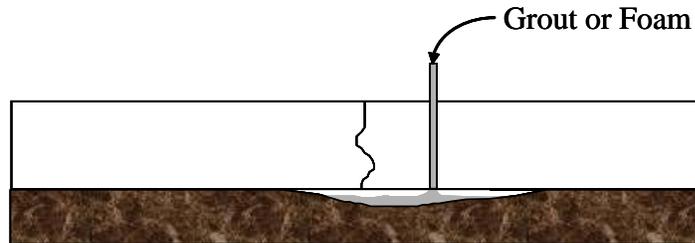
407 ETR has used each of the above techniques on a trial basis since 2002 to monitor their longevity in improving the frictional properties of the pavement. To date, 1,875 m<sup>2</sup> (2,242 yd<sup>2</sup>) of diamond grinding, 158,000 m<sup>2</sup> (188,966 yd<sup>2</sup>) of grooving, 11,625 m<sup>2</sup> (13,903 yd<sup>2</sup>) of Blastrac, and 3,500 m<sup>2</sup> (4,186 yd<sup>2</sup>) of Skidabrader texturization have been completed. Each of the techniques has been very effective in improving the initial frictional properties of the pavement. The various technology test sites continue to be monitored so that in the future, when more extensive action may be needed to address friction, 407 ETR will have a good understanding of the most optimal technique to use to address any pavement frictional issues. The cost of the treatments varies between about US \$ 2 and \$ 8 per m<sup>2</sup> (2008 dollars). Based on the performance of the various texturization techniques to date and the experience reported by others (2), it is expected that texturization will have a service life of some 4 to 6 years.



**Figure 12. Blastrac concrete pavement surface texturization.**

## Under-Slab Sealing and Jacking

Slab stabilization is a rehabilitation technique that fills voids underneath a PCC slab with grout or foam without raising the slabs. Slab jacking fills voids underneath PCC slabs and raises the grade of the slabs. Slab stabilization is also called slab subsealing and under slab grouting (Figure 13).



**Figure 13. Concept of under-slab sealing and jacking.**

The purpose of slab stabilization is to stabilize the pavement slab by pressurized injection of a material such as grout underneath the slab. The objective is to fill existing voids and restore full slab support, particularly at transverse joints and cracks. The main benefit of subsealing is slowing down the erosion of base and subgrade materials caused by excessive pavement deflections through pumping action of traffic.

The purpose of slab jacking is to raise pavement slabs permanently to a desired grade by pressurized injection of grout underneath the slab. At the same time, slab jacking will also stabilize the slab. The objective is to improve rideability (pavement smoothness) and to fill voids underneath the pavement. Slab jacking can raise PCC slabs by up to about 50 mm (2 in.).

407 ETR has used both slab-sealing and slab-jacking techniques. Slab sealing has typically been used where voids are present beneath concrete slabs due to settlement of the underlying base and subgrade materials. Ground-penetrating radar has been used to evaluate the presence and extent of voids. In one case, slab sealing was used to stabilize the subgrade beneath the pavement due to a partial washout caused during a pipe-jacking operation to install a utility service beneath the pavement. Slab jacking has also been used extensively during the widening of the pavement. From 2004 through 2008, much of the existing concrete pavement was widened with two or more lanes installed to the inside of the existing lanes. Prior to the widening, pavement settlement areas were raised using slab-jacking techniques to improve the longitudinal profile of the pavement so that when the widening lanes were installed, they could be matched to the new profile instead of the settled profile. A total of almost 10,000 kg (22,046 lb) of foam were injected beneath the pavement to lift the slabs (Figure 14). The cost for foam injection is in the order of US \$ 15 per kg (6.82 per lb) (2008 dollars), and the service life of the technique would depend on the extent and severity of the initial problem.

While foam injection has been used extensively by 407 ETR, care must be taken in the amount of foam injected and its location. The concrete pavement on Highway 407 has an asphalt cement treated, open-graded drainage layer (OGDL) directly below the concrete. If foam is injected into the OGDL as shown in Figure 14, the permeability of the OGDL will be substantially reduced in the area of foam injection. This was accepted for the slab jacking as it provided the best layer to inject to lift the slabs. Also, it is very important to monitor the lifting of the slabs during the jacking operation. Lifting the slabs by more than about 50 mm (2 in.) can result in cracking of the slabs. In these situations, a load transfer retrofit treatment was used across any of the cracks to assist in stabilizing the slabs.



**Figure 14. Slab jacking using foam.**

### **Full-Depth Concrete Slab Replacement**

Full-depth repair of PCC pavements (Figure 15) is a rehabilitation method that involves the removal of an entire slab, or a substantial portion of the entire slab (full depth), the installation of load transfer devices, and the replacement of the PCC material.

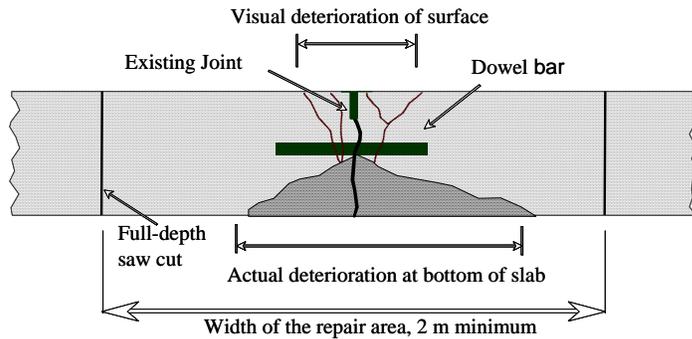
The purpose of full-depth repairs is to repair slabs that can no longer be repaired using other less expensive techniques. This includes slabs with broken concrete, midslab cracking, slabs damaged by frost heaving and subgrade settlement, slabs with poor load transfer, and slabs where dowels are exposed, etc. The objective of the repair is to restore smoothness and structural integrity of the pavement and to arrest further deterioration.

Depending on the requirement to open the area to traffic, PCC repair materials can be a regular PCC paving mix using normal portland cement or “fast-track” early-strength cement. Modified cement mixtures with the addition of accelerating admixtures, polymers, or specialty proprietary cement materials are also used.

Full-depth repairs should be done on the full width of the lane and should have the minimum length of 2.0 m (6.6 ft). The maximum length should be such that at least 2.0 m (6.6 ft) of the original slab remain in place. If the remaining slab is less than 2.0 m (6.6 ft) long, the entire slab should be replaced (figures 16 and 17).

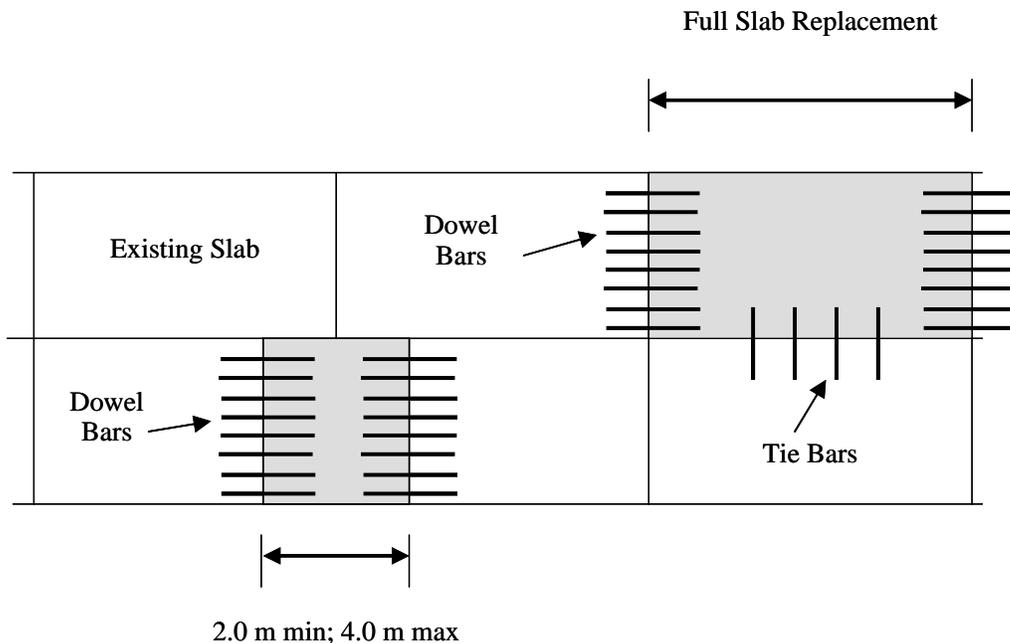


**Figure 15. A 2-m full-depth repair area prepared for concrete placement.**



**Figure 16. Cross section of deteriorated transverse joint.**

Given the relatively young age of the Highway 407 ETR pavement (10 years), full-depth slab repairs have only been necessary on a limited basis. Yu et al. (1) found through their review of the Strategic Highway Research Program C206 test sites, that high-early-strength PCC can provide good long-term performance, however, adverse temperature conditions during installation can cause premature failures. If the difference in the average PCC temperature during curing and overnight low temperatures is large, longitudinal cracking is possible, as the thermal contraction in the transverse direction is restrained by dowel. The results of this evaluation showed that in terms of fatigue damage or faulting performance, the repairs could be opened to traffic at much lower strengths than those typically recommended. However, opening at strengths much less than those recommended is not advisable because of the risk of random failures caused by a single heavy load at early age. Therefore, 407 ETR has completed all full-depth repairs to date during the summer and on weekends when traffic is reduced and conventional curing can be completed for at least 48 hours.



**Figure 17. Example layout for full-depth repairs for a two-lane section.**

To date, about 400 m<sup>2</sup> (478 yd<sup>2</sup>) of full-depth repairs using conventional concrete have been completed on the highway at a cost of about US \$ 375 per m<sup>2</sup>. This translates to a cost of about US \$ 2,800 (2008 dollars) for a 2-m (6.5-ft) repair or about US \$ 5,500 (2008 dollars)

for a typical full slab. The performance of the full-depth repairs to date has been excellent, and a service life of at least 20 years is expected (1).

### **Thin Surface Restoration Techniques**

An overlay of PCC pavements is a rehabilitation technique that may include repairs of structural deficiencies in the existing PCC slabs, application of a bonding agent (tack coat) or a layer intended to mitigate the propagation of reflection cracking, and placement of an overlay. The purpose of an overlay is to improve functional deficiencies of the PCC pavement, such as low pavement surface friction, excessive pavement–tire noise, inadequate cross-slope, and roughness.

407 ETR has utilized thin surface restoration techniques such as microsurfacing and thin specialty overlays (Novachip) on a trial basis to determine their effectiveness. While these techniques are typically used for flexible pavements, experience with their use on concrete pavements is somewhat limited.

Microsurfacing is an unheated mixture of polymer-modified asphalt emulsion, high-quality frictional aggregate, mineral filler, water, and other additives, mixed and spread over the pavement surface as a slurry. The aggregate skeleton used for microsurfacing consists of high-quality interlocking crushed aggregate particles. Consequently, it is possible to place microsurfacing in layers thicker than the largest aggregate size, or in multiple layers, without the risk of permanent deformation (Figure 18).



**Figure 18. Micro-surfacing over a concrete pavement.**

407 ETR has used microsurfacing extensively for flexible pavements to extend the service life of hot-mix asphalt concrete exhibiting early signs of ravelling, in particular on bridge decks. 407 ETR has also used microsurfacing over concrete pavement on a trial basis with 1,125 m<sup>2</sup> (1,436 yd<sup>2</sup>) of microsurfacing placed on a freeway-to-freeway ramp in 2006. Prior to placing the microsurfacing, one lane of the pavement was swept and the other sand-blasted to determine if there would be any difference in the adhesion of the microsurfacing to the pavement. To date, the performance of the microsurfacing has been quite good with no difference in performance between the lanes. However, as can be seen in Figure 18, the microsurfacing has cracked over the underlying joints, which are 12 mm (0.5 in.) wide in the concrete pavement and been lost from the pavement surface. This is likely due to the

relatively high stiffness of the microsurfacing material. These relatively wide joints will need sealing in the future. Although the microsurfacing was only recently placed, a service life of about 5 to 7 years is anticipated. The cost for microsurfacing is in the order of US \$ 4/m<sup>2</sup> (2008 dollars).

Proprietary thin asphalt concrete overlays are typically 15 to 20 mm (0.6 to 0.8 in.) thick and include an open-graded, high-quality aggregate passing the 13.2-mm (0.5-in.) sieve size. The mix is applied by a specialized paver with built-in application of a tack coat. The tack coat assists to ensure that there is a good bond between the concrete pavement and hot-mix overlay. Thin hot-mix overlays such as this can improve pavement friction and provide a quiet pavement surface because of their porosity (Figure 19).

Thin asphalt concrete overlays were placed directly over the exposed concrete pavement for three ramps and 500 m (1,640 ft) of the mainline pavement in 2004. While the thin overlays exhibited reflection cracking at each of the joints, the thin overlay material is more flexible than the microsurfacing and did not exhibit any raveling of the joints. To date, the performance of the thin overlays has been good, although there has been some snowplow damage to the asphalt concrete, particularly at the transition from the exposed concrete to the overlaid pavement. Based on its current performance, a service life of about 8 to 10 years can be expected. The cost for the thin overlay is in the order of US \$ 7 per m<sup>2</sup> (2008 dollars).



**Figure 19. Thin asphalt concrete overlay over a concrete pavement.**

## **SUMMARY AND CONCLUSIONS**

In summary, through the use of a very aggressive pavement preservation program, 407 ETR is able to maintain its exposed concrete pavements at a very high level of service for a relatively modest cost. A variety of pavement preservation techniques such as crack stitching, dowel bar retrofit, expansion joint retrofit, surface texturization, slab sealing and jacking, full-depth slab replacement, micro-surfacing and thin overlays are being used and evaluated for their performance and cost-effectiveness. A summary of the techniques, expected performance and costs is shown in Table 1.

**Table 1**  
**Summary of Pavement Preservation Techniques, Performance and Cost**

Treatment	Expected Benefit (life-span), years	Typical Unit Cost	
		Unit	Cost (US \$)
1. Crack stitching	> 10	Each	70
2. Dowel bar retrofit	8–10	Lane	1,300
3. Expansion joint retrofit	15–20	Lane	4,200
4. Texturization	4–12 <sup>1</sup>	m <sup>2</sup>	2 to 8
5. Slab sealing and jacking	>10 <sup>2</sup>	kg	15
6. Full-depth slab replacement	> 20	m <sup>2</sup>	375
7. Microsurfacing	5–7	m <sup>2</sup>	4
8. Thin hot-mix asphalt overlay	8–10	m <sup>2</sup>	7

1. Based on current performance. Long-term effectiveness still unknown.

2. Depends on the extent and severity of the initial problem.

While there are many techniques available to assist in extending the performance of concrete pavements, their application is not uniform from agency to agency. In other words, what works for one agency may not be as effective for another as the performance of individual techniques will likely depend on local practice, experience, and conditions.

The ideal way to approach pavement preservation is through the life-cycle economic analysis that takes into consideration the initial construction as well as all subsequent maintenance and rehabilitation treatments. The objective is to have a pavement structure that provides the most cost-effective service through the combination of initial construction and subsequent maintenance and rehabilitation treatments.

407 ETR has taken a proactive approach in using and evaluating treatments for more widespread use including the installation of trial sections and engineering monitoring of their performance. With continued experience, decision trees and other tools for selecting the optimum pavement preservation treatments will be developed and implemented. Although cost must be considered, it is not always the decisive factor in the treatment selection process, which is particularly the case for toll facilities where disruptions to traffic can impact revenue.

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# Evaluation and Decision Strategies for the Routine Maintenance of Concrete Pavement

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## ABSTRACT

This paper is to provide assistance for the pavement evaluation and selection of method of repair for routine maintenance relative to the extension of service life. The visual identification of various distress types is discussed, and evaluation techniques using nondestructive testing are introduced that are key to determining proper routine maintenance activities. According to the areas selected from the simplified checklist of visual distress types, ground penetration radar for detecting voids below the slab and the presence of trapped water, falling weight deflectometer for structural condition evaluation, and dynamic cone penetrometer for estimating the in situ strength of base and subgrade soils are used to provide current information on pavement condition for selection of needed repair methods using a simple, systematic decision process. During field investigations, poorly performing areas were identified and possible fixes determined as a means of guideline development. Key routine maintenances activities are categorized in five levels; performance monitoring, preservative, functional concrete pavement repair (CPR), structural CPR, and remove and replace. Each level of maintenance is arranged for the use of repair treatments in a consistent, logical framework to ensure their effective and timely use and employment. Since the decision process is focused on monitoring the early stages of deterioration, it should result in more cost effective maintenance programs.

## INTRODUCTION

Concrete pavement has performed well in urban area and Interstate highway settings for many years because of its inherent stiffness and capability for providing long service life. However, rapidly increasing heavy traffic accelerates pavement deterioration and increases the need for more maintenance than in the past. If proper maintenance is not employed at low levels of deterioration in a timely manner, acute degradation of pavement serviceability will occur and major repair costs may be incurred; therefore, preservative or minor concrete pavement repair (CPR) should be executed strategically at early stages of pavement deterioration to extend pavement life at lower cost.

Figure 1 shows the concept of pavement condition degradation with pavement age. Different stages of maintenance are noted as a means to address specific pavement conditions. Performance monitoring (prior to the pavement condition deteriorating to the level where structural CPR is needed) is extremely important since preservative

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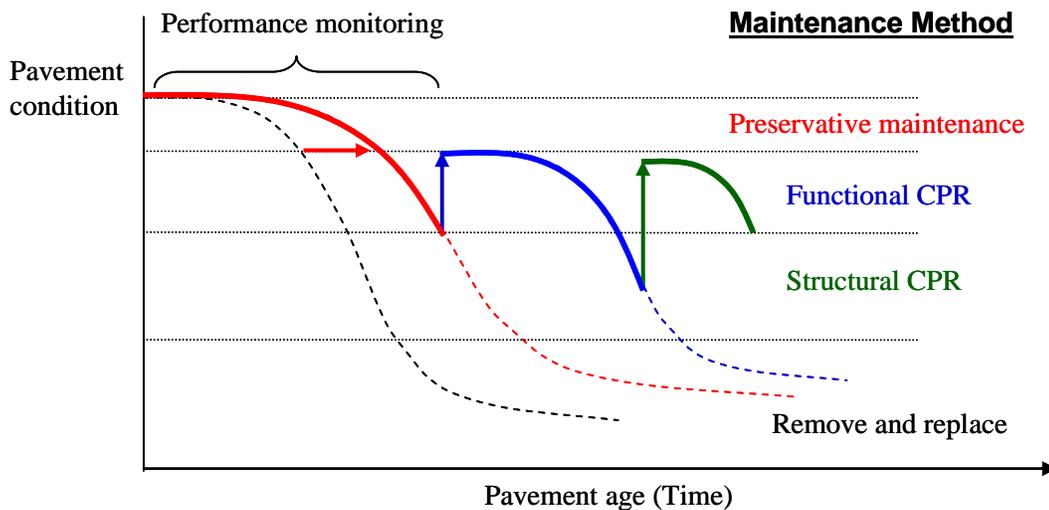
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maintenance is effective in slowing down the rate of degradation and extending pavement life.

Key routine maintenance activities are categorized into five levels:

- Performance monitoring.
- Preservative maintenance.
- Functional CPR.
- Structural CPR.
- Remove and replace.



**Figure 1. Pavement condition and maintenance stages.**

Table 1 outlines a strategic overview of routine maintenance activities in terms of pavement condition, assessment, and recommendations for repairs, and Table 2 summarizes the comparison of selected routine maintenance treatments in terms of repair cost, life extension, and working time. All cost and life extension numbers are averages and may vary from those listed in the table (1).

**Table 1**  
**Routine Maintenance Strategy Guidelines**

<b>Level of Routine Maintenance</b>	<b>Type of Activity</b>	<b>Type of Condition</b>	<b>Quantifiable Condition Factors</b>	<b>Repair Type and Notes</b>
Performance monitoring	Distress Survey	Pavement age; Portland cement concrete (PCC) >10 years; asphalt concrete overlay (ACOL) > 2 years	Pavement age	Monitor age for pavements more than 10 years old PCC or ACOL pavements more than 2 years old.
	Distress and FWD Survey	Pavement deflection data > 3 years	Recent FWD data	Conduct FWD testing based on visual survey results.
	FWD and GPR Survey; DCP Testing	Pumping with or without staining; missing joint seal material; edge drop-off; shoulder separation	Pumping; joint seals condition; surface dielectric constant (DC) of GPR; penetration ratio (PR) of DCP	Conduct selected FWD and DCP testing based on visual and GPR survey results. PR > 2-in./drop indicates soft subgrade materials, soil modulus < 6,000 lbf/in <sup>2</sup> . GPR is useful to detect subsurface moisture and voided areas. DC > 9 indicate presence of subsurface water.
Preservative	Crack sealing	Working cracks	Crack width > 0.03 in.	Crack sealing for working crack in continuously reinforced concrete (CRC) pavement.
	Reseal joints and cracks	Visible sealant damage on transverse and longitudinal joints and sealed cracks	Sealant age; visible sealant damage—cracking and debonding	Keep joint well width < 1 in.; widened joint wells may be noisy. Trapped subsurface water should be removed before re-sealing operations.
	Transverse grade re-profiling	Trapped surface water in depressed areas	Trapped surface water in depressed areas	Depressed area degrades riding quality and cause impact loading. Trapped surface water can cause safety problem.
	Retrofit edge drains	Standing water; Trapped surface water; Saturated base layer and subgrade	Presence of standing water; Slab staining; Surface DC; Subgrade strength	Edge drain is not recommended if the base is unstabilized, the base contains > 15 percent fines, or the pavement structure is undrainable.
Functional CPR	Partial-depth repair	Spalled joint/crack; deep delamination in CRC pavement	Density, width, and depth of spalling (>2 in.)	Spalling depth should be less than 1/3 the thickness of the slab and no reinforcing steel exposure; deep delamination with no other distress and steel is not corroded.
	Diamond grinding	Rough and noisy patches; faulting; bump	Density of patching; depth of faulting	Restore load transfer before grinding if structurally defected.
	Thin ACOL	Rough and noisy patches; faulting; hard aggregate; settlement	Density of patching; depth of faulting; aggregate type	Employ for hard aggregate pavements. Restore load transfer before the overlay if structurally defected. Use crack attenuating mix and good aggregate.
Structural CPR	Restore load transfer	High deflection; low load transfer efficiency (LTE); reflection crack in ACOL	Faulting; deflection; LTE; crack width and density of spalling in ACOL	Dowel bar retrofit. Check the deflection basin area and LTE of joint/crack. Employ RLT when 2- in.-wide spalled joint in ACOL > 20 percent.
	Cross stitching	Longitudinal crack; separated shoulder joint; low LTE	Width of the crack or shoulder joint separation; lane to shoulder LTE; pumping	Joint seal only when shoulder joint separation < 1/2 in. Cross stitching and joint seal when shoulder joint separation is between 0.5 in. and 1 in. Remove and replace shoulder when joint separation > 1 in. Slab undersealing where pumping and void detected.
	Slab undersealing	Water-filled voids at or under joints; settlement	Presence of voids; slab staining	GPR is recommended to locate holes in a way that will ensure good grout distribution and void filling.
Remove and replace	Full-depth repair	Corner break; shattered slabs; punchouts; broken cluster area	Severity and number of cracks; spalling; faulting	Soft subgrade materials may require removal. Full-depth repair for broken cluster should be extended to one-half of crack spacing between next cracks.

**Table 2**  
**Summary of Routine Maintenance Repairs (1)**

Repair Stage	Repair Type	Object	Limitations	Unit Repair Cost (\$/ft <sup>2</sup> )	Expected Life Extension	Typical Repair Work Time	Recommendations
Preservative	Reseal joints and cracks	Reduce infiltration of moisture and incompressive material. Reduce pumping and faulting.	Questionable for long-term effectiveness	\$0.75–\$1.25/ft (hot pour), \$1.00–\$2.00/ft (silicon)	3–8 years	5,000 ft/day (hot pour)	Select proper sealing material based on temperature and moisture conditions.
	Retrofit edge drains	Provide drainage of surface water. Reduce pumping, faulting, and other moisture damage.	May accelerate deterioration if not maintained well, not recommended if no base or base contains excessive amount of fines (>15 percent passing No. 200 sieve)	\$2.00–\$4.00/ft	Life of existing pavement	1 mi/day	Proper design, construction, and maintenance are essential.
Functional CPR	Partial-depth repair	Repair spall and distress without removing entire slab.	Full-depth repair is needed if the damage extends below 1/3 the slab thickness.	\$325–\$500/yd <sup>3</sup>	3–10 years	4–12 repairs/hr, curing time not included	Partial-depth repairs should restore the joint face and joint should be sealed properly.
	Diamond grinding	Provide smooth riding surface with good texture. Reduce noise.	Roughness will return if underlying causes not addressed.	\$1.80–\$7.80/yd <sup>2</sup>	8–12 years	2,500 yd <sup>2</sup> /day	Grinding should not be employed on pavements with material problems.
	Thin ACOL	Restore functional capacity such as rideability but increase structural capacity insignificantly.	Susceptible to reflection cracking	\$1.45–\$3.25/yd <sup>2</sup> -in	5–15 years	2–4 mi/day/lane	Existing structural distresses must be repaired to avoid premature failure.
Structural CPR	Restore load transfer	Restore load transfer to reduce faulting, pumping, and crack/joint deterioration.	Pavements exhibiting material related distresses such as D-cracking or reactive aggregate are not good for dowel bar retrofitting.	\$25–\$35/dowel	10–15 years	150 joints/day	Diamond grinding is needed to remove existing faulting. Slab stabilization is needed to address loss of support.
	Cross stitching	Hold longitudinal crack or joint together and prevent opening of crack or joint.	Applicable for fair condition and may not prevent secondary cracking or crack propagation.	\$9–\$10/bar	10–15 years	1,500 ft/day	Rehabilitation is required when secondary cracks develop.
	Slab undersealing	Restore uniform support by filling void and reduce corner deflection, pumping, and faulting.	Difficult to identify poorly supported area, restrictions on climatic condition, and can increase damage if slab is lifted.	\$1.30–\$1.40/yd <sup>2</sup>	3–6 years	100 slabs/day	Experienced contractor and proper inspection are essential.
Remove and replace	Full-depth repair	Remove all deterioration in the distress area. Restore load transfer at joints and cracks.	Additional joints introduced by full-depth repairs may add to the pavement roughness.	\$90–\$100/yd <sup>2</sup>	5–15 years	4 to 6 repairs/hr, curing time not included	If deterioration is widespread over the entire project length, an overlay or reconstruction may be more cost effective.

## PAVEMENT CONDITION EVALUATION TECHNIQUES

Pavement condition evaluation is the key to determining proper routine maintenance activities. It is needed to validate the extent of distress-related damage, quality of drainage, and relative base/subgrade layer strength. Pavement distress condition is considered relative to functional and structural performance in the decision process. The following evaluation techniques are recommended for strategic routine maintenance decisions (2): visual survey, ground penetration radar (GPR), falling weight deflectometer (FWD), and dynamic cone penetrometer (DCP).

### Visual Survey

Selected project sites can be scanned to identify distressed areas to select locations for further inspection. There are many well-organized visual pavement condition survey protocols used by highway agencies to monitor and record pavement distresses. However, current survey protocols often require a level of inspection detail greater than what is normally needed for a routine maintenance survey; therefore, a simplified survey list is provided in Table 3 to assist in the collection of routine maintenance information to meet critical decision criteria.

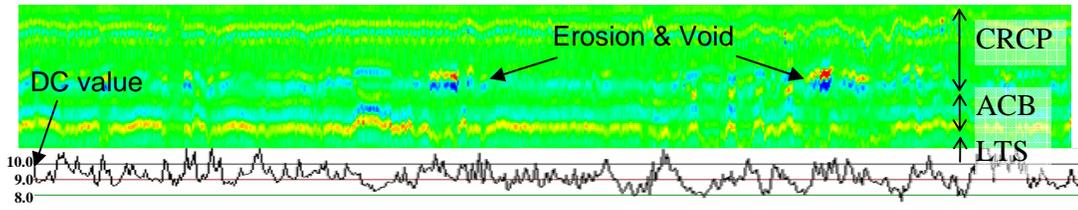
**Table 3**  
**Pavement Condition Survey Form for Routine Maintenance**

No.	Check list	Further Inspection (Circle all that apply)
1	Pavement age (yr.) and aggregate type (hard or soft)	
2	Year of recent pavement distress survey (yr.)	
3	Year of recent pavement deflection survey (yr.)	
4	Joint sealant age (yr.)	
5	Sealant damage of transverse joint or crack (%)	
6	Sealant damage of longitudinal joint or crack (%)	
7	Sealant damage of sealed crack (%)	
8	Trapped surface water in depressed area	
9	Standing water or slab staining	GPR, DCP
10	Pumping with or without staining	GPR, DCP
11	Bump (stable or unstable, depth, in.)	GPR, DCP
12	Settlement (stable or unstable, depth, in.)	GPR, DCP
13	Joint spall (width, depth, % of joint spall > 2 in.)	FWD
14	Crack spall (width, depth, % of crack spall > 2 in.)	FWD
15	Deep spall (depth, in.)	FWD, GPR, DCP
16	Patching (number/mi)	FWD, GPR, DCP
17	Faulting (depth, in.)	FWD, GPR, DCP
18	Transverse crack (width, number/slab)	FWD, GPR, DCP
19	Longitudinal crack (width, number/slab)	FWD, GPR, DCP
20	Shoulder separation (width, in.)	FWD, GPR, DCP
21	Corner break (spall width, fault depth, % of slab)	FWD, GPR, DCP
22	Deep delamination (depth, in.)	Steel corrosion
23	Punchout (spall width, fault depth, % of slab)	FWD, GPR, DCP
24	Reflection crack in ACOL (spall width, fault depth, number/mile)	FWD, GPR, DCP

FWD = falling-weight deflectometer; GRP = ground-penetrating radar; DCP = dynamic cone penetrometer

## GPR Testing

GPR testing is a fast and effective test method to determine base conditions such as voids and the presence of water trapped in and between underlying pavement layers. Moreover, GPR survey can be used for PCC pavement layer thickness estimation, layer interface condition assessment, and dowel misalignment evaluation. In pumping areas, dowel locations, voids, and subsurface water under the slab could be detected using an air-coupled system vehicle or ground coupled, as shown in Figure 2. Although no standard procedures have been documented for detection of voids under the concrete slab using GPR, image analysis or dielectric constant (DC) analysis could be used to detect void and subsurface moisture for the routine maintenance purpose.



**Figure 2. Example of the GPR testing image.**

### *Image Analysis*

Detection of voids under the concrete slabs may require determination by trained personnel, but generally the following can help to analyze GPR images. In the color image, blue strips represent voids and red strips represent moisture, while in the gray scale image, black strips represent voids and white strips represent moisture. Intervallic blue dots (black in the gray scale image) indicate dowel locations.

### *DC Analysis*

The DC value of GPR is shown as blue line below the layer image in Figure 2. It can be used to detect subsurface moisture. DC values range from 1 (air) to 81 (distilled water), and generally DC of aggregate base is around 6 to 7. In the pavement system, DC is an efficient indicator of the presence of subsurface water if the DC of base or subgrade is higher than 9.

## FWD Testing

Load transfer efficiency (LTE) and deflection testing can be used as simple means of determining routine maintenance needs. Deflection test using FWD can evaluate the structural condition of pavement such as layer stiffness, LTE, and loss of support below the slab. Therefore, the areas selected from the checklist of visual survey items needs to be evaluated relative to structural capacity for such stiffening measures as load transfer retrofitting. Highly spalled or faulted joints and cracks should be tested to evaluate LTE and continuity of support. Moreover, deflection and basin area determinations at the center of slab should be carried out occasionally as a reference of good support conditions.

### *LTE Testing*

LTE testing is recommended to check the structural capacity of joints or cracks. Deflections on loaded and unloaded side of a joint or crack are measured, and used to determine the LTE as follows:

$$LTE = \frac{d_U}{d_L} \times 100 \quad (1)$$

Where, LTE = Load transfer effectiveness, percent

$d_U$  = Deflection on the unloaded side of the joint or crack, mils

$d_L$  = Deflection at the loaded side of the joint or crack, mils

It is recommended that testing be completed when the ambient air temperature is above 80 °F (27 °C), and below 60 °F (16 °C). Above 80 °F (27 °C), the LTE is generally over 90 percent for temperature expanded concrete pavement; load transfer retrofitting could be considered if it is determined that the joint or crack LTE is lower than 70 percent a substantial amount of time which can be more easily detected at temperatures below 60 °F (16 °C).

### *Deflection Testing*

Deflection basin area is a simple means to detect possible deteriorated areas. The locations which show low deflection basin areas could be interpreted as problematic as the same meaning of a low LTE. The typical range of basin area for rigid pavements is between 24 and 33 in., and load transfer retrofitting may be recommended when basin area is lower than 25 in. Deflection basin area can be calculated as follows (3):

$$\text{Basin area} = \frac{6(d_0 + 2d_1 + 2d_2 + d_3)}{d_0} \quad (2)$$

Where, Basin area = FWD deflection parameter, in.

$d_0, d_2, d_3, d_4$  = Deflection at 0, 1, 2, and 3 ft from the loading position, mils

### **DCP Testing**

DCP testing indicates the in situ strength of base and subgrade soils. The test provides a correlation between the strength of the soil and its resistance to penetration. It is a fast and easy test method and can be used to make a rough estimate the elastic modulus of each layer and sublayer for evaluation purposes. Conduct DCP testing on selected areas where visual and GPR surveys indicate the evidence of pumping or subsurface water. Equation 3 shows the relationship between the penetration ratio and elastic modulus of soils (4).

$$E = 2550 \times CBR^{0.64} \quad (3)$$
$$CBR = 292 / PR^{1.12}$$

Where, E = Elastic modulus, lbf/in<sup>2</sup>

CBR = California bearing ratio

PR = Penetration ratio, mm/blow

A plot of the DCP data is useful to find the slope of the linear trendline. Typical flexible base modulus is 60,000 to 80,000 lbf/in<sup>2</sup> (413.69 to 551.58 MPa ) or PR is 1 to 2 mm/blow (0.05 to 0.1 in./blow). The PR value higher than 2 in./blow indicates very soft subgrade materials which implies the soil modulus < 6,000 lbf/in<sup>2</sup>(41.37 MPa).

## REPAIR DECISION FLOWCHART FOR ROUTINE MAINTENANCE

Based on the pavement condition evaluation, the following decision flowchart provides guidance for effective routine maintenance. The decision flowchart is self-explanatory and provides guidance for effective routine maintenance.

### Performance Monitoring

Performance monitoring is basic and is the most important step to achieve effective routine maintenance for extending pavement service life. General pavement information needs to be assembled. Pavement age, aggregate type, traffic conditions, weather, and usual construction issues help decision makers understand the current pavement distress. In Figure 3, if pavement age is more than 10 years for PCC or 2 years for AC overlaid pavement, the pavement condition survey is recommended to be updated. Based on visual survey results, distressed areas may need further testing using pavement condition evaluation techniques such as FWD, GPR, and DCP to evaluate the structural condition. When pumping evidence is monitored, GPR surveying is especially useful to detect subsurface moisture and voided areas. FWD and DCP testing is useful to score the pavement's structural condition for selecting proper maintenance repairs.

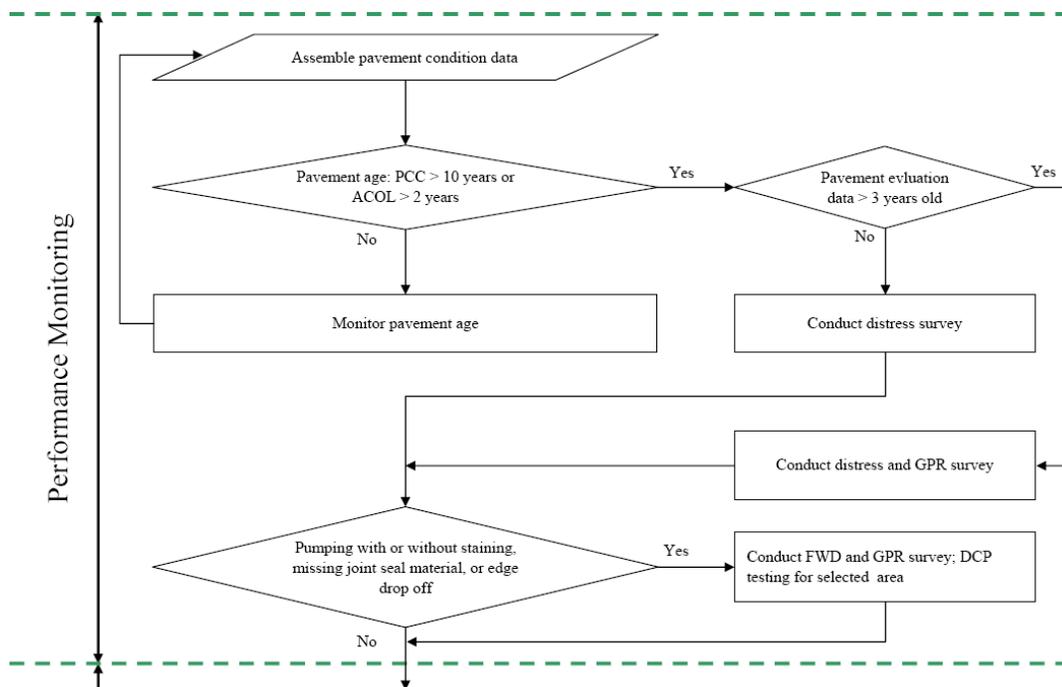


Figure 3. Routine Maintenance Decision Flowchart – Performance Monitoring.

### Preservative Maintenance

Preservative maintenance is focused on providing minor treatment to minimize potential moisture damage, which is one of the most significant causes of deterioration of a concrete pavement system. The condition of the joint and crack seals and overall pavement drainage are key factors to prevent moisture damage due to water infiltration, and decision making is guided as in Figure 4.

## Reseal Joints and Cracks

Crack and joint sealing condition is the first thing to check in a visual survey. Crack sealing is recommended when crack width is wider than 0.03 in. (0.8 mm) or reflection cracking occurs on asphalt concrete overlay (ACOL) to prevent infiltration of water and incompressible materials. Resealing joints and cracks is recommended when sealants are damaged more than 20 percent along the joint or crack to reduce infiltration of moisture and incompressible material over time.

This treatment can reduce pumping and faulting potential. Selection of proper sealing material should be based on temperature and moisture conditions; trapped subsurface water should be removed before re-sealing operations. Moreover, keep the joint well width smaller than 1 in. (25 mm) since widened joint wells may be noisy and degrade rideability (5).

## Transverse grade re-profiling

If depressed or uneven areas are present along the longitudinal pavement edge, transverse grade re-profiling should be considered to enhance the functional condition or reduce blocked drainage of the surface water. Standing surface water could be a source of infiltration that may eventually lead to further settlement, and depressed areas induce degrading of riding quality, impact loading, and create safety issues.

## Retrofit Edge Drains

If geometry and circumstances facilitate drainage, retrofitting the edge drain is recommended when water under the slab is identified or when the DC value is higher than 9 and the subgrade PR is smaller than 2 in. (51 mm) per blow. However, edge drainage is not recommended if the base is unstabilized and contains more than 15 percent fines (passing No. 200 sieve) or if the pavement structure is undrainable. Proper design, construction, and maintenance are essential because edge drainage may accelerate deterioration of the base and subgrade if not well-maintained (6).

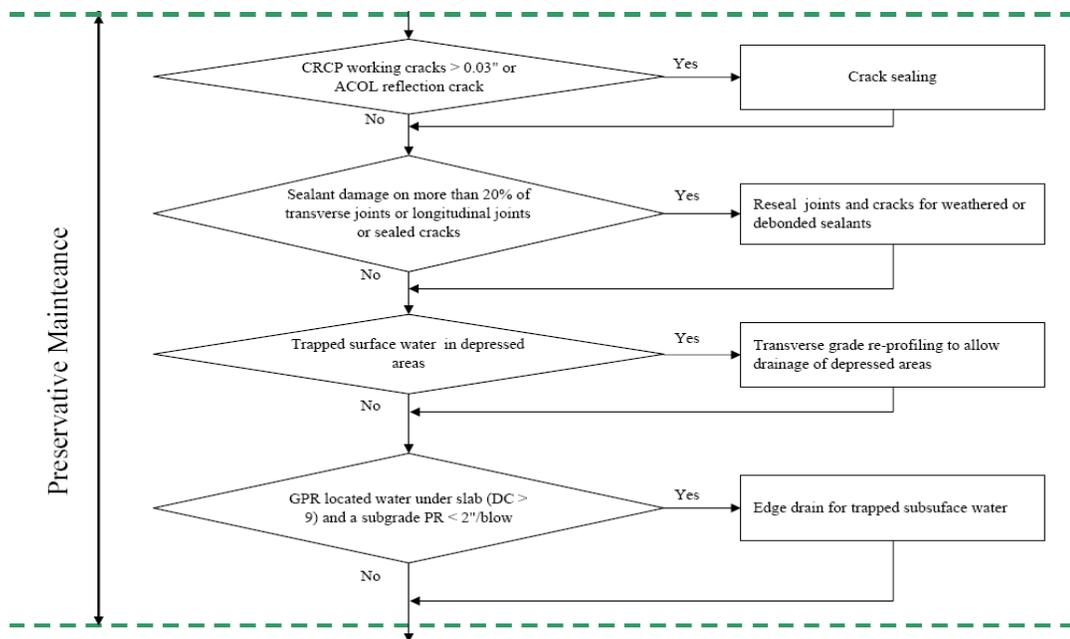


Figure 4. Routine Maintenance Decision Flowchart – Preservative Maintenance.

## Partial-Depth Repair

The objective of partial-depth repair is to repair spall and distress without removing the entire slab. When spalls 2 in. (51 mm) wide are more than 10 percent of the crack or joint, partial-depth repair should be employed using proper patching material for PCC pavement and AC-overlaid PCC pavement. The depth of spall should be less than one-third the thickness of the slab, and the pavement should have no reinforcing steel exposure. Partial-depth repair also can be applied for deep delamination up to half of the CRC pavement thickness if the remaining slab is strong with no other distress and the steel is not corroded. Partial-depth repairs should restore the joint face, and the joint should be sealed properly (6, 7).

## Diamond Grinding

Rough and noisy patches, faulting, and bumps can be eliminated cost-effectively using diamond grinding. When patches are more than 10 per mile and faulting is more than 0.25 in. (6 mm), diamond grinding provides a smooth riding surface with good texture and reduces noise. When stabilized bumps or settled areas are present, diamond grinding can also be effective. However, roughness will return if underlying causes are not addressed; therefore, restoration of load transfer before grinding is recommended. Grinding should not be used for pavements with material problems or if the aggregate type is too hard to grind economically (6).

## Thin ACOL

A thin AC overlay with a paving fabric can be used to restore the functional capacity of a pavement and improve rideability. Employing a thin AC overlay for hard aggregate pavements may be a good alternative to diamond grinding. Existing structural distresses must be repaired and restored before the overlay is placed. This is important particularly if the pavement is structurally deficient to avoid premature failure. Use of a crack-attenuating mix with good aggregate is recommended to minimize reflection cracking.

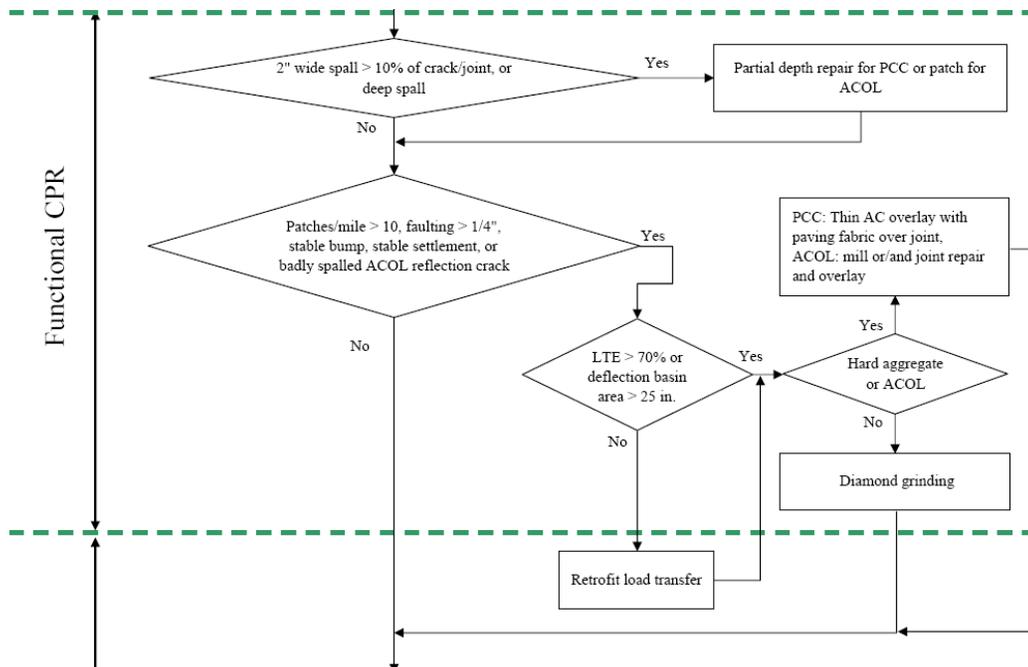


Figure 5. Routine Maintenance Decision Flowchart – Functional CPR.

## **Structural CPR**

The objective of structural CPR is to eliminate the cause of structural distresses and retrofit structural joint capacity to extend pavement service life. Structural CPR at the optimal time should increase pavement life. Generally, structural CPR needs functional CPR at the same time to achieve adequate results. Figure 6 shows guideline for structural CPR.

## **Retrofit Load Transfer**

Retrofit load transfer should be considered when faulting, high deflections, low LTE of the joint/crack, or reflection cracks in the ACOI are detected. When LTE is lower than 70 percent, the basin area is less than 25 in. (635 mm), and joints are spalled more than 2 in. (51 mm) wide over more than 20 percent, then restoration of load transfer is recommended to address faulting, pumping, and crack or joint deterioration. Pavements exhibiting material-related distresses such as D-cracking or reactive aggregate are not candidates for retrofit load transfer. Before and after restoring load transfer, slab stabilization may be needed to address loss of support and diamond grinding needed to remove the existing faulting (6).

## **Cross Stitching**

Cross stitching holds the longitudinal crack or joint together and prevents opening and closing over time. Cross stitching should be considered when longitudinal cracks or shoulder joints separate wider than 0.5 in. (13 mm). In the case of shoulder joint separation, cross stitching and joint sealing are recommended when shoulder joint separation is between 0.5 in. (13 mm) and 1 in. (25 mm) wide. For joint sealing to be effective, the shoulder joint separation should remain smaller than 0.5 in. (13 mm) (8).

## **Slab Undersealing**

Slab undersealing is used to restore uniform support by filling voids and reducing corner deflection, pumping, and faulting. Experienced contractors and proper inspection are essential to properly identify and underseal damaged areas. Therefore, GPR is recommended to locate holes in a way that will ensure good grout distribution and void filling. Slab undersealing is recommended when GPR-indicated voided cracks or joints are more than 20 percent of the inspected section or where unstable bumps or unstable settlement is present (6).

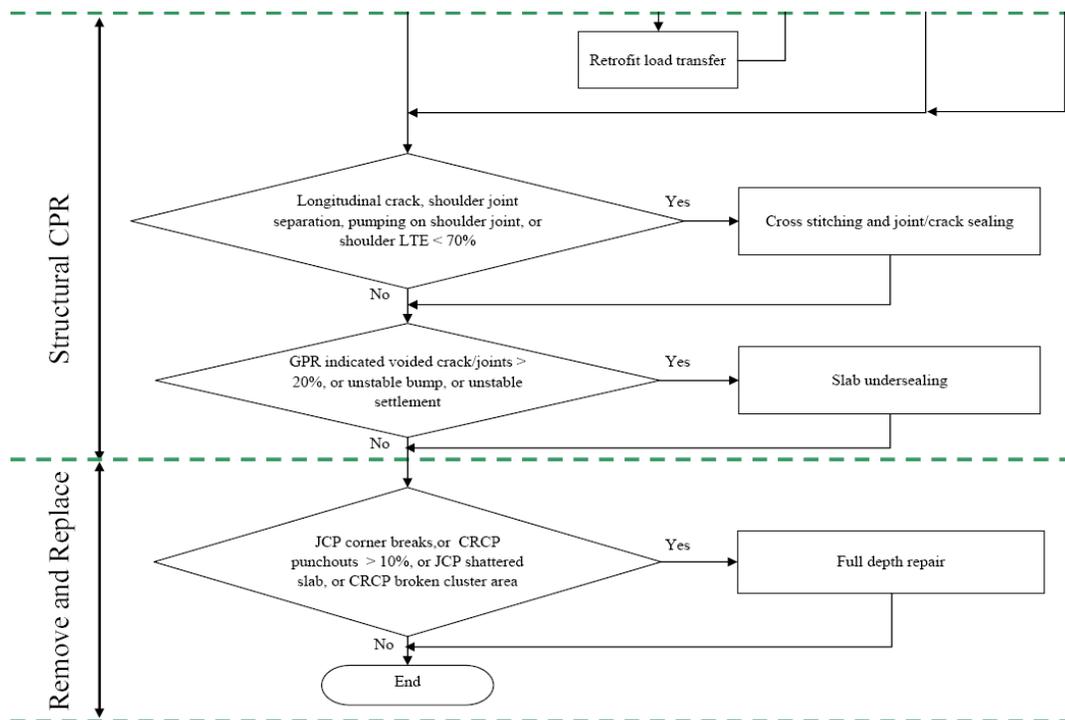
## **Remove and Replace**

Remove and replace is the most expensive repair solution and extreme stage of repair of PCC pavement and is used when other maintenance techniques are not appropriate. Since full-depth repair can be time-consuming, precast concrete panels are recommended to reduce traffic congestion costs. Figure 6 shows guidelines for remove and replace.

## **Full-Depth Repair**

Full-depth repair removes all deterioration in the distress area and restores load transfer at joints and cracks. This repair method should be considered when corner breaks are more than 10 percent or the slab is shattered in JC pavement and when punchouts are more than 10 percent in CRC pavement. Soft subgrade materials may require removal and full-depth repair, particularly in areas where cluster cracking is present. Grinding may be used to

improve roughness created by placement of the FDR. If the deterioration is widespread over the entire project length, an overlay or reconstruction may be more cost-effective (6).



**Figure 6. Routine Maintenance Decision Flowchart – Structural CPR and Replace.**

### EXAMPLE FIELD EVALUATION

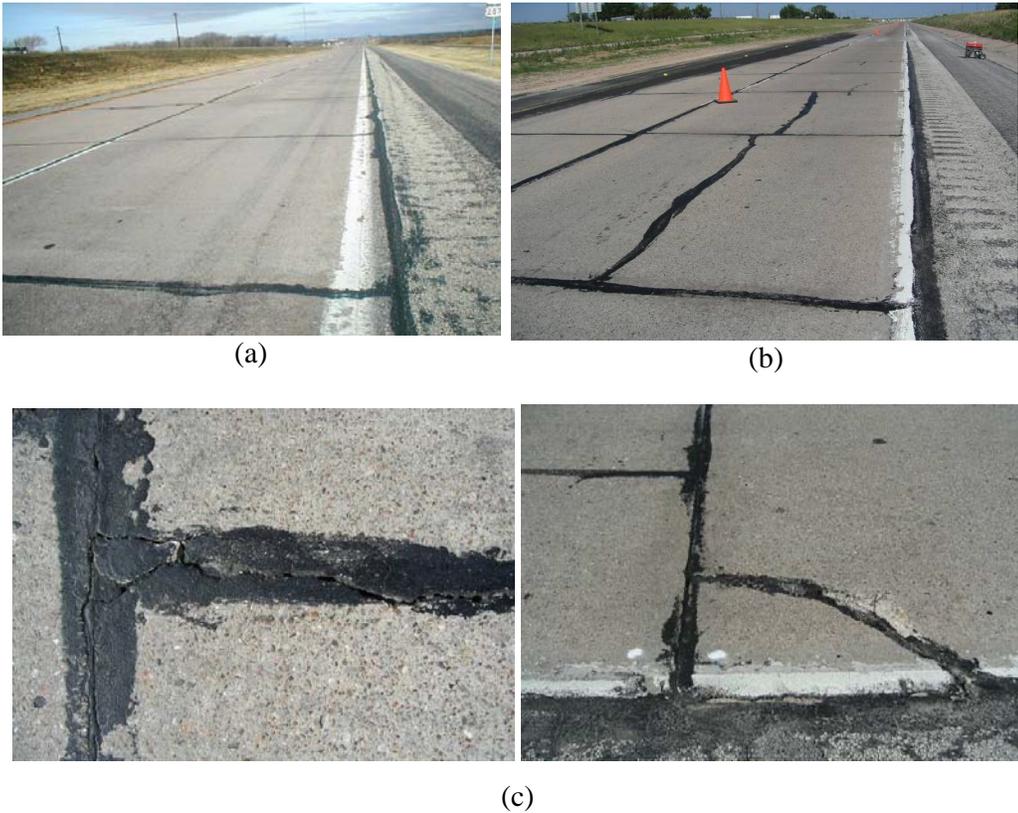
US-287 northbound JC pavement sections near Vernon, Texas, were sampled, and Figure 7 shows good and poor condition sections based on visual distress survey results. Both sections, located 3 mi (4.8 km) apart, consist of 10-in. (254-mm), 15 ft (4.6-m), jointed concrete pavement. The significant difference between them is that the good condition section has better joint sealing than the poor condition section. The crack and joint sealing in the poor condition section is severely damaged (or improperly resealed) in many areas, and the shoulder joint is wide, as shown in Figure 7.

FWD test results in Figure 8-a show no significant difference in LTE and BA between good and poor condition areas (visually decided). Both good and poor sections have lower than 70 percent of LTE with high standard deviation and an apparent restoration of load transfer to prevent further deterioration. As mentioned previously, a good joint sealing condition may result in relatively good performance even though the level of LTE is low.

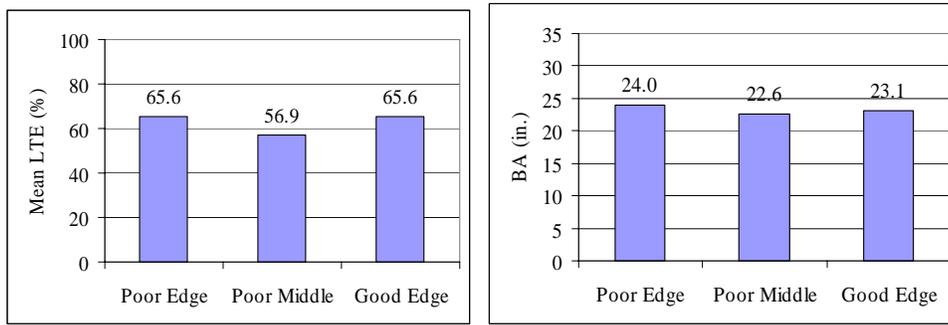
DCP test results shown in Figure 8-b indicate a weak subbase condition for both sections in the Vernon area. It is noted, however, that the modulus of the good performing area is lower than the modulus in the poorly performing area. A possible reason is that the good condition section has experienced less erosion and lower pumping damage due to good joint sealing but the poor condition section has experienced more damaged due to surface water infiltration through opened cracks and joints.

Figure 8-c shows the core hole over a longitudinal crack location in the poor condition section. A horizontal crack at mid-depth and erosion in the base layer are noted. Eroded

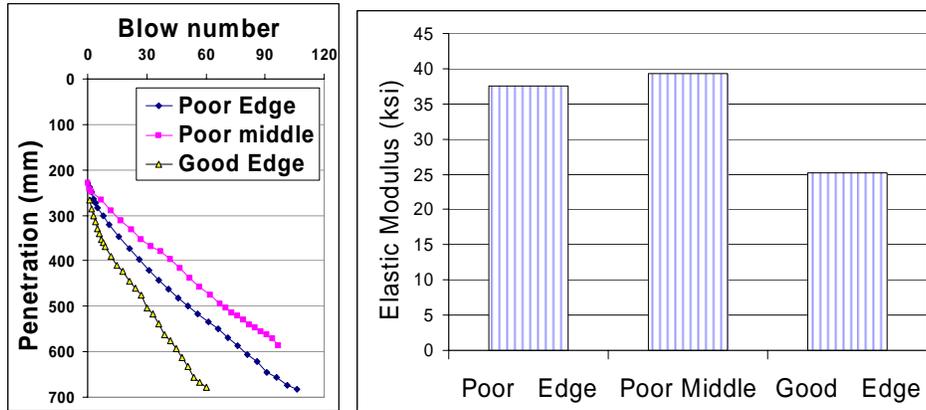
finer were detected, and the erosion depth was about 1 to 1.5 in. (25 to 28 mm) in the poor condition section. Since the aggregate type of the poor condition section is river gravel, the crack/joint opening with a higher curling effect may have contributed to pumping and erosion damage to the subbase material. The good condition section showed a smaller depth of erosion. As feasible maintenance means, joint resealing, slab undersealing, and edge drainage may be needed to fill voids under the slab and reduce further moisture damage to the base and subgrade.



**Figure 7. Test sections in Vernon: (a) good condition, (b) poor condition, (c) crack and deteriorated joint sealing in poor condition section.**



(a)



(b)



(c)

**Figure 8. US-287 sample section: (a) load transfer efficiency and basin area, (b) elastic modulus of subbase, (c) coring in poor condition section.**

AC-overlaid JC pavement on US-59 in Legget, Texas, was tested using rolling dynamic deflectometer (RDD) and GPR. Interesting repairs were under taken during rehabilitation work carried out in 2002 consisting of the replacement of an existing AC overlay where the joints in the JC pavement were repaired using two different materials: fiber-reinforced polymer and HMA with a fabric underseal, as shown in Figure 9 (9). The pavement surface condition was visually good, and there was virtually no distress over the repair section after 5 years of service. However, the first section showed an LTE of 65 percent, while the second section showed an LTE of 81 percent based on RDD results as well as a high possibility of debonding and moisture presence between the overlay and the jointed pavement in the GPR image (shown in Figure 9-c). Moreover, eroded or wet conditions under the PCC slab were detected by the GPR. According to the decision criteria, these sections should be monitored routinely for reflection cracking and sealed in a timely manner as preservative maintenance to prevent rapid joint deterioration.

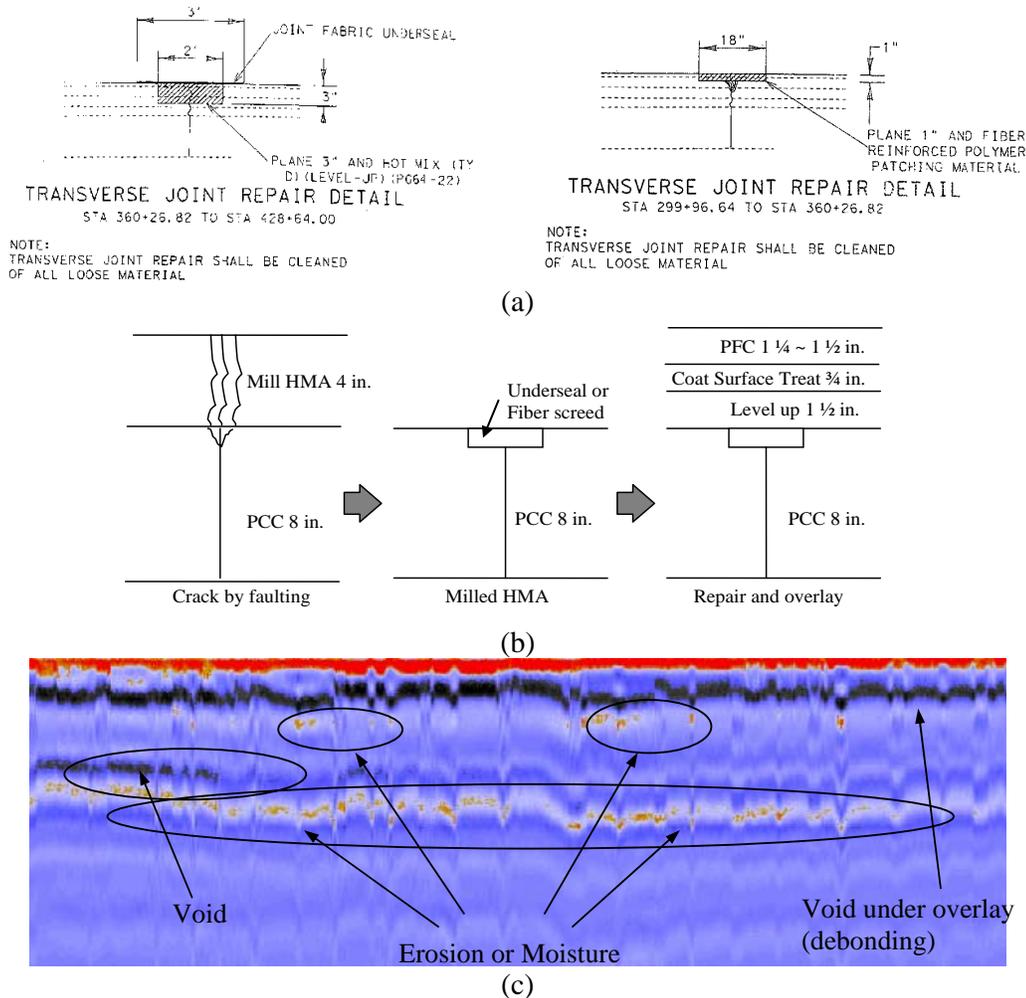


Figure 9. US-59 ACOL joint repairs: (a) standard plans, (b) repair steps, (c) GPR image.

## CONCLUSIONS

There are many well-organized visual pavement condition survey protocols used by highway agencies to monitor and record pavement distresses. However, current survey protocols often require a level of inspection detail greater than what is normally needed for a routine maintenance survey; therefore, simplified survey tables have been provided.

Based on the areas selected from the checklist of visual survey, deflection testing using FWD can evaluate the structural condition of pavement such as layer stiffness, LTE, and loss of support below the slab. GPR testing determines base conditions such as voids and the presence of water trapped in and between underlying pavement layers. Moreover, GPR survey can be used for PCC pavement layer thickness estimation, layer interface condition assessment, and dowel misalignment evaluation. DCP testing indicates the in situ strength of base and subgrade soils. It is a fast and easy method to estimate layer stiffness which is useful to estimate the effective stiffness and thickness of the pavement system.

In the field tests, the poor condition section showed more joint and crack seal deterioration than the good condition section. FWD data show no difference between good and poor

condition sections, but the LTE and basin areas are low, possibly requiring retrofit load transfer. DCP test results show how good joint sealing can effectively prevent pumping and erosion damage since the good condition area has lower subbase erosion by well-maintained joint sealing. AC overlaid sections showed no reflection cracking at the surface but did show low LTE, voids under the slab, and wet base conditions indicative of a high potential for rapid deterioration when reflection cracking takes place. Certain conditions are to be met to justify the use of retrofit load transfer and edge drains as a long-term maintenance solution but routine monitoring and timely sealing of joints and cracks should extend good conditions cost effectively.

## ACKNOWLEDGMENTS

This project was conducted in cooperation with TxDOT and FHWA. The authors wish to express their appreciation to the Federal Highway Administration and the Texas Department of Transportation personnel for their support throughout this project, as well as the Project Coordinator, Dennis R. Cooley, P.E.; the project director, Paul D. Montgomery, P.E.; and members of the Project Monitoring Committee.

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# **Pavement Preservation for High Traffic Volume PCC Roadways: Phase I Findings From SHRP 2 Project R26**

David Peshkin,<sup>1</sup> Angie Wolters,<sup>2</sup> Cesar Alvarado,<sup>3</sup> and Jim Moulthrop<sup>4</sup>

## **ABSTRACT**

The practice of pavement preservation in general, and preventive maintenance in particular, is a growing trend among transportation agencies around the United States. However, the practice of preservation on high traffic volume roadways is not as well documented as it is on lower volume roadways. Nonetheless, the preservation of high traffic volume roadways is as important as the preservation of lower traffic volume roadways.

Under the direction of SHRP 2, Project R26, *Preservation Approaches for High Traffic Volume Roadways*, was initiated to assess the state of the practice and to provide guidance on pavement preservation for high traffic volume roadways. The ultimate outcome of this study is the development of guidelines that lead to higher volume roadways being maintained in serviceable condition for longer periods of time before rehabilitation is needed, at a lower cost, in a safer manner, and with less disruption to the traveling public. A secondary objective of this study is to identify promising pavement preservation strategies for application on high traffic volume roadways that might not commonly be used, and to make recommendations for further research opportunities.

As part of the first phase of this study, a 24-question survey of practice was distributed to all state highway agencies and Canadian provinces, as well as selected large cities and several agencies in other countries. Ultimately, 57 responses were received. While survey questions addressed both hot-mix asphalt and portland cement concrete (PCC) pavements, this paper focuses on the responses related to pavement preservation on PCC pavements.

## **INTRODUCTION**

The practice of pavement preservation in general, and preventive maintenance in particular, is a growing trend among transportation agencies around the United States. Over the past decade alone, a number of State highway agencies (SHAs) have created or formalized their preservation programs. At the same time, other agencies that might have been practicing preservation for a longer time have extended their programs to cover a greater proportion of their pavement network than ever before while still other agencies are currently in the process of creating formal preservation programs.

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Agencies at the forefront of the preventive maintenance movement have a number of shared practices. For example, several agencies have created a departmental position of Pavement Preservation or Preventive Maintenance Engineer. Many of these same agencies have developed, or are developing, formal guidelines for preservation, such as Caltrans' Maintenance Technical Advisory Guide for rigid pavements (Caltrans 2008). Furthermore, States such as Texas and California have established pavement preservation centers, where researchers and practitioners work together to improve preservation practices. In addition to such centers, States have aligned themselves into regional partnerships to facilitate the exchange of ideas and best practices regarding pavement preservation.

Based on transportation agency practices in this area, the growing significance of preservation practices is indisputable. However, the practice of preservation on high traffic volume roadways is not as well documented as it is on lower volume roadways. There are several possible explanations for this.

- Preservation is simply associated with lower volume roads.
- There is an implied liability problem associated with the failure of certain treatments on higher volume roadways (for example, treatment failures in the 1990s in New York and in the past 6 years in Colorado and Michigan).
- The potential benefit of preservation on higher traffic volume roadways might not be as readily recognized or as well-documented.
- There is a smaller set of materials and procedures that can be employed successfully on high traffic volume roadways.
- Shorter available closure times for busy roadways make treatment construction more complicated.
- Because these pavements are typically designed and built to higher standards than lower volume roadways, they might therefore deteriorate in different ways, rendering typical preventive maintenance treatments less effective.

Nonetheless, the preservation of high traffic volume roadways is as important as the preservation of lower traffic volume roadways, as many of the same conditions hold true:

- Agency resources are limited, and it is important to make the best use of available funding, personnel, and equipment in managing pavements.
- In the long run, pavement preservation saves money.
- Preservation provides benefits to the traveling public, including safer, smoother roads.
- Preservation can be performed more rapidly than rehabilitation, with fewer adverse effects on the traveling public.

Under the direction of SHRP 2, Project R26, *Preservation Approaches for High Traffic Volume Roadways*, was initiated to assess the state of the practice and to provide guidance on pavement preservation for high traffic volume roadways. The ultimate outcome of this study is to develop

guidelines that lead to higher volume roadways being maintained in serviceable condition for longer periods of time before rehabilitation is needed, at a lower cost, in a safer manner, and with less disruption to the traveling public. A secondary objective of this study is to identify promising pavement preservation strategies for application on high traffic volume roadways that might not commonly be used, and to make recommendations for further research opportunities.

As part of the first phase of this study, a 24-question survey of practice was distributed to all SHAs and Canadian provinces, as well as selected large cities and several agencies in other countries. Information was sought on the following:

- Successful techniques for pavement preservation on high traffic volume roadways currently in use.
- Potentially successful techniques for pavement preservation approaches that are not yet fully deployed.
- Challenges and solutions to implementation on high traffic volume roadways.
- Special considerations for quality control/quality assurance (QC/QA).

Ultimately, 55 responses were received from 102 distributed surveys for a response rate of over 55 percent. As shown in Table 1, respondents include 40 SHAs and 7 provinces.

**Table 1**  
**Summary of Survey Respondents**

<i>State Highway Agencies</i>			<i>Canadian Provinces</i>
Alaska	Louisiana	Ohio	Alberta
Arizona	Maine	Oklahoma	British Columbia
Arkansas	Michigan	Pennsylvania	Manitoba
California	Minnesota	Rhode Island	New Brunswick
Colorado	Mississippi (4)*	South Carolina	Ontario
Connecticut	Missouri	South Dakota	Quebec
Florida	Montana	Tennessee	Saskatchewan
Georgia	Nebraska	Texas	
Hawaii	Nevada	Utah (3)*	<i>Cities</i>
Illinois	New Hampshire	Virginia	Phoenix, AZ
Indiana	New Mexico	Washington	San Diego, CA
Iowa	New York	Wisconsin	
Kansas	North Carolina	Wyoming	Texas Turnpike
Kentucky			

\* Agencies that submitted multiple responses from various districts within the State have the number of responses indicated in parentheses.

While survey questions addressed both hot-mix asphalt (HMA) and portland cement concrete (PCC) pavements, this paper focuses on the responses related to pavement preservation on PCC pavements. It should be emphasized, however, that some of the questions about agency practices or issues did not differentiate between HMA and PCC.

## RESULTS

One of the first issues to consider is what defines a high traffic volume roadway. The survey purposely did not provide arbitrary guidelines, instead asking respondents to define these based on their local practice or policies. Furthermore, a distinction between rural and urban roadways was maintained throughout the survey in recognition of differences in practice that might exist among some agencies. About two-thirds of the respondents said that they use different treatments on rural and urban roadways, and in some respects this differentiation was borne out by the data. As for the “definition” of high traffic volumes, there were broad ranges reported for both rural and urban high traffic volumes, in both cases ranging from average daily traffics (ADTs) of 1,000 to 100,000. However, the trends in the responses to this survey support the use in any further analysis of 5,000 ADT as the cut-off for high traffic volume rural roadways and 10,000 ADT as the cut-off for high traffic volume urban roadways.

Respondents were also asked to identify what factors they considered to be important when selecting preventive maintenance treatments for high traffic volume roadways and to rank these as high priority, medium priority, and low priority. The results are summarized in order below, with the percent of respondents identifying the consideration indicated in parentheses.

### *High Priority*

- Safety concerns (76 percent)
- Treatment cost (74 percent)
- Durability/expected life of treatment (64 percent)

### *Medium Priority*

- Availability of experienced contractor (60 percent)
- Work zone considerations (59 percent)
- Risk associated with treatment failure (57 percent)
- Closure time (57 percent)

### *Low Priority*

- Availability of alternate route(s) (40 percent)
- Noise issues (39 percent)
- Public perception (36 percent)

While respondents could enter information on any treatments, the treatments indicated in Table 2 were provided as being representative of those that could be used for the preventive maintenance of PCC pavements.

**Table 2  
Summary of PCC Treatments**

<b>PCC Treatments</b>
<ul style="list-style-type: none"> <li>• Concrete joint resealing</li> <li>• Crack sealing</li> <li>• Diamond grinding</li> <li>• Diamond grooving</li> <li>• Partial-depth concrete pavement patching</li> <li>• Full-depth concrete pavement patching</li> <li>• Dowel bar retrofit (load-transfer restoration)</li> <li>• Thin PCC overlays</li> <li>• Thin bonded wearing course (e.g., HMA &lt; 25 mm [1 in.])</li> <li>• Thin HMA overlay (&lt; 40 mm [1.5 in.])</li> <li>• Drainage preservation</li> </ul>

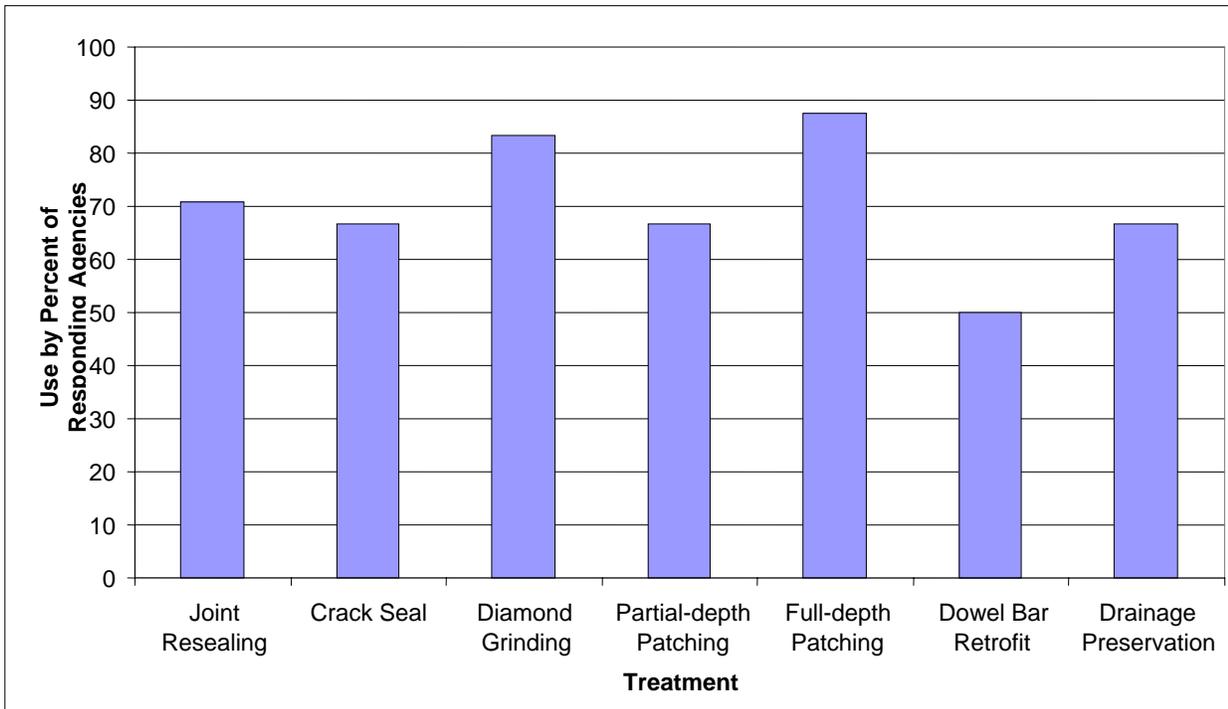
The overall responses on the use of PCC treatments on pavements with traffic volumes greater than 5,000 and 10,000 ADT for rural and urban roadways respectively are summarized in Table 3 and Figures 1 and 2. As part of the analysis of results, the responses were also divided into subsets of “low,” “medium,” and “high” traffic volumes for rural and urban roadways, and the results are summarized in Figures 3 and 4.

**Table 3  
Survey Responses of Commonly Used Preventive Maintenance  
Treatments on PCC High Traffic Volume Rural and Urban Roadways**

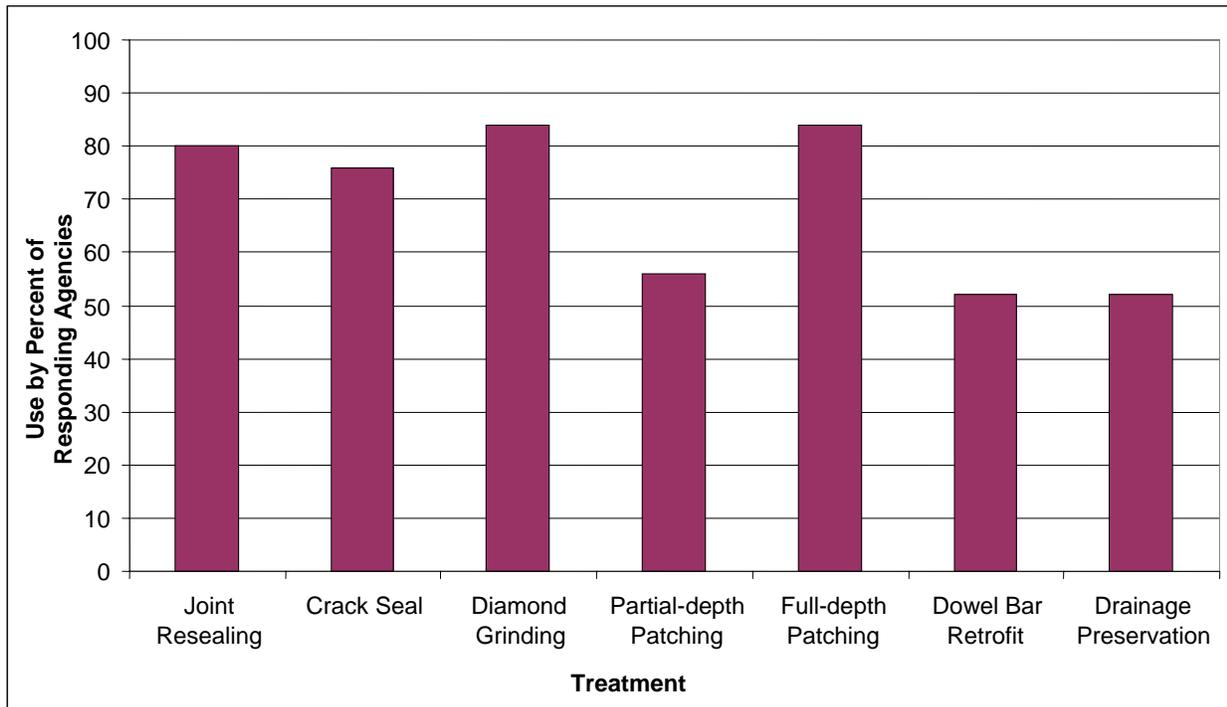
<b>Rural (≥ 5,000 ADT)</b>	<b>Urban (≥ 10,000 ADT)</b>
<i>Treatments used by more than 50% of responding transportation departments:</i>	
Crack seal Partial-depth patching Dowel bar retrofit Drainage preservation	Partial-depth patching Dowel retrofit Drainage preservation
<i>Additional treatments used by more than 70% of responding transportation departments:</i>	
Joint seal Diamond grinding Full-depth patching	Joint seal Crack seal Diamond grinding Full-depth patching

An interesting result is that for most agencies there appears to be very little difference between the use of preventive maintenance treatments for high traffic volume PCC roadways in rural and urban areas. There are several possible explanations for this, including that these treatments are not very sensitive to traffic volumes, that their use is not very risky, and that their performance is deemed to be acceptable under a fairly wide range of conditions.

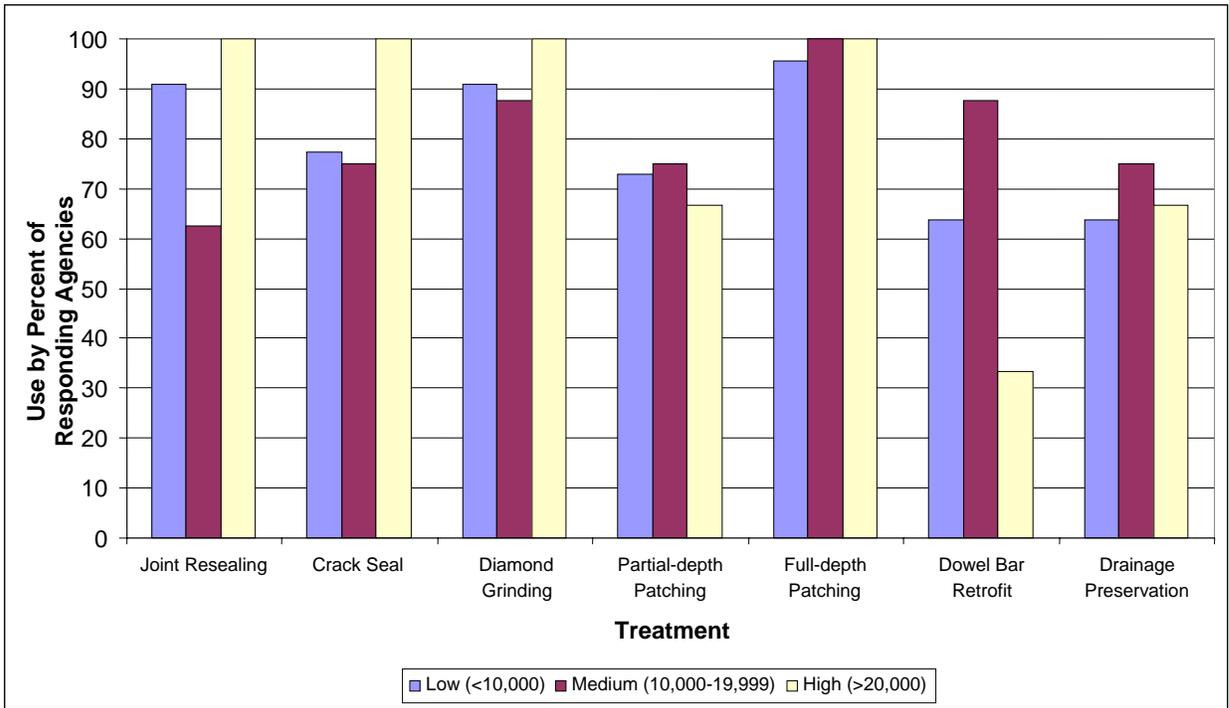
However, as shown in Figure 4, when different ranges of traffic volumes were considered for urban roadways, some differences did emerge. For example, as traffic volumes increase, the more likely an agency was to use diamond grinding and full-depth patching. Similarly, as traffic volumes increased, the less likely agencies were to use partial-depth patching, dowel bar retrofit, and drainage preservation.



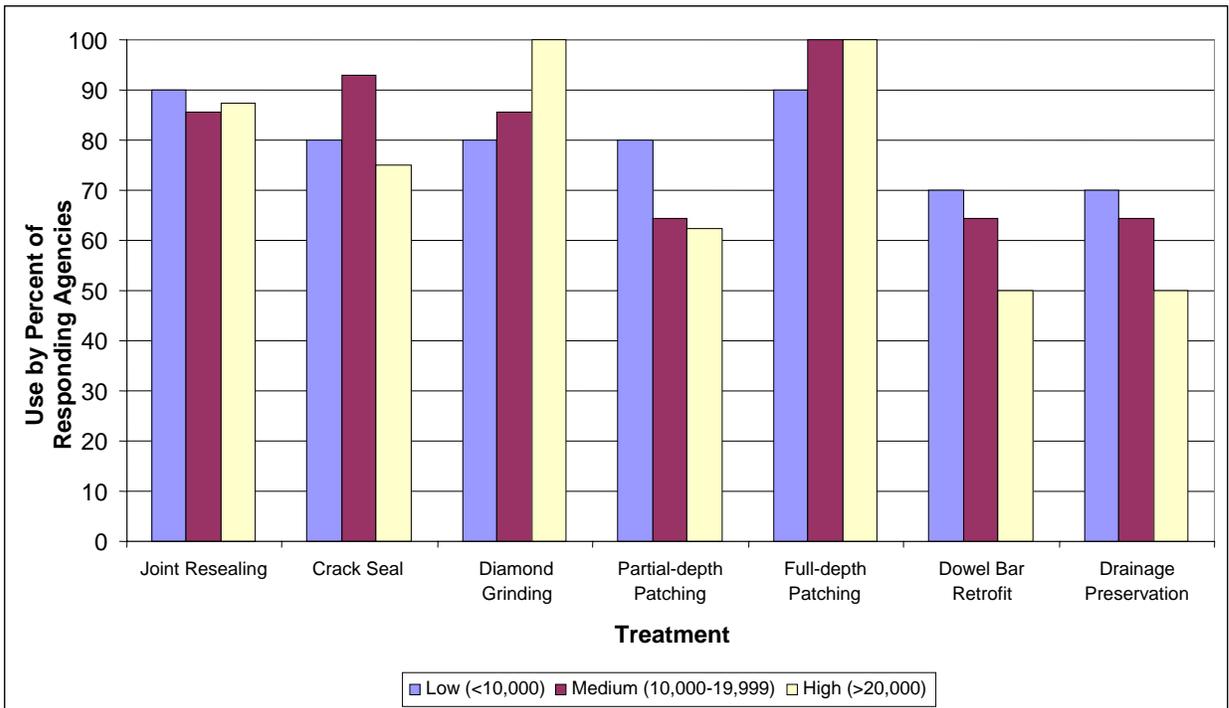
**Figure 1. Percent of agencies reporting treatment use on rural high volume traffic PCC roadways ( $\geq 5,000$  ADT).**



**Figure 2. Percent of agencies reporting treatment use on urban high volume traffic PCC roadways ( $\geq 10,000$  ADT).**



**Figure 3. Percent of treatment use on rural PCC roadways for different ranges of high traffic volumes.**



**Figure 4. Percent of treatment use on urban PCC roadways for different ranges of high traffic volumes.**

Input was also sought on those treatments considered not appropriate for preventive maintenance on high traffic volume roadways. As indicated in Table 4, the most commonly cited treatments that are not used are all thin surfacings. Possible explanations include the shorter life of these treatments, their reduced durability, and the higher risk associated with the failure of a thin treatment applied over a high traffic volume roadway.

**Table 4**  
**Summary of Preventive Maintenance Treatments Considered by Respondents To Be Not Applicable For PCC Rural and Urban Roadways**

<b>Treatments Identified Not Applicable for High Traffic Volume PCC Pavements</b>	<b>Percent of Respondents Rural / Urban</b>
Thin PCC overlays	62 / 55
Thin bonded wearing course (e.g., HMA < 1 in.)	75 / 73
Thin HMA overlays (< 1.5 in.)	62 / 55

The primary reasons cited for not using treatments included lack of agency experience, a bias against the treatment, treatment cost, and the durability/expected life of the treatment. Previous treatment failures were also given as an explanation.

Another important consideration is the available construction time associated with placing treatments on high traffic volume roadways. It is expected that the higher the traffic volumes, the less likely it is that an agency will be able (or be willing) to consider longer closures. A typical scheme for distinguishing among closure times is to divide them into single shift or overnight closures (these are typically 6 to 8 hours long), weekend closures (for example, from Friday evening after rush hour until Monday morning at the start of the next rush hour), and longer closures. As shown in Table 5, in response to the question “Under which of the following available closure time scenarios you consider using the listed treatments on urban roadways?” almost all of the treatments were identified as being able to be placed with even the shortest (overnight or single shift) closure; only thin PCC overlays were identified as more appropriate for longer closures. These results suggest that closure times are not a barrier to the use of preservation treatments on high traffic volume PCC roadways.

Another factor covered in the survey is the type of contracts used to ensure quality for preservation treatments on high traffic volume roadways. Survey responses are summarized in Table 6, and indicate that most agencies use QC/QA procedures in the placement of these treatments, followed by contract maintenance, and performance specifications. Warranties are hardly used for the construction of any of these treatments.

**Table 5**  
**Summary of Closure Time Scenarios Considered When Using**  
**a Preventive Maintenance Treatment for Urban High Traffic Volume PCC Roads**

<b>PCC Pavement Treatments for High Traffic Volume Roadways</b>	<b>Percent of Respondents</b>		
	<b>Overnight or Single Shift</b>	<b>Weekend</b>	<b>Longer</b>
Concrete joint resealing	92	30	12
Concrete crack sealing	92	32	15
Diamond grinding	95	36	13
Diamond grooving	91	30	4
Partial-depth concrete pavement patching	68	49	30
Full-depth concrete pavement patching	67	50	38
Dowel bar retrofit (load-transfer restoration)	65	44	44
Thin PCC overlays	39	30	57
Thin bonded wearing course (e.g., HMA < 1 in.)	87	26	4
Thin HMA overlay (< 1.5 in.)	88	20	8
Drainage preservation	93	30	19

Note: Overnight (e.g., from 10 p.m. to 6 a.m.); Single Shift (e.g., 9 a.m. to 4 p.m.); Weekend (e.g., from 8 p.m. Friday to Monday at 5 a.m.); Longer (Longer than 2 Days)

**Table 6**  
**Summary of Contracting Mechanisms Used to Ensure Quality for**  
**a Preventive Maintenance Treatment for High Traffic Volume PCC Roads**

<b>PCC Pavement Treatments for High Traffic Volume Roadways</b>	<b>Contracting Mechanisms Used</b>			
	<b>QC/QA</b>	<b>Performance Specifications</b>	<b>Warranties</b>	<b>Contract Maintenance</b>
Concrete joint resealing	56	31	6	39
Concrete crack sealing	56	31	6	42
Diamond grinding	59	38	6	35
Diamond grooving	55	40	5	30
Partial-depth concrete pavement patching	59	22	6	44
Full-depth concrete pavement patching	58	29	8	39
Dowel bar retrofit (load-transfer restoration)	58	32	6	35
Thin PCC overlays	56	38	0	38
Thin bonded wearing course (e.g., HMA < 1 in.)	72	33	6	33
Thin HMA overlay (< 1.5 in.)	75	35	5	40
Drainage Preservation	75	20	5	45

## SUMMARY AND CONCLUSIONS

Phase I of this study was intended to assess the state of the practice regarding pavement preservation practices for high traffic volume roadways, identify successful techniques that are not yet fully deployed, and outline challenges to the implementation of pavement preservation. As has been shown with the previously presented results, almost all of the treatments used on PCC pavements can be successfully used on high traffic volume roadways. Even treatments that are not widely used (by greater than 70 percent of respondents) are consistently used by over 50 percent of the respondents. The treatments that are not consistently used have in common that they are thin surfacings, and among the stated reasons for not using these treatments are a lack of experience and the durability/expected life of the treatment.

Those agencies that were not using pavement preservation or preservation strategies were also asked what additional guidance they needed. Over 50 percent said that they needed significant guidance on treatment durability/expected life, and over 40 percent cited information on appropriate climatic regions for treatments and applicable traffic volumes. Some guidance was identified as needed on many factors, including the following:

- Other agency experience.
- Typical noise associated with treatment.
- Treatment production rate.
- Treatment costs by region.
- Obtaining experienced contractors.
- Material availability.
- Opening to traffic.

In addition to further refinement of the Phase I results, the purpose of the second phase of this project is to develop guidelines for using pavement preservation on high traffic volume roadways. It is clear that the guidance for PCC pavements needs to specifically address the state of the practice that has made the common treatments widely used on roadways carrying all levels of traffic. Furthermore, providing information on the following factors should help to increase the use and performance of such treatments:

- Durability.
- Expected life.
- Material selection.
- Guidelines for opening to traffic.
- Noise (where appropriate).
- Costs.

In addition to considering the above factors, subsequent work in Phase II will include developing general guidance on the following aspects of treatment selection:

- Pavement condition.
- Climate/environment.

- Traffic volume.
- Available work hours.

Treatment selection tools will also be developed in phase II to help agencies make better decisions regarding the use of PCC pavement preservation on high traffic volume roadways.

## **REFERENCE**

Caltrans. *Maintenance Technical Advisory Guide, Volume II—Rigid Pavement Preservation*. Second Edition. 2008. California Department of Transportation, Sacramento.



# Performance of Edge Drains in Concrete Pavements in California

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## ABSTRACT

The California Department of Transportation (Caltrans) recently completed a study to evaluate the performance of edge drain systems placed along portland cement concrete (PCC) pavements. To date, a variety of edge drain designs, backfill materials, and placement methods have been used and have resulted in varying degrees of success when measured against overall pavement performance. This study investigated several different types of edge drain systems that have been used by Caltrans. Their performance was evaluated, and it was observed that more than 70 percent of the surveyed edge drains were not performing efficiently or as designed. This poor overall result can be attributed to design flaws, improper construction practices, and lack of maintenance. Generally, the performance of originally constructed edge drains was better than retrofit projects, since originally constructed edge drains are generally equipped with larger diameter drain pipes, deeper trenches, and treated permeable bases. Edge drain trenches in retrofit projects are generally not deep enough to effectively collect all infiltrated water from the PCC and base layers. The geotextile filter fabric materials found in excavated projects are not soil-specific, which can cause clogging and eventually reduce the ability of these edge drains to allow free flow of water. Improper construction practices, such as high percentages of cement in cement-treated permeable base backfill material and improper placement of geo-fabric material were observed in a few of the surveyed edge drain projects. Among surveyed projects, more than 50 percent of the edge drain outlet pipes were either buried or clogged, which can be attributed to lack of maintenance.

## INTRODUCTION

Excess free water in a portland cement concrete (PCC) pavement section can cause erosion in these sections, eventually leading to loss of slab support, joint faulting, and slab cracking. This process reduces ride quality and results in early maintenance and rehabilitation needs. The most typical problem is that many concrete pavements are essentially a built-in “bathtub without a drain,” and once water gets into the bathtub, it can stay there for a long period of time. If

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certain design features are not addressed regarding free water intrusion, serious problems can develop. These problems became quite apparent in the 1970s in California and other States as truck traffic increased (see Figure 1).



**Figure 1. Typical pumping and erosion of jointed plain concrete pavement with no dowels.**

At that time, States, including California, took action to reduce damage occurring in existing concrete pavements, which included keeping out water by sealing joints and cracks and draining water from the structural section by installing retrofit edge drain systems. In the 1980s and 1990s, at the strong encouragement of the Federal Highway Administration (FHWA), many States adopted full subdrainage systems consisting of a permeable drainage layer directly beneath the concrete surface, with edge drains and lateral outlets on the sides to remove water from the pavement section.

A wide array of subdrainage designs were constructed in California. The retrofit drains showed early promise in reducing pumping and joint faulting in PCC pavements. However, after a few years, these benefits virtually disappeared, and erosion and joint faulting continued, along with premature slab cracking. These problems were investigated, and it was determined that many of the retrofit edge drains, and even the full subdrainage systems, became ineffective due to design inadequacies, construction problems, and (especially) lack of maintenance. Many joints and cracks were not sealed regularly to reduce the inflow of water. Many of the retrofit edge drains were in areas with nondoweled joints that had large differential deflections as heavy wheel loads rolled over the joints. This action caused large water pressures that eroded the weakened base, subbase, and shoulder materials, causing further loss of support beneath pavement panels.

Full subdrainage systems also experienced a wide range of performance and effectiveness deficiencies. Many of these problems were related to design, materials used, and construction errors. Others were related to the failure to maintain otherwise functioning edge drains and outlets. Outlet pipes eventually clogged and stopped draining moisture from the subbase and subgrade.

## OBJECTIVES

The objectives of this study were as follows:

1. Determine the effectiveness of edge drains placed in PCC pavement systems throughout California.
2. Investigate the different types of edge drain systems used in California to ascertain any changes that could be implemented to improve their performance.

## EFFECTS OF DAMAGE DUE TO EXCESS MOISTURE IN PAVEMENTS

Damage caused by excess moisture in pavement structures depends on several key design features and material properties, as well as subdrainage capabilities. PCC pavements constructed in California with undoweled joints, erodable base materials, or erodable materials in the shoulder are all susceptible to moisture damage. In fact, the placement of edge drains along the pavement edge cannot prevent continuing erosion due to the high differential deflections at slab corners, as heavy axle loads roll over these joints. Pumping is inevitable and is often followed by joint faulting. Doweling of the joints is required to reduce differential deflections at the joints and corners. The addition of a tied PCC shoulder significantly reduces deflections and, with sealed joints, reduces the amount of water entering the longitudinal joint. Some State agencies, such as the Georgia Department of Transportation (GDOT), have elected not to place edge drains, but rather to intensify the sealing of joints to reduce moisture ingress as much as possible.

As noted in the preceding, many conventional pavement sections are built in a de facto “bathtub,” which results in water infiltrating into the roadbed structure beneath the concrete surface course. When combined with heavy truck traffic and the presence of moisture-susceptible materials, the service life of a PCC pavement is bound to be reduced.

Moisture can come from many different sources—the subgrade, the surrounding terrain, and directly through the pavement structure via surface penetration (see Figure 2). Moisture may seep upward from a high groundwater table due to capillary action, vapor movement or pumping; it can drain into the pavement substrate from the surrounding hills and drainage basin; it can infiltrate through unsealed cracks and joints on the pavement surface; or it may flow laterally from the pavement edges and side ditches (1).

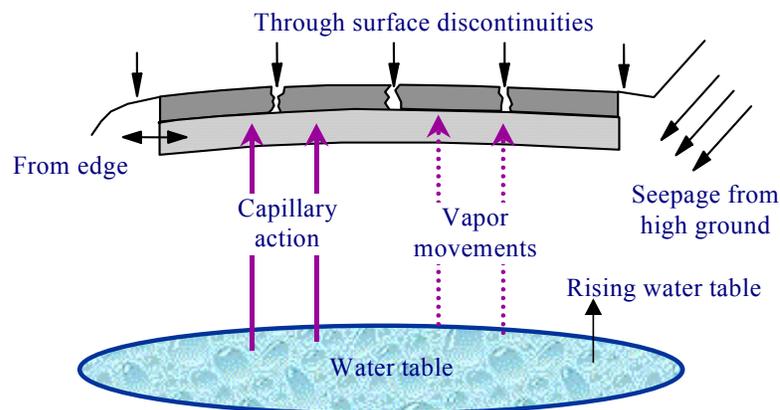


Figure 2. Sources of moisture in pavement systems (1).

Problems caused by prolonged exposure to excess moisture fall into three broad categories:

- Softening of pavement layers and the subgrade as they become saturated, and remain saturated, for lengthy periods of time.
- Degradation of moisture susceptible materials.
- Loss of bond between pavement layers due to saturation.

### **Erosion Process**

- Water infiltrates and accumulates at the bottom of slabs and stabilized base courses. Up to 80 percent of ongoing precipitation can enter through longitudinal lane or asphalt shoulder joints.
- Heavy repeated loads cause differential deflections at slab edges and corners, especially when no load transfer devices (dowel bars) are used at the transverse joint, or when the PCC shoulders are not tied to the lane.
- Differential deflections force water at a high velocity and pressure over and beneath the stabilized underlying materials, thus causing erosion of the base, subbase, and shoulder materials.
- The buildup of materials under the approach slab results in joint faulting and increases roughness over time.
- The resulting loss of support beneath the leave slab causes higher slab stresses, which eventually lead to corner, transverse, and longitudinal cracks.
- The pavement fails prematurely due to lack of structural support, repeated loadings, inadequate maintenance, and increased roughness.

### **SITE SELECTION**

Selection of project sites was based on the presence of drainage systems, variability of geographical locations, and range of traffic. A number of sites were selected from the previous Caltrans edge drain evaluation study (2). In addition, consideration was given to ease of access at the site (without traffic control) to conduct a preliminary survey. The investigation included both retrofit and original construction edge drain projects.

After reviewing project information, numerous project sites throughout the State were short-listed, representing various traffic mixes and geographical locations. From the short list, specific projects were then selected. A visual pavement condition survey, along with a detailed drainage evaluation, was conducted at each project site. Any distresses observed in the survey were rated and recorded for all identified sites. Various distress types that were recorded in accordance with severity level are presented in Table 1.

**Table 1**  
**Jointed Concrete Pavement Distress Types and Severity Levels (3)**

Distress Type	Severity Level		
	Low	Medium	High
Corner breaks	No measurable faulting	Faulting: < 13 mm	Faulting: 13 mm and up, corner piece is broken into two or more pieces
Longitudinal cracking	Crack widths: < 3 mm, no spalling or faulting	Crack widths: 3 to <13 mm; or spalling: < 75 mm or faulting: < 13 mm	Crack widths: ≥ 13 mm; or spalling: ≥ 75 mm or faulting: ≥ 13 mm
Transverse cracking	Crack widths < 3 mm, no spalling or faulting	Crack widths: 3 to < 6 mm; or spalling: < 75 mm or faulting: < 6 mm	Crack widths: ≥ 6 mm; or spalling: ≥ 75 mm or faulting: ≥ 6 mm
Spalling of longitudinal joints	Spalls: < 75 mm wide	Spalls: 75 to 150 mm wide	Spalls: > 150 mm wide; broken into two or more pieces
Spalling of transverse joints	Spalls: < 75 mm wide	Spalls: 75 to 150 mm wide	Spalls: > 150 mm wide; broken into two or more pieces
Faulting of transverse joints and cracks	Faulting: < 6 mm	Faulting: 6 to 13 mm	Faulting: > 13 mm
Lane / Shoulder dropoff or separation	Severity level not applicable; a complete record of the measurements taken is much more desirable, however, because it is more accurate and repeatable than severity level		
Patch deterioration	No faulting; no pumping	Faulting: < 6 mm; no pumping	Faulting: ≥ 6 mm; pumping
Pumping	Severity level not applicable because the amount and degree of water bleeding and pumping changes with varying moisture condition		

1 mm = 0.039 in.

A total of 30 projects in 20 counties throughout California, representing seven Caltrans districts, were identified for the preliminary surveys (4). Among these projects, six had been overlaid with AC; hence, no drainage evaluations were conducted on these projects. Therefore, as shown in Figure 3, 24 projects were surveyed in 15 different counties, and 9 were selected for further evaluation by excavating the shoulder (see Table 2). The selection for shoulder excavation was based on information collected from the field survey and the availability of traffic control. A brief description of the performance of edge drains is presented in the following section.

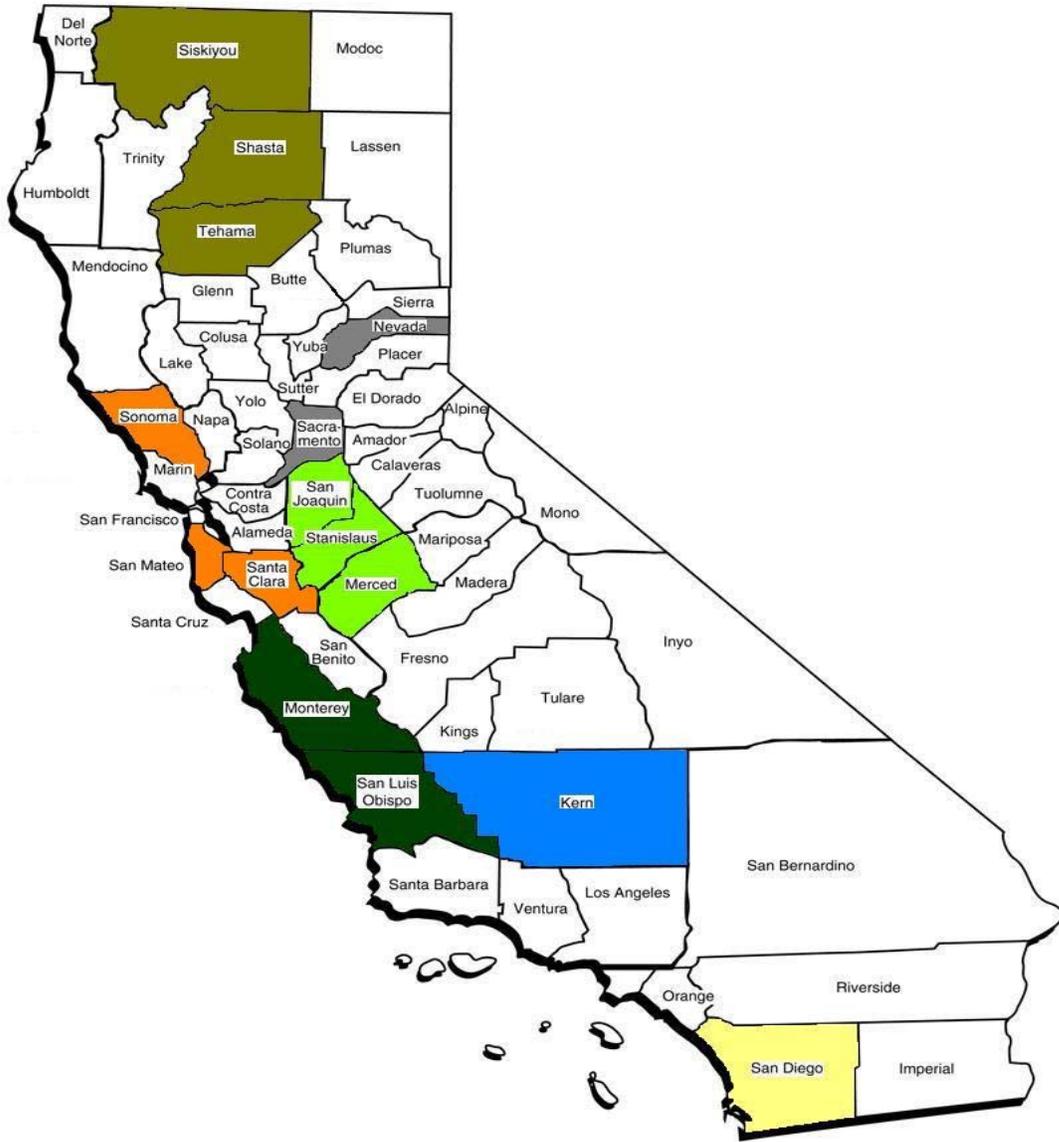


Figure 3. Locations of edge drain projects (4).

**Table 2**  
**Field Investigation Sites (4)**

<b>Sl. No./Route/District/County</b>	<b>Post Mile</b>	<b>Permeable Material</b>	<b>Field Survey/ Shoulder Excavation</b>	<b>Edge Drain Observation</b>	<b>Observed Distresses</b>
1 / I-5 (NB) / 2 / Siskiyou	51–58	UTPM	Survey	Flowing water through outlets	No notable distresses
2 / I-5 (NB) / 2 / Siskiyou	11.17–18.30	UTPM	Survey	Clogged outlets	About 1-in. of joint faulting
3 / I-5 (NB) / 2 / Shasta	14–18	UTPM	Survey	Clogged outlets	About 0.5-in. of joint faulting
4 / I-5 (NB) / 2 / Tehama	41–42	CTPM	Excavation	70% clogged drainage pipe	Medium severity transverse cracking
5 / I-5 (NB) / 2 / Tehama	23.05–25.65	UTPM	Survey	Buried outlets	High severity corner breaks & patch deterioration
6 / I-5 (NB) / 3 / Sacramento	4.70–12	-	Survey	Buried outlets	No notable distresses
7 / I-80 (EB&WB) / 3 / Nevada	2.80–7	-	Survey	Clean outlets	No notable distresses
8 / US-101 (NB&SB) / 4 / Sonoma	44.95–46.85	-	Survey	Buried outlets	No notable distresses
9 / I-280 (SB) / 4 / San Mateo	15–16	CTPM	Excavation	90% clogged drainage pipe	No notable distresses—ground pavement
10 / US-101 (NB) / 4 / Santa Clara	16–17	UTPM	Excavation	No edge drains Found	High severity corner breaks, transverse and longitudinal cracks
11 / US-101 (NB) / 5 / Monterey	27.50–30	-	Survey	Clogged outlets	No notable distresses
12 / US-101 (NB) / 5 / San Luis–Obispo	37.70–42.50	-	Survey	Clean outlets	No notable distresses
13 / I-5 (NB) / 6 / Kern	0–3.50	-	Survey	Buried outlets	High severity faulting, transverse and longitudinal cracking; pumping
14 / SR-99 (NB) / 6 / Kern	31.50–34.43	-	Survey	Buried outlets	No notable distresses
15 / SR-120 (WB) / 10 / San Joaquin	1.60–6.40	-	Survey	Flowing water through outlets	No notable distresses
16 / I-205 (EB) / 10 / San Joaquin	3–15	UTPM	Survey	Clogged outlets	Medium severity transverse cracking and joint faulting
17 / SR-99 (NB) / 10 / Stanislaus	18.90–24.60	CTPM	Survey	Buried outlets	Medium severity faulting
18 / SR-99 (NB) / 10 / Stanislaus	10.20–14.90	CTPM	Survey	Buried outlets	Medium severity faulting
19 / SR-99 (NB) / 10 / Merced	36–37	CTPM	Excavation	100% clogged drainage pipe	High severity corner breaks, transverse and longitudinal cracking, and faulting—slab replacement in progress
20 / SR-99 (NB) / 10 / Merced	21–2	CTPM	Excavation	100% clogged drainage pipe	High severity corner breaks, transverse and longitudinal cracking, and faulting—slab replacement in progress
21 / I-15 (SB) / 11 / San Diego	46–47	CTPM	Excavation	Clear drainage pipe	High severity corner breaks, transverse and longitudinal cracking, and faulting
22 / I-5 (NB) / 11 / San Diego	35–36	CTPM	Excavation	No Edge Drains Found	No notable distresses
2 / I-8 (EB) / 11 / San Diego	40–41	CTPM	Excavation	Clear drainage pipe	No notable distresses
24 / I-8 (WB) / 11 / San Diego	52–53	CTPM	Excavation	40% clogged drainage pipe	High severity lane/shoulder patch deterioration

UTPM = untreated permeable material; CTPM = cement treated permeable material

## PERFORMANCE OF EDGE DRAINS IN CALIFORNIA

Only a few (< 30 percent) of the edge drains investigated were operating in an acceptable manner (see, for example, Figure 4). The properly operating edge drains were generally in areas of higher rainfall, especially in the upper foothill and mountainous areas, where the natural materials were coarser and noncohesive in nature. Considering this fact, it can be assumed that the volume and velocity of water drained from these pavement sections prevent the fines from settling and clogging the drainage pipes.



**Figure 4. Flowing water from edge drain outlets on northbound I-5 in Siskiyou County.**

The majority of the remaining sites investigated appear to have had little or no maintenance. These sites revealed drain pipes clogged with soil from both roadbed drainage and the shoulder area, and a majority of outlets were completely covered with dirt or were overgrown with vegetation (see for example Figures 5 and 6). These existing conditions would not allow the systems to drain properly, thus trapping the water beneath the concrete pavement and adversely affecting the base and subgrade and causing premature pavement failure. However, other factors such as traffic, age of the pavement, materials, design, construction, and climate also contribute to pavement failure. In addition, generally due to recent pavement rehabilitation, a few pavement sections did not show any visible distresses although the edge drains were clogged. Accordingly, the correlation between observed pavement distresses and clogged edge drains was not found to be significant (see Table 2).

Many of the pavement sections investigated with retrofit edge drains were more than 30 years old. Maintenance is badly needed to preserve the structural integrity of the edge drain structure and prevent further accumulation of fines in the drain pipes. Also, a majority of the outlet pipes have no end wall protection, which can cause clogging. The use of end walls reduces (but does not eliminate) the need for maintenance.



**Figure 5. Clogged edge drain outlet pipe on northbound I-5 in Sacramento County.**



**Figure 6. Clogged and buried edge drain outlet pipe on northbound I-5 in Tehama County.**

Apart from lack of maintenance, design flaws and improper construction practices were also observed in a majority of the excavated edge drain projects. A summary of observed edge drain discrepancies are as follows:

- The lane/shoulder interface becomes critical when it comes to water infiltration into the base section. In the section of the roadway on I-15 in San Diego County, water runs through the interface and accumulates on the cement-treated base extension along with water trapped under the PCC slabs (see Figure 7), thus, allowing base erosion through slab action and resulting in loss of slab support.
- On I-5 in Tehama County, the nonwoven filter fabric, which has a high permeability, was double layered around the circumference of the permeable material (see Figure 8). Complete encapsulation of the permeable material with nonwoven filter fabric caused fines to infiltrate—and eventually clog—the edge drain.
- In the same section, no outlets were found within the project limits. The outlets were probably buried under the soil, not allowing the water to drain. The fines from the shoulder area might be entering through outlets and clogging the drainage pipe (see Figure 9).
- As per the Standard Plans (see Figure 10) for SR-99 in Merced County, slotted drain pipes were placed at the same level as the slab/base interface. This design feature does not provide enough depth for the drain pipe to collect all infiltrated water from the PCC and base layers.
- On SR-99 in Merced County, due to the low rainfall in the region, there was a tendency for fine particles to settle at the bottom of the drainage pipe, resulting in an inadequate velocity to drain the fine particles which were mixed with infiltrated water. This often causes sedimentation, eventually resulting in clogged drain pipes (see Figure 11).

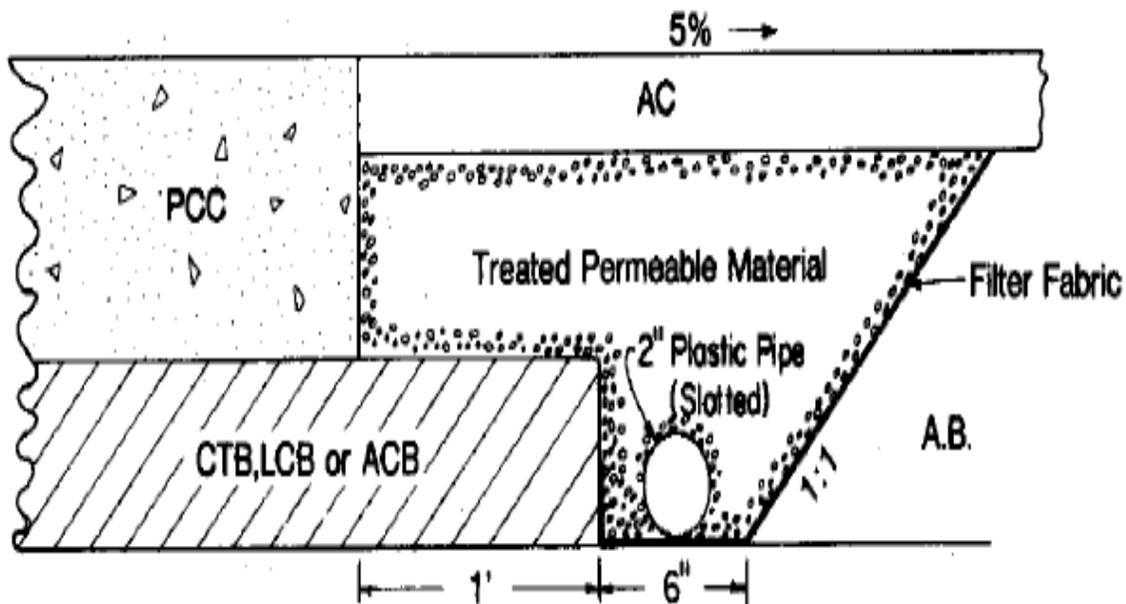


Figure 7. Edge drain design on southbound I-15 in San Diego County (5).



**Figure 8. Double-lined woven filter fabric on northbound I-5 in Tehama County.**



**Figure 9. Clogged edge drain pipe on northbound I-5 in Tehama County.**

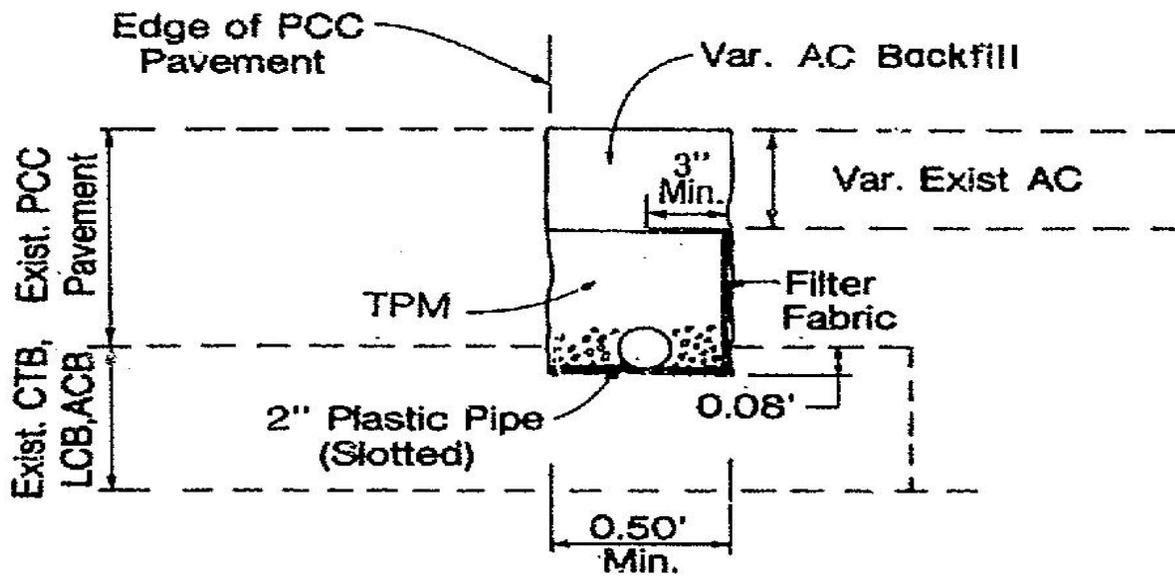


Figure 10. Edge drain design for northbound SR-99 in Merced County (6).



Figure 11. Fully clogged edge drain pipe on northbound SR-99 in Merced County.

## CONCLUSIONS

Based on a literature review, engineering knowledge, and an extensive survey of existing edge drain sites in California, the following conclusions can be drawn:

- Among the nine excavated projects, 71 percent of the edge drain pipes were clogged. Of the 24 surveyed projects, 77 percent of the outlets were clogged.
- The performance of originally construction edge drains is relatively better than retrofit projects, as original construction edge drains generally have larger diameter drain pipes, deep trenches, and treated permeable bases.
- The performance of edge drains in high rainfall areas is better than those at relatively lower rainfall areas.
- One hundred percent of the retrofit edge drains with slotted pipes 2 in. (51 mm) in diameter were totally clogged, while 50 percent of the retrofit edge drains with slotted pipes 3 in. (76 mm) in diameter were partially clogged.
- The edge drain trenches in retrofit projects are, generally speaking, not deep enough to effectively collect all infiltrated water from PCC and base layers.
- The geo-textile filter fabric materials found in excavated projects are not soil-specific, which can cause clogging and eventually reduce the ability of the edge drain system to function properly.
- As a result of improper construction procedures, several edge drains were installed in the higher side of cross slope, which prevents the water flowing to the drain pipe.
- Also, improper construction practices, such as high percentages of cement in cement-treated permeable base backfill material and improper placement of geo-fabric, were observed in few edge drain projects.
- Among the 24 surveyed projects, 73 percent of the edge drain pipes were not draining water properly, which can be attributed to lack of maintenance.

## RECOMMENDATIONS

- Edge drain systems should not be installed if there is no long-term commitment to maintain such a system.
- The amount of recent and historical rainfall that has occurred in the project area as well as the permeability of the natural soil in the area should be investigated prior to designing an edge drain system.
- When edge drains are selected for a given project, they may not be required throughout the entire project, but rather only in critical drainage areas.

- Soil investigation should be conducted and findings should be used to determine the appropriate geo-fabric required for edge drain design.
- Filter fabric should be placed along the shoulder side and bottom of the trench to prevent migration of aggregate base fines into the drainage medium.
- Slotted pipes 4 in. (102 mm)—instead of 2 or 3 in. (51 or 76 mm)—in diameter should be used, because the larger diameter will allow video inspections for maintenance purposes.
- Dual outlet features should be included in the design for easier maintenance.
- Retrofitted edge drain pipes should be placed at the bottom of the base so that all infiltrated water from the PCC and base can be effectively collected by the drain pipe.
- Construction inspection will ensure that the trench configuration, geo-fabric installation, drain pipe placement, and permeable backfill placement all meet the project design and specifications.
- Local maintenance personnel must be trained to conduct timely maintenance of edge drain systems in their districts.
- With the introduction of load transfer devices (dowel and tie bars), daylighted permeable base sections and asphalt concrete interlayers, edge drain systems may not be as beneficial in the long run and may not considerably improve the performance of PCC pavements.

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## **Part 3**

# **Concrete Pavement Repair Techniques and Experiences**



# **Retrofit Dowel Bars in Jointed Concrete Pavement— Long-Term Performance and Best Practices**

Thomas Burnham<sup>1</sup> and Bernard Izevbekhai<sup>2</sup>

## **ABSTRACT**

As jointed concrete pavements age, they typically experience panel cracking, joint or crack faulting, and surface distress. To maintain user satisfaction and safety, the agency or owner must identify the causes of the distress and consider types of feasible repairs, if any. These repairs range from partial to full-depth concrete repairs for spalled or cracked panels, to load-transfer and ride-quality restoration schemes for faulted cracks and joints. Retrofitting dowel bars into a distressed and faulted concrete pavement has become a proven technique for restoring or improving the capacity of jointed concrete pavements. The backfill materials and installation techniques used in retrofitting dowel bars must, however, be carefully designed. Numerous field and laboratory trials have been carried out in Minnesota in the recent past, allowing engineers and contractors to refine the installation techniques and materials necessary to produce long-lasting and effective projects. This paper provides a history of the development of best practices for retrofitting dowel bars into jointed concrete pavements located in the extreme climate of Minnesota. The performance of field test sections, up to 13 years old, are discussed in relation to dowel bar location, long-term load-transfer capability, and durability of backfill materials. Implemented design changes based on results from accelerated loading laboratory studies are discussed. An effective installation method and materials testing process, required of contractors before constructing retrofit projects in Minnesota, is described. Retrofit dowel bar installation, in conjunction with restoration of the surface through diamond grinding, has been proven to significantly extend the capacity and serviceable life of many concrete pavements in Minnesota.

## **INTRODUCTION**

This paper discusses the long-term field performance of four experimental retrofit dowel bar test sections in Minnesota. It also outlines the development of the current best practices used by the Minnesota Department of Transportation (Mn/DOT) for joint and crack load-transfer restoration, including those influenced by accelerated laboratory testing.

### **Background**

Highway agencies typically expect long and relatively maintenance-free performance from jointed concrete pavements. However, due to age, increased traffic loads, older design standards, or poor construction, these pavements eventually will require some maintenance or rehabilitation.

As jointed concrete pavements age, they typically experience one of three distresses. Often the first distress to appear will be panel cracking, either across the middle of the panel, or

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near the corners of the panels. Mid-panel cracking is most often caused by designs with panel lengths greater than 20 ft (6.1 m). These designs were common in Minnesota during the 1960s and 1970s, and often contained reinforcing steel near mid-panel to keep the expected crack tight. Corner cracking is often found in designs with skewed transverse joints, a design feature carried over from projects with undoweled joints.

The second distress to occur in jointed concrete pavement is faulting, or stepping, of the transverse cracks or joints. For longer panel length designs, the faulting often occurs at the mid-panel crack after the reinforcing steel has been weakened by rust, often to the point of rupture. Other designs will fault at the transverse joints if dowel bars become significantly deteriorated, or if they were never placed initially due to low design traffic volumes. In some cases, poor construction practices lead to joints with misaligned dowel bars, which can rapidly lead to joint faulting under heavy traffic volumes.

The final distress to occur in jointed concrete pavements is surface distress. This is often demonstrated by shallow surface spalling or delamination caused by material incompatibility, chemical attack, or environmental exposure (freeze/thaw).

Once a concrete pavement experiences some type of distress, the agency or owner must identify the cause and consider types of feasible repairs, if any. These repairs range from partial to full-depth concrete repairs for spalled or cracked panels, to load-transfer and ride-quality restoration schemes for faulted cracks and joints.

The use of retrofit dowel bars for load-transfer restoration has become a proven technique over the last two decades. Good information on other agency's experience with retrofit dowel bars can be found in references by Hall et. al. (1), Larson et.al (2), and Pierce et al. (3). Much of the research and development on this technique has been done with regards to the number and geometry of the dowel bars required to effectively transfer load across cracks or joints. Another important aspect to be studied however, is the backfill mortar material. This material needs to both secure the dowel bars to the pavement for transfer load, and demonstrate durability for long-term performance.

Load-transfer restoration using retrofit dowel bars has become a common pavement rehabilitation technique in Minnesota. Like many other states however, most of the early designs were experimental. Several dowel bar patterns and backfill materials were incorporated into test sections within rehabilitation projects, to investigate their performance in Minnesota's extreme climate. Mn/DOT first installed retrofit dowel bars into a distressed concrete pavement in 1994. To speed up the understanding of various retrofit dowel bar design features, Mn/DOT and the University of Minnesota teamed together in the late 1990's to develop an accelerated load test platform, known as the "Minne-ALF." Testing results from the device resulted in the implementation of several innovative retrofit dowel bar designs and installation techniques. More information on the Minne-ALF results can be found in references by Embacher et al. (4) and Popehn et al. (5).

## **LONG-TERM PERFORMANCE OF TEST SECTIONS**

The following sections discuss the design and long-term field performance of four retrofit dowel bar projects constructed in Minnesota from 1994 to 1999.

## TH 52, Zumbrota

### Project Description

The first retrofit dowel bar project in Minnesota was constructed in 1994 on Minnesota Trunk Highway 52 (TH 52) near Zumbrota. The pavement on this project was originally constructed in 1984 of jointed reinforced concrete, 225 mm (9 in.) thick. After 10 years of heavy-truck traffic and extreme weather, the 8.2-m-long (27-ft) panels contained mid-panel cracks that demonstrated virtually no faulting. The mid-panel reinforcing steel appeared to be intact, and the original sawed transverse joints with dowels were in good condition. The objective of this project was to determine if retrofit dowel bars could prevent, or significantly slow down, the development of faulted mid-panel cracks, and thus extend the service life of the pavement. Since there was no faulting of the cracks (or sawed joints) prior to retrofitting, surface grinding was not included in this project.

### Design Variables

The test sections in this project were designed to study the configuration, length, and number of retrofit dowel bars necessary to preserve long-term load transfer across transverse mid-panel cracks. Two types of backfill mortar material were used on this project to compare their load-transfer capability and durability. Except for three experimental slots, the retrofit dowel bars slots on this project were established using the saw-and-chip method. More information on this installation method will be discussed later in the best practices section of this paper.

The layout of the five test sections is shown in Figure 1. Retrofit dowel bar pattern variations included combinations of three, two, or no bars in the wheel tracks for both the driving and passing lanes. Dowel to dowel spacing in the wheel tracks was 300 mm (12 in.). For comparison, a control section containing no retrofit dowel bars, but sealed mid-panel cracks, was monitored as part of the study.

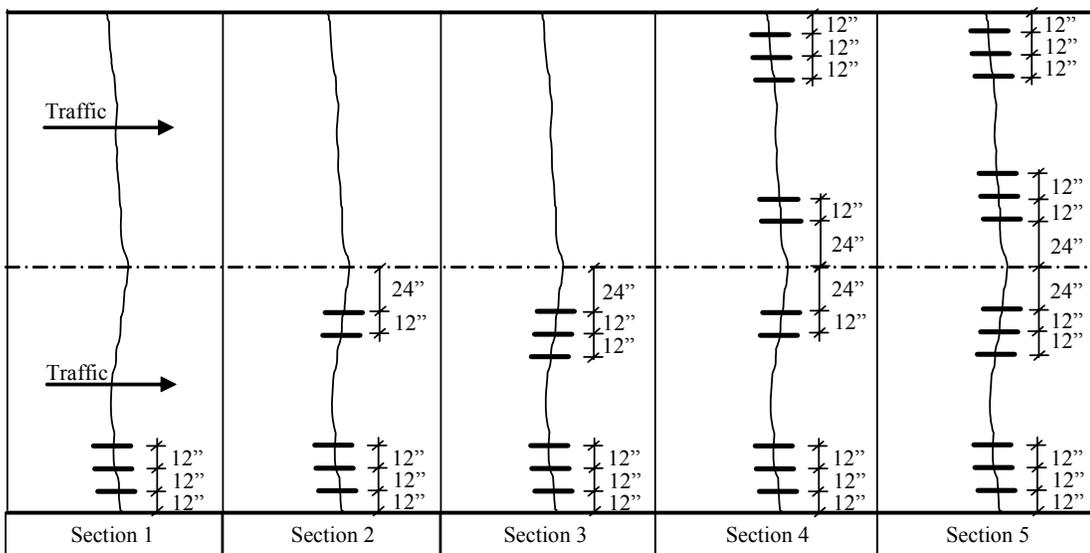


Figure 1. TH 52 test section retrofit dowel bar patterns.

Each test section is approximately 322 m (1,056 ft) long and is divided into two subsections. Transverse mid-panel cracks in the first subsection (containing approximately 20 panels) were retrofit with dowel bars 38 mm (1.5 in.) in diameter by 380 mm (15 in.) long. Transverse mid-panel cracks in the second subsection were retrofit with dowel bars 38 mm (1.5 in.) in diameter by 457 mm (18 in.) long. Each subsection also began with the first four mid-panel cracks retrofit using a polymer-modified quickset patching (PMQP) backfill mortar material. The remaining retrofit dowel bars in each subsection were installed using Mn/DOT's high early strength concrete repair patching material, designated as Mn/DOT 3U18. The primary reason for testing two types of backfill mortar is that Minnesota's extreme climate places severe demands on such materials. Pavement temperatures in Minnesota range from  $-40^{\circ}\text{C}$  to  $55^{\circ}\text{C}$  ( $-40^{\circ}\text{F}$  to  $131^{\circ}\text{F}$ ), and frequent freezing and thawing conditions in the winter and spring necessitate the use of chemical deicers. In addition, retrofit dowel bar projects typically require minimal traffic disruption, therefore requiring early strength gain of the mortar. Discovering backfill mortars that meet early strength and long-term durability criteria is the key to developing best practices for this pavement rehabilitation technique. The performance of the retrofit mortars on this project will be discussed shortly.

### *Observed Performance*

The retrofit dowel bar test sections on TH 52 were installed in 1994, and therefore to date have had over 14 years of exposure to traffic (heavy commercial average daily traffic [HCADT] = approximately 2,100)(6) and weather. Recent site visits took place in October 2006 and February 2007. During those visits, surface distress on the test sections was recorded as being minimal. Faulting of the transverse mid-panel cracks remains very low. There are a small number of distressed retrofit dowel bar groups, predominantly affected by longitudinal cracking in the panels (see Photo 1). Despite early concerns about shrinkage cracking in the many of the slots (unpublished report by Mn/DOT Geology Unit), there is little deterioration of the backfill mortar after 14 years of service. Photo 2 shows retrofit dowel bar slots with Mn/DOT 3U18 backfill mortar in very good condition. Several slots with PMQP backfill mortar experienced a loss of material of approximately 13 mm (0.5 in.) below the surface, however the integrity of the remaining material in the slot appeared to be sound (see Photo 3).



**Photo 1. Distress in TH 52 retrofit dowel bars caused by longitudinal panel cracks.**



**Photo 2. TH 52 slots with Mn/DOT 3U18 backfill mortar after 13 years.**



**Photo 3. Loss of PMQP mortar near surface of slots on TH 52.**

The objective of the research for the TH 52 test sections was to determine the long-term capability and durability of retrofit dowel bars placed across transverse mid-panel cracks. The two prominent performance measures to examine are the crack faulting and load-transfer efficiency. Periodic fault measurements of the mid-panel cracks were recorded throughout the life of the project. A randomly selected number of retrofit cracks were

measured in October 2006 and revealed an average fault value of only 1.7 mm (0.067 in.). This low level of faulting is certainly barely perceptible to the traveling public. Therefore, in terms of reducing or preventing mid-panel crack faulting in reinforced concrete pavements in Minnesota, the installation of retrofit dowel bars across previously unfaulted mid-panel cracks has successfully achieved the objectives.

Although the retrofitted cracks in TH 52 have not begun to significantly fault, there is interest in understanding when they might began that process. One common measure of a crack or joint's condition is load-transfer efficiency (LTE). LTE is defined as the deflection of the unloaded side of a joint or crack, divided by the deflection of the adjacent loaded side, multiplied by 100 percent. To measure LTE, Mn/DOT uses a falling- weight deflectometer (FWD) device. LTE testing has been done several times on the TH 52 project since the installation of the retrofit dowel bars in 1994. Based on measurements taken in October 2006, a range of LTE from 60 to 80 percent was demonstrated by the various test sections. There are some indications from the data that LTE declines more rapidly in cracks retrofit with only three dowels in the outside wheel track of a panel. Additional data will be collected in the near future to confirm any such trend. Despite the low level of LTE (60 percent) in some sections, the cracks were still not appreciably faulting after 13 years of traffic. Perhaps this is not surprising, since the embedded supplemental mid-panel steel across the crack may still be providing resistance to faulting of the crack.

With regards to the various dowel patterns, it seems the performance in terms of ride quality at year 13 is insensitive to the number and length of the retrofit dowel bars as they were designed in this project. The benefit of retrofitting dowels across mid-panel cracks on TH 52 can clearly be seen by observing the faulting that occurred on a number of nearby unsealed mid-panel cracks that had not received retrofit dowel bars (see Photo 4).



**Photo 4. Faulted mid-panel cracks without retrofit dowels on TH 52.**

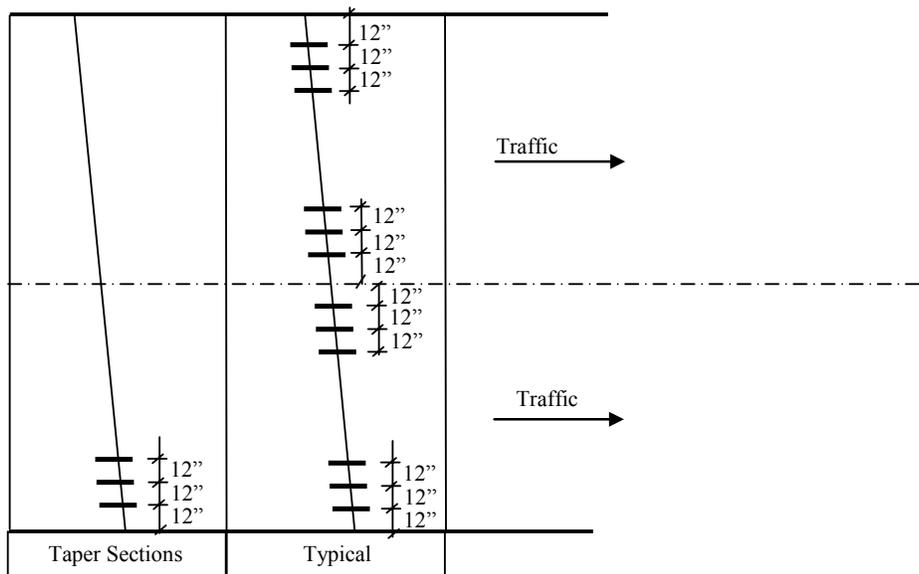
## TH 12, Willmar

### *Project Description*

In 1996, a retrofit dowel bar project was constructed on Minnesota Trunk Highway 12 (TH-12) near Willmar, Minnesota. The pavement on this project was originally constructed in 1981 of jointed plain concrete, 200 mm (8 in.) thick, with panels 4.6 m (15 ft) long and undoweled transverse joints. By 1996, the transverse joints were experiencing substantial faulting in the outside wheel tracks. Some of the joints had openings as wide as 32 mm (1.25 in.). The objective of this project was to determine if retrofit dowel bars and surface grinding could effectively extend the service life of this pavement.

### *Design Variables*

This project was designed to study the performance of retrofit dowel bars installed across significantly faulted transverse joints. The project included two retrofit dowel bar patterns. The layout of the dowel bar patterns is shown in Figure 2. Short sections of tapering lanes contained transitions from three dowel bars in the outside wheel track, to three dowels in both the outer and inner wheel tracks. Otherwise the latter configuration (six dowels) were retrofit into both the driving and passing lanes. Dowel to dowel spacing in the wheel tracks was 300 mm (12 in.). The first dowel from the edge of the driving lane shoulder was placed at 300 mm (12 in.). The slots were formed using a milling machine, rather than the standard method of saw cutting and chipping. All retrofit dowel bars were 38 mm (1.5 in.) diameter by 457 mm (18 in.) long, and installed using Mn/DOT 3U18 patch mix backfill material.



**Figure 2. TH 12 test section retrofit dowel bar patterns.**

### *Observed Performance*

The retrofit dowel bars on TH 12 were installed in 1996, and therefore have been exposed to over 12 years of traffic (HCADT = approximately 600)(6) and weather. Recent site visits took place in October 2006 and February 2007. During those visits, minor surface distress on the retrofit dowel bar slots was noted to occur predominantly near the joints (see Photo 5). The distresses might be linked to the slot milling process, when material near the joint

was weakened. Otherwise, there was little deterioration of the Mn/DOT 3U18 backfill mortar after 10 years of service.

LTE testing was measured for a select number of retrofitted joints in October 2006 and ranged from 51 to 65 percent. Initial LTE measurements from shortly after retrofit dowel installation could not be found, so trends could not be established. Despite the low LTE values, fault measurements recorded at the same time as the LTE testing revealed an average value of only 1.5 mm (0.059 in.). It is suspected that faulting levels may accelerate as the LTE continues to decline.

It appears that the installation of retrofit dowel bars on this project, in conjunction with surface grinding, has successfully enhanced the performance and lengthened the life of the pavement.



**Photo 5. Backfill mortar distress near joints on TH 12.**

## **TH 23, Mora**

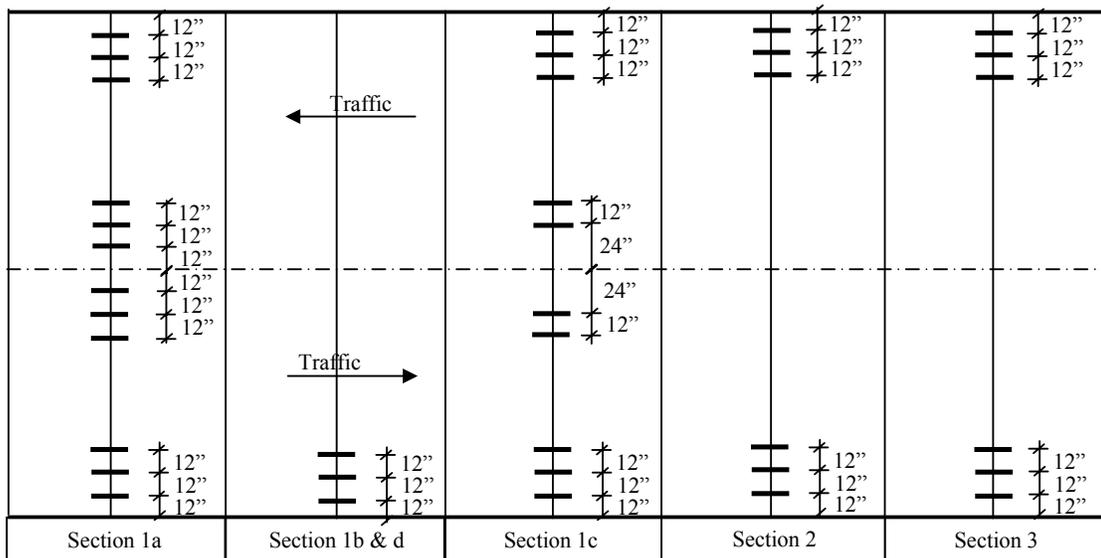
### *Project Description*

Another experimental retrofit dowel bar project was constructed in 1998 on Minnesota Trunk Highway 23 (TH 23) near Mora, Minnesota. The pavement on this project was originally constructed in 1952 of jointed plain concrete with a trapezoidal design of 225–175–225 mm (9–7–9 in.) thick. Panels were 4.9 m (16 ft) long, and the transverse joints were undoweled. After 46 years of traffic, the transverse joints were experiencing minimal faulting in the inside wheel track and substantial faulting in the outside wheel track. The objective of this project was to determine if retrofit dowel bars and surface grinding could effectively extend the service life of this very old pavement.

### Design Variables

The test sections in this project were designed to study the configuration, length, and number of retrofit dowel bars necessary to restore load transfer across (previously undoweled) faulted transverse joints containing minimal aggregate interlock. Two types of backfill mortar material were utilized to compare their load-transfer capability and durability.

The retrofit dowel bar pattern variations in this project included combinations of three, two, or no bars in the wheel tracks for both the driving and passing lanes. The layout of the three test sections is shown in Figure 3. Test Section 1 consists of 160 total joints utilizing a repeating pattern of 20 joints of the sequences 1a, 1b, 1c, 1d as shown in Figure 3. Retrofit dowel bars were 380 mm (15 in.) long and 38 mm (1.5 in.) diameter. The backfill mortar type was Mn/DOT 3U18. Test Section 2 consists of 80 total joints, with 40 joints using rapid set mortar (RSM) backfill, and 40 joints using Mn/DOT 3U18 patch mix. The retrofit dowel bars in this section were 380 mm (15 in.) long and 38 mm (1.5 in.) diameter. Test Section 3 also consists of 80 total joints, utilizing a repeating pattern of 20 joints with 325 mm (13 in.) long retrofit dowel bars, then 20 joints with 380 mm (15 in.) long dowel bars. All of the dowel bars in this section were 38 mm (1.5 in.) diameter and the backfill mortar was Mn/DOT 3U18. Dowel to dowel spacing in the wheel tracks was 300 mm (12 in.). For comparison, a control section containing no retrofit dowel bars, but a diamond ground surface, was monitored as part of the study. Additional information on the layout and construction of TH 23 test sections can be found in a report by Embacher (7).



**Figure 3. TH 23 test section retrofit dowel bar patterns.**

### Observed Performance

The retrofit dowel bar test sections on TH 23 were installed in 1998, and therefore to date, have had over 10 years of exposure to traffic (HCADT = approximately 500)(6) and weather. Recent site visits took place in October 2006, and February 2007. During those visits, surface distress on the test sections was recorded as being minimal. Photo 6 shows retrofit dowel bar slots with Mn/DOT 3U18 backfill mortar in very good condition.



**Photo 6. TH 23 slots with Mn/DOT 3U18 backfill mortar after 8 years.**

LTE testing was measured for a select number of retrofitted joints in October 2006 and ranged from 64 to 80 percent. Fault measurements recorded at the same time as the LTE testing revealed an average value of only 0.5 mm (0.02 in.). It appears that the installation of retrofit dowel bars on this project, in conjunction with diamond surface grinding, has successfully lengthened the performance of this very old pavement.

The performance of the various dowel bar configurations indicates that three dowels in the outside wheel track only is adequate for long-term load transfer across a previously faulted transverse joint (for the traffic level on TH23). The difference in the performance of retrofitting a 380-mm (15-in.) versus a 325-mm (13-in.) dowel bar seems negligible at this time.

Laboratory testing of retrofit dowel bars 325 mm (13 in.) long on the Minne-ALF device by Embacher et al. (4), confirms the possibility of using a shorter, lower cost dowel bar, while still achieving long-term load-transfer performance. Shorter bars also provide the benefit of requiring shorter saw cuts, for additional cost savings.

## **I-90, Beaver Creek**

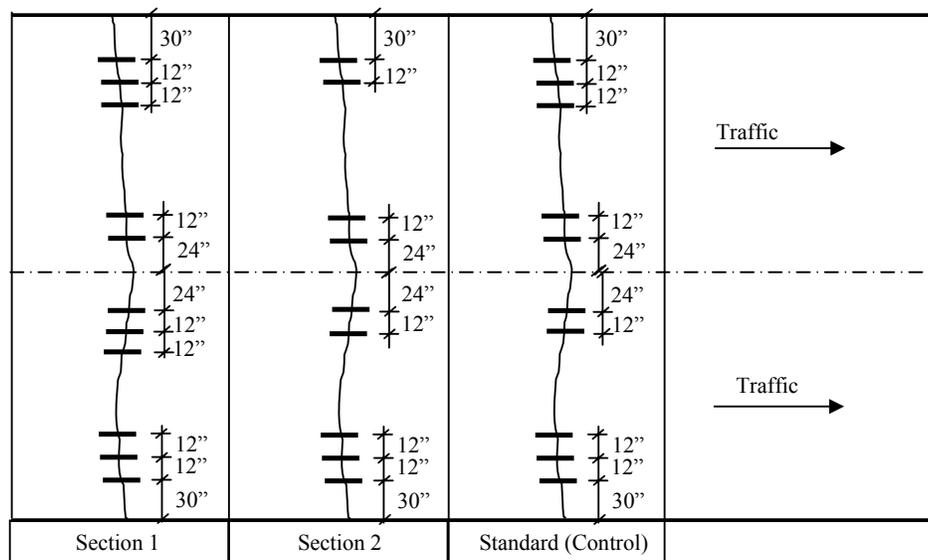
### *Project Description*

In 1999, a retrofit dowel bar project was constructed on Interstate 90 (I-90) near Beaver Creek, Minnesota. The pavement on this project was originally constructed in 1984 of jointed reinforced concrete 225 mm (9 in.) thick. After 15 years of heavy interstate truck traffic, the panels, 8.2 m (27 ft) long, contained mid-panel cracks that demonstrated enough faulting to affect ride quality. The objective of this project was to determine if retrofit dowel bars and diamond surface grinding could restore pavement performance and significantly slow down the redevelopment of faulted mid-panel cracks.

### Design Variables

The test sections in this project were designed to study the configuration and number of retrofit dowel bars necessary to achieve long-term load transfer across faulted transverse mid-panel cracks. The layout of the two test sections is shown in Figure 4. Retrofit dowel bar pattern variations included combinations of two or three bars in both wheel tracks of the driving and passing lanes. Test sections 1 and 2 are each ten panels long (in both eastbound and westbound directions). All retrofit dowel bars were 38 mm (1.5 in.) by 380 mm (15 in.) long. Dowel to dowel spacing in the wheel tracks was 300 mm (12 in.). The first dowel from the edge of the driving lane shoulder was placed at 760 mm (30 in.).

All retrofit dowel bars on this project were installed using a PMQP mortar mixture. It was noted during construction that the existing reinforcing steel in the pavement was causing rapid wear of the saw blades used to form the slots. Therefore, many of the retrofit dowel bars on this project were placed 6 mm (0.25 in.) higher than the standard dimension of half the thickness of the pavement.



**Figure 4. I-90 Test section retrofit dowel bar patterns.**

### Observed Performance

The retrofit dowel bar test sections on I-90 were installed in 1999, and therefore to date, have over 9 years of exposure to traffic (HCADT = approximately 1,200)(6) and weather. Recent site visits took place in October 2006 and February 2007. During those visits, surface distress on the retrofit dowel bars slots was recorded as being minimal. Photo 7 shows retrofit dowel bar slots with the PMQP backfill mortar in very good condition. Most of the current slot distresses are caused by longitudinal or transverse panels cracks that formed after the retrofit dowel installation.

The performance of the various dowel bar configurations indicates three dowels in the outside wheel track and two in the inner wheel track are adequate for long-term load transfer across a transverse mid-panel crack under the traffic loads experienced by I-90. A third dowel bar installed within the inside wheel track of the driving lane does not seem to add benefit to the performance and therefore is not cost beneficial in this case. Also, the performance of the retrofit dowel bars in the passing lane shows that the reduced pattern of

two dowels in each wheel track is adequate to maintain performance for the reduced volume of traffic that this lane carries.



**Photo 7. Typical condition of I-90 slots with PMQP backfill mortar after 7 years.**

The objective of the research for the I-90 test sections was to determine the long-term capability and durability of retrofit dowel bars placed across previously faulted transverse mid-panel cracks. Fault measurements in October 2006 showed an average value of 0.3 mm, indicating that virtually no crack faulting has returned after 7 years of service. LTE tests taken at the same time revealed values ranging from 70 to 89 percent, indicating the retrofit dowel bars are continuing to contribute to load transfer across the crack.

### **Load-Transfer Efficiency Trends**

Throughout each of the projects described previously, it is noted that several retrofit dowel bar patterns and lengths were being tested. Ideally, designers are interested in using the least amount of well-placed retrofit dowel bars, while still maintaining long-term performance. Most designs incorporate as a minimum, three retrofit dowel bars in the outer wheel track. It is believed that loads on the inner wheel track are carried in part by the adjacent lane (if tied with reinforcing steel). The question is whether adding bars in the inside wheel track (if even necessary to begin with) will simply carry local loads, or possibly benefit the bars in the outer wheel path. The only way to know is to monitor the LTE in both wheel paths.

It was discovered in preparation of this paper that, while various LTE tests were conducted at various times throughout the life of each project, the location assignments for individual joints or cracks was inconsistent. In addition, LTE testing of the inside wheel track was never conducted. Therefore, throughout this paper, only ranges of average LTE (based on measurement in the outside wheel track) could be presented. This prevented the determination of individual performance trends related to dowel bar patterns or lengths. It

will be next step in this study to do more comprehensive LTE testing of the current projects, and to decode the past testing results.

Even though specific trends in LTE could not be determined, the observed uniform good performance of the projects in this study show that perhaps some of the design details considered are not as sensitive as once believed. These projects have demonstrated that using fewer and shorter dowel bars (AASHTO 1993 [8] recommends bars 457 mm [18 in.] long) can still provide long-term LTE performance for both joints and mid-panel cracks.

### Ride Quality Trends

While adequate long-term LTE is important to the owners and maintainers of a retrofit dowel bar project, the traveling public is more interested in ride quality. Mn/DOT has a comprehensive pavement management program that frequently measures the pavement distress and ride quality of the entire road network operated by Mn/DOT. Through these measurements, long-term trends are established and pavement performance models can be determined.

For the time period covered in this study, Mn/DOT’s pavement management system rated roads based on what is termed the PQI, or “Pavement Quality Index.” The PQI is calculated based on a number of parameters defined as such:

$$PQI = (SR * PSI)^{1/2} \quad (\text{Equation 1})$$

where PSI = “Present Serviceability Index”, which is a function of the IRI (International Roughness Index), and SR, which is a surface condition rating. Recently Mn/DOT substituted the term Ride Quality Index (RQI) for PSI, but the latter term is retained for convenience in this paper.

Figure 5 shows a performance decay model for the TH 12 project. The rate of decay can be significantly influenced by the residual life and existing condition of the concrete prior to

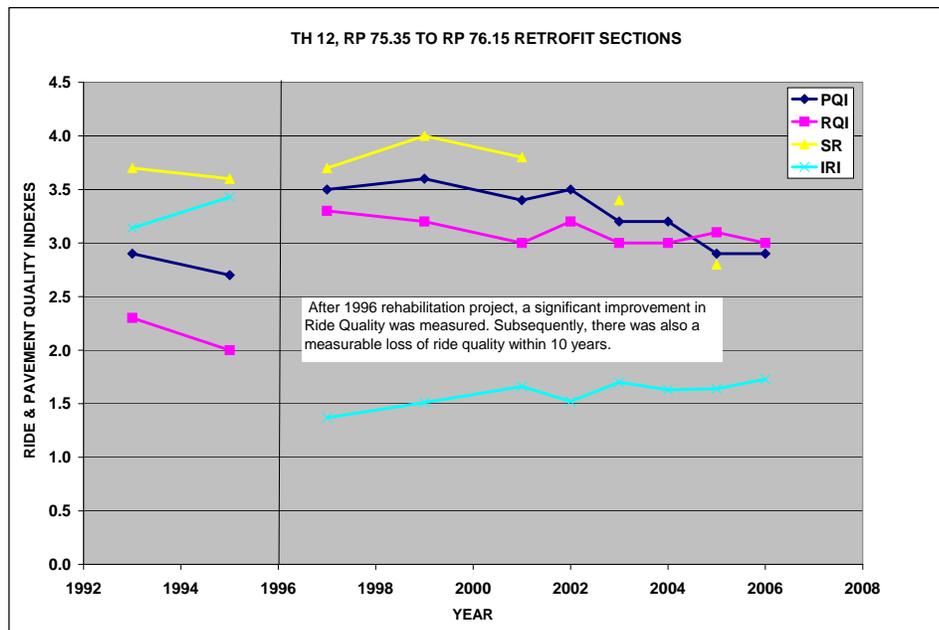


Figure 5. TH 12 pavement quality index trends.

rehabilitation. It is sufficient to note that there was a significant increase in ride quality after the retrofit project was completed, and after 10 years of service, the PQI is still above the value before the retrofit project was done. Unfortunately, the network-level data available for the other projects in this study did not allow for the establishment of similar decay curves at this time.

### **Backfill Mortar Performance**

While the long-term performance of retrofit dowel bar projects is well accepted in terms of improved LTE, the materials used to anchor the bars to the existing pavement have often experienced problems. The projects in this study used various backfill materials to discover which ones could meet both the quick construction time criteria and long-term durability. The extreme climate of Minnesota often presents durability problems for rapid setting, high early strength grouting materials.

Recent site visits to the projects in this study revealed that all the backfill materials have demonstrated long-term strength and durability in Minnesota, when designed and constructed properly. There have been some minor problems with some of the PMQP materials on the TH 52 and I-90 projects, however long-term, the problems seem to be more cosmetic than structural in nature. Ironically enough, Mn/DOT's simple 3U18 repair material, even though not technically a nonshrink grout, has performed very well, despite sometimes displaying shrinkage cracks on the surface.

## **BEST PRACTICES**

### **Development**

Developing best practices for any design or construction feature comes from a combination of empirical field observation, analysis, and laboratory testing. The important issues related to the retrofit dowel bar technique in Minnesota revolve around economical construction and long-term durability in an extreme climate. The current best practices are based on the early performance of the projects in this study, results from laboratory load testing, and experiences learned from other States(9).

The successes and problems experienced on the projects in this study influenced much of the current design for retrofit dowel bars in Minnesota. Retrofit dowel bar slots are sawed, rather than milled, based on construction problems experienced on the TH 12 project. While the milled slots have performed reasonably well, they have experienced more distress than sawed slots. Successful milling or other slot-forming techniques may be developed in the future that could significantly improve construction efficiency and therefore reduce costs.

Minnesota's standard dowel bar length for new concrete pavement construction is 15 in. (380 mm). It was therefore desirable to know whether this length could provide long-term LTE when used in retrofitting joints and cracks. To accomplish this, an accelerated load-testing stand, developed in cooperation with the University of Minnesota, was used to test various retrofit design features (4). Design features tested included the number of retrofit dowels in a wheel track, the length of the dowel bars, and the type of backfill material used. It was determined that not only did dowels 380 mm (15 in.) long perform well, but even shorter bars, 330 mm (13 in.) long, performed just as well. Shorter bars not only reduce dowel cost, but also reduce sawing and chipping costs, due to shorter dowel-slot requirements. It was also demonstrated that two dowel bars in a wheel track can be as efficient long-term as three dowel bars in a wheel track, however joint deflections tend to be

higher. Testing also revealed that Mn/DOT 3U18 patch mix performs just as well as some of the proprietary high-strength backfill mortar products.

### **Project Selection**

When designing a joint or crack load-transfer restoration project utilizing retrofit dowel bars, the most important aspect is project selection. Mn/DOT currently expects a retrofit dowel bar project to increase the service life of a pavement by 12 to 15 years. Based on that expectation, Mn/DOT's recommendations when choosing a retrofit dowel bar project is that "the concrete is structurally sound and the main deficiency of the pavement is load transfer." This means the concrete near the transverse joints or mid-panel cracks must be capable of being sawed and chipped without growing in size due to cracked or weak material. Also, if faulting is significantly affecting the ride quality, it is recommended that diamond grinding or surface planing be done after the retrofit dowel bar installation. Grinding the pavement afterwards also allows for quicker retrofit dowel bar installation, since the backfill material in the slots does not need to be finished as smoothly.

### **Dowel Bar Placement**

While much experimentation has been done with regard to the number and placement of retrofit dowel bars, Mn/DOT's experiences have led to the following recommendations. For two-lane roads with joint or mid-panel crack faulting predominantly near the outside edge of the pavement (near the shoulder), three retrofit dowel bars placed in the outside wheel track work well. For two-lane roads with joint or crack faulting across the entire lane width, three retrofit dowel bars placed in the outside wheel track and two dowels placed in the inside wheel track are the recommended option. For divided, multilane highways, the pattern of retrofit dowel bars should be based on the type of faulting that occurred. Full-lane-width faulting should be retrofit with three retrofit dowel bars placed in the outside wheel track and two or three dowels placed in the inside wheel track. Similar to two-lane roads, joints or cracks with faulting only near the shoulder should be retrofit with three dowels placed in the outside wheel track only. Since there is no rational design method established for retrofit dowel bars, design decisions should be based on the desired amount of heavy-truck traffic applications a particular lane will carry in the future.

Transverse alignment of the dowel bars within a lane can be flexible, however, the best performance is achieved when the center of a dowel bar group is centered in the wheel track. Dowel-to-dowel transverse spacing is recommended to be 305 mm (12 in.). When placed across transverse mid-panel cracks, retrofit dowel bars should be aligned longitudinally such that equal embedment length is realized.

### **Dowel Bar Design**

Mn/DOT recommends epoxy-coated plain steel dowel bars 32 mm (1.25 in.) in diameter by 380 mm (15 in.) for retrofit dowel bar projects. Although shorter 330 mm bars, (13 in.) long, have been shown to work for retrofit dowel projects, 380-mm (15-in.) bars are very commonly used in conventional concrete pavement projects throughout Minnesota.

Each retrofit dowel bar is fitted with plastic end caps that allow for 6 mm (0.25 in.) of expansion movement at each end of the bar. They also contain nonmetallic support chairs that provide a minimum of 13 mm (0.5 in.) of clearance between the bottom of the dowel bar and the floor of the slot in the pavement. A compressible foam core board is also placed at the midpoint of the dowel to reestablish the joint or crack. Photo 8 shows Mn/DOT approved retrofit dowel bar assemblies.



**Photo 8. Mn/DOT approved retrofit dowel bar assemblies.**

### **Backfill Materials**

Mn/DOT’s special provisions to the specifications(10) state that: “The Contractor shall use a ‘Mn/DOT Approved Non-Shrink Rapid Set Concrete Material for Dowel Bar Retrofit Repairs’ conforming to ASTM C 928 and the requirements on file in the Mn/DOT Concrete Engineering Unit. This material may be extended with CA-80 as specified by the manufacturer, as backfill material.” The Mn/DOT CA-80 designation represents an aggregate gradation as shown in Table 1.

**Table 1  
Gradation Specifications for Mn/DOT Designation CA-80 (Mn/DOT 2005)**

<b>Aggregate Designation</b>	<b>Percent by Mass (weight) Passing Square Opening Sieves</b>	
	<b>9.5 mm (3/8 in.)</b>	<b>4.75 mm (#4)</b>
CA-80	100	55-95

Note: Not more than 5 percent shall pass the 300 um (#50) sieve.

Although not specifically designed to be a nonshrink retrofit dowel bar backfill material, Mn/DOT’s 3U18 material has nevertheless performed very well in Minnesota when used for concrete rehabilitation repairs. The designation 3U18 can be decoded as follows:

3 = Air-entraining concrete for durability in freezing conditions,

U = Refers to cementitious content of the mix, where  $U \geq 800$  lb cement per cubic yard of concrete,

1 = Refers to slump level in inches,

8 = Descriptor for the aggregate gradation, which is the same as CA-80 (Table 1).

Despite some shrinkage problems observed along retrofit slot faces, Mn/DOT’s 3U18 material demonstrates long-term strength and durability when used in retrofit dowel bar projects. Other States (11) have experimented with Mn/DOT 3U18 material for retrofit projects and report similar findings.

Several other proprietary, nonshrink, rapid-setting materials have been, and continue to be, successfully used for retrofit dowel bar projects in Minnesota.

## Construction Method

Historically, the challenges faced during the installation of retrofit dowel bars have included the following:

- *Uniformity of placement.* The installation is often done quickly, and rapid-setting backfill materials can be improperly handled.
- *Avoidance of honeycombing and entrapped air.* The strength requirements for nonshrink grouts do not provide much tolerance for honeycombing.
- *Avoidance of unconsolidated concrete under the dowels.* With such small volumes of backfill material available to bond the dowel to the existing pavement, voids can significantly reduce the available load-transfer capacity of the retrofit dowel bar.
- *Compatibility at the concrete interface.* Retrofit slots must be rough and clean for the backfill materials to be able to bond. Also, the grout cannot shrink to the extent the bond with the slot is lost.
- *Vertical or horizontal misalignment.* Retrofit dowel bars must be aligned properly or they might cause binding of the joint, which may lead to further pavement distress.

To ensure that a contractor will be able to follow the recommended best construction practices on a retrofit dowel bar project, Mn/DOT requires that a small test section be constructed before commencing with a project. The details of this test section are summarized as follows:

1. There will be 24 retrofit dowels in the test operation.
2. Twenty-four hours after completion of the test operations, the Contractor shall take three 150 mm [6 in.] diameter full depth cores as directed by the Engineer to determine the completeness of the removal and installation operations.
3. If the placement is in accordance with the Plans, the Mn/DOT Standard Specifications, and these Special Provisions; and if the Contractor's retrofitting operation has not damaged the surrounding in place concrete; then, upon approval from the Engineer, the Contractor may begin production operations and shall proceed on a performance basis.
4. The work in this paragraph shall be paid at the unit price for dowel bar retrofit.

The (minimum) three core samples described above are taken to monitor the following:

- One core is taken through the dowel to observe that there is continuity at the dowel/concrete interface and that there is consolidation beneath the dowel.
- One core is taken over the dowel supports to assure that there was no collapse that could result in vertical or horizontal misalignment.
- One core is taken to include both the retrofit backfill mortar and the existing concrete, to assure that compatibility and continuity exists at the interface of new and existing concrete.

Any problems identified during the examination of the cores must be addressed by the contractor before a project can proceed. One example of such problems on a project is described next.

A retrofit dowel bar project was constructed in 1994 on Minnesota TH 59. A contractor test site was constructed and the specified cores were taken and tested. Many of the core samples did not attain the required ASTM 928 prescribed strength. The cores also revealed severe honeycombing. This is shown in Photo 9. These results necessitated a switch from one approved backfill product to another.



**Photo 9. Core from TH 59 test site showing honeycombed retrofit mortar.**

The typical steps involved in the installation of retrofit dowels bars in Minnesota are as follows:

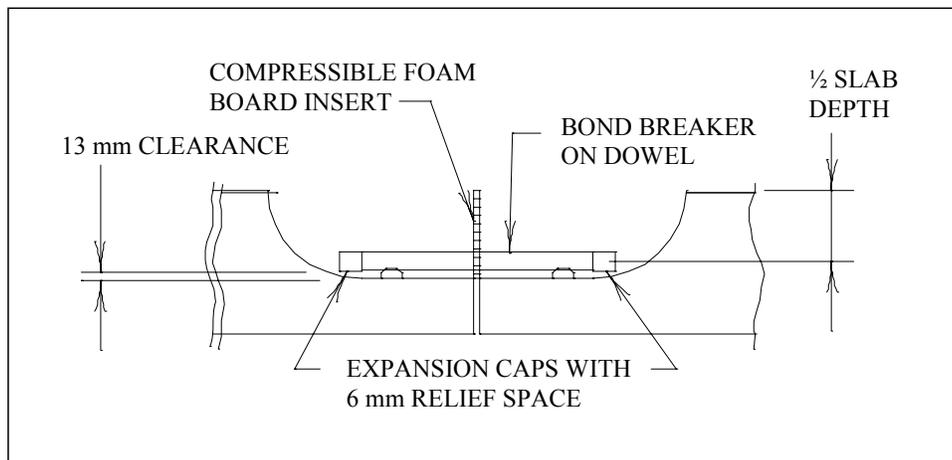
1. *Saw the slot for each dowel bar.* This is often accomplished by “gang saws,” which provide cuts for multiple parallel slots at one time. Typical slot dimensions for a dowel 380 mm (15 in.) long are 650 mm (26 in.) long by 65 mm (2.5 in.) wide at the surface of the pavement, and 400 mm (16 in.) long by 65 mm (2.5 in.) wide at the bottom of the slot. Slots for mid-panel cracks may be longer to accommodate skew in the crack.
2. *Remove concrete to form kerf and rinse and dry.* Material between the saw cuts is typically removed using a chipping air hammer, limited to a weight of 13.5 kg (30 lb) or less. The bottom of the slots should be sufficiently cleaned to allow the dowel bar assembly to sit parallel to the pavement surface. Any water residue or saw slurry created by removal operations shall be contained and vacuumed immediately from the road surface.
3. *Sand blast and vacuum clean slot.* Sand blasting removes any loosened material and roughens the side faces of the slot. All exposed surfaces and cracks are then further cleaned with a “moisture and oil free” high-pressure air-blasting lance (690 kPa

[100 lbf/in<sup>2</sup>] minimum pressure) immediately before beginning the sealing work. The contractor must protect nearby traffic from sand and air blasting operations.

4. *Seal bottom and sides of slot.* Prior to placement of the dowel bar and filler material, the contractor must seal the existing transverse joint or crack within the slot. Mn/DOT approved silicone joint sealant material is applied and allowed to cure for a minimum of 2 hours or until it is tack free (according to the manufacturer's recommendations) prior to placing the backfill material. This is done in a manner sufficient to keep the backfill material from leaking into the joint or crack.
5. *Place and align dowel bar and joint/crack filler material.* Just prior to installation, dowel bars are lightly coated with an approved release agent, fitted with the compressible foam or filler material, support chairs, and the expansion caps on both ends. The dowel bars are then placed in the slots, parallel to the centerline and pavement surface, all to a tolerance of 3 mm (0.125 in.). Figure 6 depicts a side view of a typical retrofit dowel bar within a slot. Photo 10 shows actual retrofit dowel assemblies placed in the slots prior to backfilling.
6. *Place backfill material, finish, and cure.* Approved non-shrink rapid set concrete material is placed within the slot. To ensure that backfill material fully surrounds the dowel and fills remaining voids in the slot, a "spud" type vibrator is inserted into the material prior to surface finishing.

The slots are finished flush and smooth if surface grinding is not scheduled before traffic is allow on the pavement. Immediately after final finishing, all retrofit slots are coated with an approved membrane curing compound. Water based curing compounds are not permitted.

Public or contractor traffic is not permitted on the newly placed backfill material until adequate strength has been achieved, according to the manufacturer's recommendations. If shrinkage cracks occur in the repair shortly after placement, the work is rejected and completely redone at the contractor's expense.



**Figure 6. Side view of typical retrofit dowel bar in slot.**



**Photo 10. Retrofit dowel assemblies placed in the slots prior to backfilling.**

7. *Seal joint or crack between slots.* Mn/DOT approved silicone joint sealant material is used to fill the joint or crack between the retrofit slots and any other areas disturbed by the construction process. Often the entire joint or crack within a lane is cleaned and filled as a standard preventative maintenance measure.
8. *Diamond grind or plane surface (optional).* Although this procedure is optional, it is very common to diamond grind or plane the surface of the pavement after retrofitting dowel bars. Typically retrofit projects are performed on pavement with significantly faulted joints or cracks that are impacting ride quality. Diamond grinding or surface planing not only restores ride quality, but also ensures that overbands of retrofit mortar and other sources of unevenness are removed and a uniform texture is imparted.

## **CONCLUSIONS**

Retrofitting dowel bars into a distressed and faulted concrete pavement has become a proven technique for restoring or improving the capacity of jointed concrete pavements. The performance of this technique relies on proper project selection and the successful application of design features including dowel bar design and placement, backfill materials, and construction methods. Improved knowledge toward understanding the long-term performance of such techniques, especially in severe climate regions, is always of great interest. In this study, the successful long-term performance of four retrofit dowel bar projects in Minnesota has been described. After 14 years of heavy traffic and extreme weather, the mid-panel cracks on the TH 52 project still have an LTE ranging from 60 to 80 percent. Similar LTEs were found on retrofit dowel bar joints after 10 years of service on TH 23, a previously undoweled pavement now over 56 years old. Despite minor cosmetic problems, all of the retrofit dowel bar backfill materials used in the projects in this study are performing well in the Minnesota climate. The experiences from these projects, coupled with accelerated laboratory testing results, have resulted in the development of successful best practices for retrofit dowel bars in Minnesota. Retrofit dowel bar installation, in conjunction with restoration of the surface through diamond grinding, has been proven to significantly extend the capacity and serviceable life of many concrete pavements in Minnesota.

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## DISCLAIMER

The contents and opinions presented in this paper are those of the authors, who are responsible for the facts and accuracy of the data. The contents do not necessarily reflect the views or opinions of the Minnesota Department of Transportation.



# Life Cycle Cost Analysis of Dowel Bar Retrofit

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## ABSTRACT

The California Department of Transportation (Caltrans) has used dowel bar retrofit (DBR) on several projects. Caltrans has experienced both success and problems with this pavement preservation method. The primary question at the end is: if DBR performs as expected, is it the most cost-efficient solution? This presents the results of a Life Cycle Cost Analysis (LCCA) project comparing DBR with grinding and asphalt overlay. The performance assumptions were based on observed performance in the field and under heavy-vehicle simulator loading. Costs were collected from industry and Caltrans construction cost records. The analysis assumed the typical Caltrans practice of using nighttime closures to minimize road user delay. The analysis was performed using Caltrans LCCA procedures based on use of the Federal Highway Administration's (FHWA's) software RealCost. This study used a 40-year analysis period. It fits the planning horizon for the activities considered and meets the recommendations of the FHWA. Sensitivity analysis was performed considering these variables:

*Initial remaining life:* This takes into account the structural condition of the pavement that is a candidate for DBR. The analysis considered 10, 20, and 30 years of expected fatigue life remaining.

*Grinding life:* This captures scenarios for the interval between grinding in the absence of DBR. The analysis considered 10, 12, 15, 17, and 20 years.

*User cost variables:* These include traffic growth, closure details (time of day/week, number of lanes affected) and traffic distribution (rural versus urban, percentage of trucks). For this analysis, all closures were considered to be on weeknights from 10:00 p.m. to 6:00 a.m. and to affect only one lane of traffic. The chosen annual growth rate was 1.5 percent.

*DBR performance:* To account for the uncertain maintenance cost of DBR (due to failed backfill material), analyses in this study were run using a failure rate of 0 percent, 3 percent, and 6 percent per year. Results were also produced for the cases of plus/minus 10 percent from the expected DBR initial cost.

*Discount rate:* A discount rate of 4 percent for LCCA was used, as typically is done by Caltrans.

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The comparison was based on a 5-mi (8-km) rural stretch of highway with an initial annual average daily traffic load of 38,500 vehicles, 24 percent of which were trucks, loosely based on a DBR site on Route 99 in Kern County. The final results of the LCCA are relatively unaffected by the details of the case study. The analysis considers the possibility of additional fatigue life from DBR. Although the *Mechanistic-Empirical Pavement Design Guide* does not show increased transverse fatigue life from DBR, RadiCal predicts increased longitudinal fatigue life. Longitudinal cracking is common in dry western environments.

## **INTRODUCTION**

The use of dowels in California concrete pavements began in 1999. Before that year nearly all portland cement concrete (PCC) pavements were undoweled jointed plain concrete pavements (JPCP). Previous testing confirmed that new construction using doweled pavements provides load transfer at the transverse joints between slabs that is superior to that of undoweled pavements, including when cemented bases are used (Reference 1 and others cited within it). To improve the load transfer between slabs for existing undoweled JPCP, a technique called dowel bar retrofit (DBR) can be used to install dowels on existing pavements. DBR can increase the faulting life of the pavement and eliminate the need for future grinding to restore ride quality. DBR may also increase fatigue cracking life. However, widespread use of DBR has been halted because of its large initial construction cost and the marginal performance history of the backfill material on several California pavement sections.

The work presented in this paper was performed as part of a project originally proposed in 2000 by the Caltrans Headquarters Division of Design. Other Caltrans divisions participating in oversight of the project included Headquarters METS Office of Rigid Pavement Materials and Structural Concrete, as well as Caltrans Districts 1 and 7.

This research is intended to provide Caltrans with information needed for decisions regarding selection of DBR and its design and construction, in order (1) to help determine where DBR may be a cost-effective strategy for rehabilitating rigid pavement and (2) to help obtain best performance where DBR is selected as the preferred rehabilitation strategy.

### **Scope**

This paper assesses the cost-effectiveness of DBR compared to traditional capital maintenance (CapM) and rehabilitation techniques for concrete pavements. A life-cycle cost analysis (LCCA) tool used by Caltrans is employed for this assessment. With data from industry, Caltrans, and academic sources, costs are estimated for both Caltrans and users, under varying conditions. User costs are calculated using the Federal Highway Administration (FHWA) software package called RealCost and the Caltrans *Life-Cycle Cost Analysis Procedures Manual*, which was developed by Caltrans and the University of California Pavement Research Center (UCPRC).

### **Objectives of LCCA Modeling**

This paper completes three main objectives. Each will help determine the overall cost-effectiveness of DBR.

- Determine the agency's life-cycle costs for DBR and its alternatives.

- Determine the users' life-cycle costs for DBR and its alternatives.
- Estimate the extension of fatigue life needed for DBR to become the preferable alternative, based on economic value.

The first two objectives are straightforward. The third objective is more complex, as it stems from the unclear relationship between DBR and fatigue life. It is thought that dowels may increase the fatigue life of concrete pavement by restraining slab curling caused by vertical temperature gradients, and by reducing stresses in the slab caused by dynamic interaction of truck suspensions with faulting at the joints caused by poor load transfer. However, these potential effects are not captured in available mechanistic-empirical models, and the range of potential fatigue life extension is therefore unknown. Because this information is unavailable, the third objective is to estimate the percentage of fatigue life that must be added by DBR for it to become a cost-effective CapM solution.

### **Potential Impact**

It is important to estimate the number of lane-miles of California pavements suitable for DBR. If only a handful of sections are candidates for this technique, then further research into the cost-effectiveness of DBR is unwarranted. To estimate the number of lane-miles, candidate projects must be filtered out from the total lane-miles of concrete pavement in the State highway system. A basic criterion for candidate pavement sections is that they consist of undoweled JPCP with no overlay. Candidate sections must also have a minimal amount of cracking, as discussed later in this paper, with regard to the remaining fatigue life of the slabs. Using data from the Caltrans Pavement Condition Survey and the Office of Construction Engineering, it was estimated there are some 8,700 lane-mi (14,001 lane-km) of pavement that can be considered for DBR. The derivation of this value is shown later in the paper.

### **VARIABLES**

This section presents a broad overview of the variables that can affect the LCCA results. While a complete list of variables is much longer than appears in this section, the following variables will significantly influence the results or are important to describe in detail. Also included are some assumptions used in the analyses.

#### **Initial Remaining Fatigue Life**

The remaining fatigue life of the PCC pavement plays a pivotal role in determining the cost-effectiveness of DBR treatment. While a DBR joint is designed to last indefinitely, it is useful only until the PCC pavement fails. When this happens, the investment of DBR is lost. It is assumed that a doweled pavement structure offers no benefit to subsequent overlays, as new undoweled cracks will be the weakest elements. This is especially true for the crack, seal and overlay (CSOL) strategy assumed in this study. For this reason, pavement sections with long remaining fatigue lives will reap greater benefits from DBR.

LCCA was completed for pavements with 10, 20, and 30 years of expected fatigue life remaining (before DBR). Because doweled JPCP became a standard design in California in the early 2000s, and most undoweled concrete pavements were constructed between 1955 and 1980, undoweled JPCP sections with over 30 years of remaining fatigue life are rare in the State highway system.

## Grinding Life

Another variable is the time period between subsequent grinding treatments. The increase in load transfer efficiency provided by DBR will reduce faulting to a negligible amount. Therefore, pavement grinding will become generally unnecessary after DBR. Assessing the financial benefit from this requires knowing the number of grinding treatments that DBR will avoid over the life of the PCC. For example, if grinding without DBR is expected to last 12 years on a given pavement segment and 30 years of fatigue life are left in that pavement, then two pavement grindings (in Years 12 and 24) will be avoided by DBR.

Grinding life is sensitive to the pavement structure, traffic, the climate, and the International Roughness Index (IRI) trigger level. Because of the many variables involved, it is difficult to provide an “average” expected life for grinding. Several previous studies have attempted to provide a single value (3, 4), but they included extrapolations and low confidence levels. Therefore, rather than adopt those values for this life-cycle cost analysis, the UCPRC used the Mechanistic-Empirical Pavement Design Guide (MEPDG) software to develop new estimates based on variables specific to California. The results from the initial estimates are shown in Table 1. PCC slab thicknesses of 8 in. and 9 in. (200 and 225 mm) were assumed since nearly all undoweled concrete pavements in the State were constructed with one of those design thicknesses.

Caltrans’ acceptable IRI level (or “trigger” level in the Caltrans Pavement Management System) before a pavement section requires maintenance is considerably higher than that of FHWA. On California highways, an IRI of less than 213 in/mi (3.37 m/km) is acceptable, whereas national levels generally mandate only an IRI of 160 in/mi (2.53 m/km) or less. The difference between these values results in grinding life that lasts twice as long in California as in other parts of the Nation, depending on the definition of failure in terms of ride quality. This does not mean that grinding is more effective in California, but only that a higher level of roughness is allowed before the pavement is expected to require treatment.

Because grinding life is affected by traffic, PCC thickness, and climate, the LCCA uses variable grinding lives of 10, 12, 15, 17, and 20 years. This range includes the expected grinding life of 17 years, as stated in a previous report to Caltrans (3).

**Table 1**  
**Estimates of Grinding Lives from MEPDG**

PCC Thickness (in.)	Yearly Traffic (ESALs)	Climate*	Years for PCC to Reach Faulting	
			0.16 in. (IRI ~ 160 in/mi)	0.26 in. (IRI ~ 213 in/mi)
8	0.025 x 10 <sup>6</sup>	Central Coast	60+	60+
		Desert	60+	60+
		Low Mountain	60+	60+
	0.5 x 10 <sup>6</sup>	Central Coast	55	60+
		Desert	40	60+
		Low Mountain	36	60+
	2 x 10 <sup>6</sup>	Central Coast	14	27
		Desert	10	23
		Low Mountain	9	22
	5 x 10 <sup>6</sup>	Central Coast	6	11
		Desert	4	9
		Low Mountain	4	9
9	0.025 x 10 <sup>6</sup>	Central Coast	60+	60+
		Desert	60+	60+
		Low Mountain	60+	60+
	0.5 x 10 <sup>6</sup>	Central Coast	60+	60+
		Desert	60+	60+
		Low Mountain	54	60+
	2 x 10 <sup>6</sup>	Central Coast	20	38
		Desert	16	34
		Low Mountain	14	31
	5 x 10 <sup>6</sup>	Central Coast	8	15
		Desert	6	14
		Low Mountain	6	13

1 in. = 25.4 mm; 1 in/mi = 15.8 mm/km

\*Note: Central Coast climate taken from weather station at San Francisco International Airport. Desert and Low Mountain climates are from Daggett and Santa Rosa, respectively.

### Analysis Period

Several elements of a project influence the analysis period selected. First, the period should correspond with the planning horizon of the decision-making agency. Second, the period should allow for as many future maintenance and rehabilitation activities as reasonable. Finally, the period should be the same for all scenarios.

This study used a 40-year analysis period, which fits the above criteria and meets the FHWA's recommendations in its *Economic Analysis Primer (5)*. Changing the analysis period may have a significant impact on the results. The analysis period should meet the needs of a specific project. Because this research is aimed at estimating the overall cost-effectiveness of using DBR, an analysis period of 40 years was chosen, to represent an average situation.

## User Cost Variables

A number of variables affect only the user cost calculations. Among these, traffic growth has the most significant influence on the analysis. Because the analysis period is 40 years, even a moderate yearly increase will sharply inflate traffic levels. For example, a modest growth of 2 percent per year for annual average daily traffic (AADT) will more than double the AADT over 40 years. Traffic growth significantly affects user costs when the capacity of the highway is reached during construction, thus creating a traffic queue. User costs have a tendency to increase precipitously as these queues develop. However, it is unrealistic to assume that moderate traffic growth will be sustained over the course of the analysis period without major enhancements being made to the capacity of the pavement section. In all likelihood, increased traffic demand will be curbed by adding a lane to the section, by creating an alternate route, or by a naturally occurring slower growth of traffic each year.

Two other notable variables that affect the user costs are the closure details (time of day/week, number of lanes affected) and traffic distribution (rural versus urban, percentage of trucks). For this analysis, all closures are considered to be on weeknights from 10:00 p.m. to 6:00 a.m. and to affect only one lane of traffic.

## DBR Maintenance

The performance of DBR projects has achieved limited success in California. Applied Research Associates (ARA) in 2004 (6) and the UCPRC in 2007 surveyed the condition of selected projects and found that the grout in many of the backfilled DBR slots has failed since being installed. In the surveyed projects some 1.2 percent to 5.9 percent of their slots were found to have failed each year. While the slots can be fixed by removing the failed grout, resetting the dowel bar, and regrouting, doing so is a relatively expensive process that costs \$10 to \$15 per slot (6). Extrapolated over a 10 lane-mi (16-km) DBR project, maintenance could run up to \$25,000 per year on that section. Moreover, when a DBR slot has severely failed, it may no longer be providing the increased load transfer efficiency between PCC slabs. The damage to the pavement, though not quantified economically in this research, will (in theory) increase the life-cycle costs of the pavement section.

California's experience with DBR is unlike that of other States. DBR projects elsewhere in the United States have required little to no repairs during their life cycles (2, 7, 8). The reason for the disparity between California's and other States' experiences is unclear, though contractor inexperience or inadequate grout materials are possible causes.

To account for the uncertain maintenance cost of DBR, analyses in this study were run using a failure rate of 0 percent, 3 percent, and 6 percent per year. This enables conclusions to be made about ideal, average, and worst-case scenarios, respectively. More information regarding the ARA and UCPRC condition surveys of the DBR sites, and estimation of failure rates, is given in references (2, 9).

## Discount Rate

Choosing an appropriate discount rate for the LCCA is crucial. Studies have shown LCCA to be highly sensitive to the chosen discount rate (10). While Caltrans has commonly adopted a discount rate of 4 percent for LCCAs (10), rates ranging from 3 percent to 10 percent are not un-

common for LCCA of pavement infrastructure. The 4 percent discount rate used by Caltrans is a reasonable value, but it should be noted that this rate has a significant impact on the results. For this LCCA, raising the discount rate would make DBR a less suitable alternative because the activities and user delays that retrofitting prevents are discounted to a lower net present value (NPV). Even small changes to the 4 percent discount rate could dramatically affect the LCCA results. External fiscal factors, such as any anticipated inflation differences between alternatives and the rate of return on investments, should also be considered (5).

## **DATA AND METHODOLOGY**

To analyze the cost-effectiveness of DBR under particular conditions, each permutation in the analysis is compared to an equivalent “base case” that does not employ the DBR strategy. The Caltrans *Life-Cycle Cost Analysis Procedures Manual* includes maintenance and rehabilitation (M&R) schedules for concrete pavements, giving base-case scenarios. These schedules can be altered to include DBR activities, resulting in customized DBR scenarios.

### **Activities and Schedules**

The scenarios consist of the following activities:

- DBR—Insertion of dowel bars into existing undoweled JPCP.
- Grind—Diamond grinding of the PCC surface to remove faulting and other surface distresses.
- CPR-A—CPR on pavements where the total number of slabs that exhibit third-stage cracking or have been previously replaced is between 5 percent and 7 percent.
- CPR-A (NG)—Same as CPR-A, but without diamond grinding.
- CPR-B—CPR on pavements where the total number of slab that exhibit third-stage cracking or have been previously replaced is between 2 percent and 5 percent.
- CPR-B (NG)—Same as CPR-B, but without diamond grinding.
- CPR-C—CPR on pavements where the total number of slab that exhibit third-stage cracking or have been previously replaced is less than 2 percent.
- CPR-C (NG)—Same as CPR-C, but without diamond grinding.
- CSOL—Crack, seat, and overlay consisting of 0.10-ft (30-mm) hot-mix asphalt (HMA) over pavement-reinforcing fabric over 0.35-ft (107-mm) HMA.
- HMA (0.10 ft)—HMA overlay of 0.10-ft (30-mm) thickness.
- HMA (0.25 ft)—HMA overlay of 0.25-ft (76-mm) thickness.
- 2 percent DO—Digout of distressed portion equivalent to 2 percent of the section
- 5 percent DO—Digout of distressed portion equivalent to 5 percent of the section

CPR, which stands for “concrete pavement rehabilitation,” consists of diamond grinding, slab replacement, spall repair, and joint seal repair.

From the above list of activities, M&R schedules were created for pavements with 10, 20, and 30 years of remaining fatigue life, for both the base case and the DBR alternative. Table 2 and Table 3 show these schedules.

**Table 2**  
**M&R Schedules for a Base Case (No DBR) and 10-Year Grinding Life**

10-Year Fatigue Life		20-Year Fatigue Life		30-Year Fatigue Life	
Yr	Activity	Yr	Activity	Yr	Activity
0	CPR-B	0	CPR-C	0	Grind
5	CPR-A (NG)	10	CPR-B	10	CPR-C
10	CSOL	15	CPR-A (NG)	20	CPR-B
28	HMA (0.10')	20	CSOL	25	CPR-A (NG)
33	HMA (0.10') + 2% DO	38	HMA (0.10')	30	CSOL
38	HMA (0.10')	43	HMA (0.10') + 2% DO	48	HMA (0.10')
43	HMA (0.25') + 5% DO	48	HMA (0.10')	53	HMA (0.10') + 2% DO

**Table 3**  
**M&R Schedules for DBR**

10-Year Fatigue Life		20-Year Fatigue Life		30-Year Fatigue Life	
Yr	Activity	Yr	Activity	Yr	Activity
0	DBR + CPR-B	0	DBR + CPR-C	0	DBR + Grind
5	CPR-A (NG)	10	CPR-B (NG)	10	CPR-C (NG)
10	CSOL	15	CPR-A (NG)	20	CPR-B (NG)
28	HMA (0.10')	20	CSOL	25	CPR-A (NG)
33	HMA (0.10') + 2% DO	38	HMA (0.10')	30	CSOL
38	HMA (0.10')	43	HMA (0.10') + 2% DO	48	HMA (0.10')
43	HMA (0.25') + 5% DO	48	HMA (0.10')	53	HMA (0.10') + 2% DO

To account for the sensitivity to variable grinding lives, four additional base cases were created for the scenario having 30 years of remaining fatigue life. These four cases are shown in Table 4. Each of these can be equitably compared to the “30-Year Fatigue Life” scenario in Table 3. The 30-year scenario was chosen because of the hypothesis that pavements with longer fatigue lives hold more promise for cost-effective DBR implementation.

Pavement grinding could be viewed as potentially decreasing the service life of a PCC pavement by reducing slab thickness. Although grinding reduces the thickness minimally, the MEPDG model was used to investigate to what extent a decrease in PCC thickness could reduce the pavement’s fatigue life. For example, if a grinding activity removes 0.25 in. (6 mm) of material from an 8-in. (203-mm) PCC slab (a 3 percent material loss), the loss of fatigue life is roughly 10 percent to 30 percent. These results were obtained using the MEPDG software to predict cracking for slabs of different thicknesses.

**Table 4**  
**M&R Schedules for Base Case (No DBR) and 30-Year Remaining Fatigue Life**

<b>12-Year Grinding Life</b>		<b>15-Year Grinding Life</b>		<b>17-Year Grinding Life</b>		<b>20-Year Grinding Life</b>	
<b>Yr</b>	<b>Activity</b>	<b>Yr</b>	<b>Activity</b>	<b>Yr</b>	<b>Activity</b>	<b>Yr</b>	<b>Activity</b>
0	Grind	0	Grind	0	Grind	0	Grind
10	CPR-C (NG)						
12	Grind	15	Grind	17	Grind	20	CPR-B
20	CPR-B (NG)	20	CPR-B (NG)	20	CPR-B (NG)	25	CPR-A (NG)
24	Grind	25	CPR-A (NG)	25	CPR-A (NG)	30	CSOL
25	CPR-A (NG)	30	CSOL	30	CSOL	48	HMA (0.10')
30	CSOL	48	HMA (0.10')	48	HMA (0.10')	53	HMA (0.10') + 2% DO

In practice, it is believed that the first grinding does not reduce thickness by more than 0.10 to 0.15 in. (about 2 to 3 mm) when taken as average over the entire slab. To test this, three levels of fatigue life loss rates (10 percent, 20 percent, and 30 percent) resulting from grinding are analyzed using grinding lives of 10, 15, and 20 years. These scenarios appear in Table 5, Table 6, and Table 7. All the scenarios are based on a PCC pavement with 30 years of remaining fatigue life. It is assumed that the initial grind (Year 0) does not decrease the fatigue life and therefore that only subsequent grindings affect the pavement's structural integrity. This is a reasonable assumption, because the comparison DBR case (30 years of fatigue life remaining—see Table 3) does not model the loss of fatigue life from grinding that is performed at the time of the DBR. Thus, neglecting the life lost from the initial grind at Year 0 creates an equitable comparison framework for the ensuing analyses.

**Table 5**  
**M&R Schedules for Loss of Fatigue Life With 10-Year Grinding Life**

<b>10% Fatigue Life Lost From Grinding</b>		<b>20% Fatigue Life Lost From Grinding</b>		<b>30% Fatigue Life Lost From Grinding</b>	
<b>Yr</b>	<b>Activity</b>	<b>Yr</b>	<b>Activity</b>	<b>Yr</b>	<b>Activity</b>
0	Grind	0	Grind	0	Grind
10	CPR-C	10	CPR-C	10	CPR-C
19	CPR-B (NG)	18	CPR-B (NG)	17	CPR-B (NG)
20	Grind	20	Grind	20	Grind
23	CPR-A (NG)	22	CPR-A (NG)	21	CPR-A (NG)
27	CSOL	25	CSOL	23	CSOL
45	HMA (0.10')	43	HMA (0.10')	41	HMA (0.10')

**Table 6**  
**M&R Schedules for Loss of Fatigue Life With 15-Year Grinding Life**

<b>10% Fatigue Life Lost From Grinding</b>		<b>20% Fatigue Life Lost From Grinding</b>		<b>30% Fatigue Life Lost From Grinding</b>	
<b>Yr</b>	<b>Activity</b>	<b>Yr</b>	<b>Activity</b>	<b>Yr</b>	<b>Activity</b>
0	Grind	0	Grind	0	Grind
10	CPR-C (NG)	10	CPR-C (NG)	10	CPR-C (NG)
15	Grind	15	Grind	15	Grind
20	CPR-B (NG)	19	CPR-B (NG)	19	CPR-B (NG)
25	CPR-A (NG)	23	CPR-A (NG)	22	CPR-A (NG)
29	CSOL	27	CSOL	25	CSOL
47	HMA (0.10')	45	HMA (0.10')	43	HMA (0.10')

**Table 7**  
**M&R Schedules for Loss of Fatigue Life With 20-Year Grinding Life**

<b>10% Fatigue Life Lost From Grinding</b>		<b>20% Fatigue Life Lost From Grinding</b>		<b>30% Fatigue Life Lost From Grinding</b>	
<b>Yr</b>	<b>Activity</b>	<b>Yr</b>	<b>Activity</b>	<b>Yr</b>	<b>Activity</b>
0	Grind	0	Grind	0	Grind
10	CPR-C (NG)	10	CPR-C (NG)	10	CPR-C (NG)
20	CPR-B	20	CPR-B	20	CPR-B
25	CPR-A (NG)	24	CPR-A (NG)	24	CPR-A (NG)
29	CSOL	28	CSOL	27	CSOL
47	HMA (0.10')	46	HMA (0.10')	45	HMA (0.10')
52	HMA (0.10') + 2% DO	51	HMA (0.10') + 2% DO	50	HMA (0.10') + 2% DO

To avoid removing too much of the structural capacity of the PCC slabs, the maximum number of grinds that can be performed is sometimes capped. In these cases, the pavement can fail by faulting rather than by cracking. To test this scenario, the M&R schedules presented in Table 5 were modified because of the maximum of two grinds (Years 0 and 10). Consequently, the CSOL activity is bumped up to Year 20 to compensate for the inability to perform another PCC grind. This makes faulting the critical distress mechanism. Table 8 shows the modified M&R schedules, which are capped at two grinds over the PCC service life.

**Table 8**  
**M&R Schedules for Loss of Fatigue Life With 10-Year Grinding Life and Maximum of Two Grinds**

10% Fatigue Life Lost From Grinding		20% Fatigue Life Lost From Grinding		30% Fatigue Life Lost From Grinding	
Yr	Activity	Yr	Activity	Yr	Activity
0	Grind	0	Grind	0	Grind
10	CPR-C	10	CPR-C	10	CPR-C
19	CPR-B (NG)	18	CPR-B (NG)	17	CPR-B (NG)
20	CSOL	20	CSOL	20	CSOL
38	HMA (0.10')	38	HMA (0.10')	38	HMA (0.10')
43	HMA (0.10') + 2% DO	43	HMA (0.10') + 2% DO	43	HMA (0.10') + 2% DO
48	HMA (0.10')	48	HMA (0.10')	48	HMA (0.10')

### Cost and Productivity Data

The basic data needed to perform an LCCA are the agency's initial and maintenance costs and the construction productivity rates (for the user delay costs). Many of the values adopted for this LCCA are taken from the Caltrans *Life-Cycle Cost Analysis Procedures Manual* (9). Other sources include the Caltrans Office of Construction Engineering (11), the *2005 State of the Pavement Report* (12), and contacts at the International Grooving and Grinding Association (IGGA) (8, 13). Historic construction cost records available on the Caltrans Intranet were also surveyed to validate certain costs. Table 9 shows the costs and productivities for the activities used in the analyses. Productivity is estimated assuming a 10-hour night shift. The initial and maintenance costs listed for DBR are for eight dowels per joint (four dowels per wheelpath) with an annual slot failure rate of 3 percent.

**Table 9**  
**Cost and Productivity Information**

Activity	Initial Cost (\$1,000/ln-mi)	Maintenance (\$1,000/yr/ln-mi)	Productivity* (ln-mi/shift)	Source
DBR	120.0	0.84 <sup>†</sup>	0.21	Roberts (8); Perez (11); Caltrans Intranet
Grind	46.3	n/a	7.00	Holloway (13); Perez (11); Caltrans Intranet
CPR-A	148.0	2.10	2.00	Caltrans (9)
CPR-A (NG)	101.7	2.10	2.33	Caltrans (9)
CPR-B	106.0	4.14	2.80	Caltrans (9)
CPR-B (NG)	59.7	4.14	3.50	Caltrans (9)
CPR-C	89.0	4.14	7.00	Caltrans (9)
CPR-C (NG)	42.7	4.14	14.00	Caltrans (9)
CSOL	279.0	1.32	0.52	Caltrans (9)
HMA (0.10')	99.0	0.63	1.87	Caltrans (9)
HMA (0.25')	299.0	0.63	0.66	Caltrans (9)
2% DO	1.0	n/a	n/a	Caltrans (12)
5% DO	2.5	n/a	n/a	Caltrans (12)

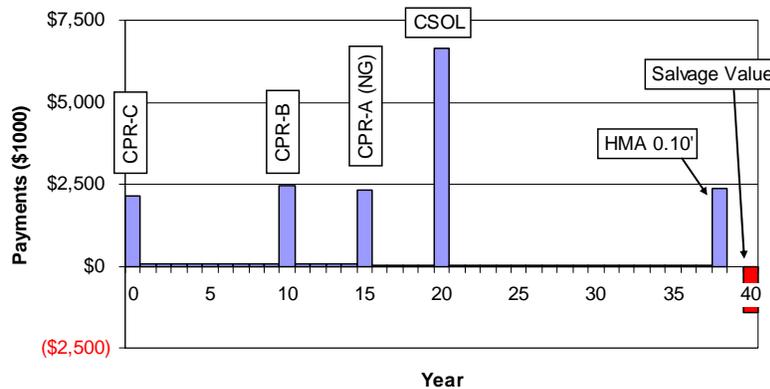
\*Assumes a 10-hr night shift. <sup>†</sup>Assumes a slot failure rate of 3%.

## Extension of Fatigue Life

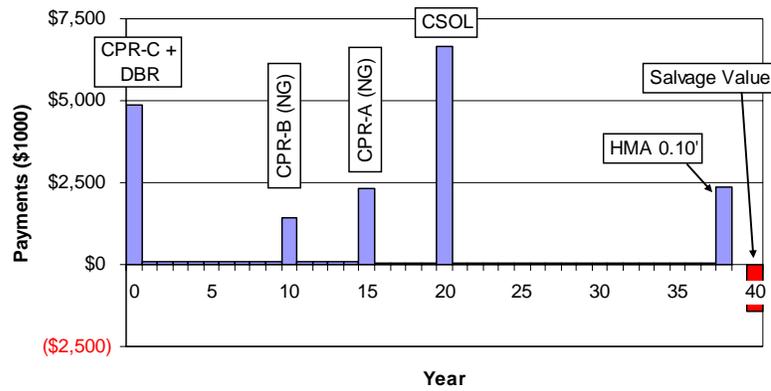
The results for this analysis are presented in terms of the fatigue life extension needed in order for DBR to become cost-effective. For instance, a value of 33.3 percent fatigue life extension needed indicates that on a pavement with an estimated 20 years of remaining fatigue, DBR would need to extend that life to 26.7 years to be cost-competitive with the base case. This includes delaying each rigid pavement maintenance activity by the same percentage. Once the pavement is cracked, sealed, and overlaid, the fatigue life extension benefits from DBR are nullified, making the duration between maintenance activities after CSOL identical for both the base case and the DBR case.

Figure 1 shows the cash flow diagram for an example base case, assuming a 20-year fatigue life remaining and a 10-year grind life. Figure 2 and Figure 3 show the cash flow diagrams for the equivalent DBR cases with 0 percent and 33.3 percent extension of fatigue life, respectively. The costs are for an assumed 20 lane-mi (32 lane-km) project, assuming a 5-mi (8-km) rural stretch of highway with two lanes in each direction (a total of 20 lane-mi [32 lane-km]). The section carries an annual average daily traffic (AADT) load of 38,500 vehicles, with an annual growth rate of 1.5 percent. Trucks account for 24 percent of the total AADT. The section is loosely based on a Route 99 DBR site in Kern County constructed in June 2000 that runs from post mile 54 to post mile 71.

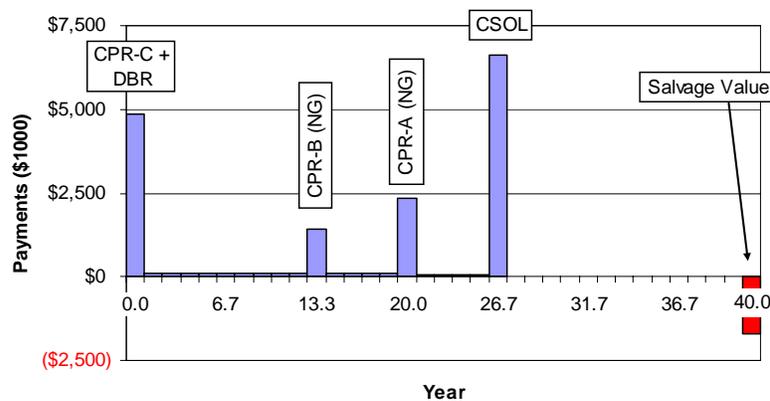
Salvage Value, shown in Figure 1 and all subsequent cash-flow diagrams, is calculated for any activities that have design lives that extend past the end of the analysis period. Salvage Value is calculated by taking the cost of the activity and multiplying it by the design life years after the end of the analysis period, and dividing by the design life. This assumes straight-line depreciation of the value of the activity and zero residual value at the end of its design life.



**Figure 1. Example base case: 20-year fatigue life remaining and 10-year grind life.**



**Figure 2. Example DBR case: 20-year fatigue life remaining.**



**Figure 3. Example DBR case: 20-year initial fatigue life remaining with 33.3 percent extension.**

It seems reasonable to expect that a well-constructed DBR could potentially extend fatigue life up to a maximum 50 percent. Although this is a great simplification, it is based on concrete pavement fatigue equations such as those in reference 14, which show that linear reductions in tensile stress in the concrete slab increase its fatigue life exponentially.

However, the current MEPDG models for fatigue cracking do not show any sensitivity to dowels, primarily due to the manner in which the finite element analyses used in the software to calculate tensile stresses were developed. The MEPDG can therefore be considered inadequate to evaluate the effect of DBR on concrete fatigue life. A comprehensive recalculation of the effects of DBR on tensile stresses is outside the scope of this project, however to illustrate the potential effects of DBR on tensile stresses causing fatigue cracking, a simple example was analyzed using a 3-D finite element program for concrete pavements called *EverFE* (15).

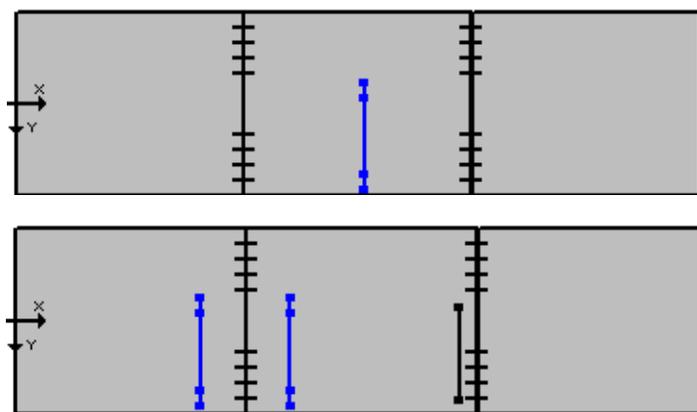
The case consisted of three slabs in a lane, 3.6 m (12 ft) wide and 4.6 m (15 ft) long. The two-layer system had PCC slabs 225 mm (9 in.) thick, a subgrade k-value of 0.06 MPa/mm (221 lbf/in<sup>2</sup>/in.), a PCC flexural strength of 4.5 MPa (650 lbf/in<sup>2</sup>), and typical stiffnesses for the PCC and dowels. The pavement was modeled as three slabs in a row in the direction of traffic.

Two loading cases which illustrate the expected extremes of the effect of dowels on stresses causing cracking were analyzed:

1. A dual-single axle of 80 kN (18,000 lbs) at the mid-point of the center slab near the longitudinal edge of the slab (Figure 4), and no temperature difference between the top and bottom of the slab. This case would occur several hours after sunrise or several hours after sunset. It was assumed that the average slab temperature was low and therefore that there was no aggregate interlock between slabs.

2. A dual-tandem axle of 140 kN (31,500 lbs) with one axle on the center slab and the other on the preceding slab, combined with a steering single axle of 55 kN (12,375 lbs) near the transverse joint of the center slab. Both axles are near the longitudinal edge of the slab as shown in Figure 4. A temperature difference was assumed in the slab of  $-15^{\circ}\text{C}$  ( $-27^{\circ}\text{F}$ ) between the top and bottom (cooler on top), which would occur in the Central Valley on a summer night.

These results were used with the fatigue equation cited above, and the percentage change in fatigue life was calculated for the cases with neither DBR nor aggregate interlock (common on cool nights) and with DBR (four dowels per wheelpath, typical spacing). The results are shown in Table 10.



**Figure 4. Load Locations for calculations shown in Table 10 (dual/single at mid-slab edge above; steering single on center slab and dual/tandem on two slabs below).**

**Table 10  
Tensile Stresses in PCC and Estimated Change in Fatigue Life for  
Example Load Cases**

	Maximum Tensile Stress in Slabs in MPa		Fatigue Life	Fatigue Life	% Change in Fatigue Life With DBR	
	1 Axle Mid-Slab	2 Axles on Slab	1 Axle Mid-Slab	2 Axles on Slab	1 Axle Mid-Slab	2 Axles on Slab
<b>DBR</b>	1.51	2.36	5.0E+11	2.4E+08	9%	626%
<b>No DBR</b>	1.52	2.58	4.6E+11	3.3E+07		

1 MPa = 145 lbf/in<sup>2</sup>

The results displayed in Table 10 show that for some load situations the dowels have little effect on estimated fatigue life, such as the case with one axle on the slab in Table 10, which shows that dowels have little effect on mid-slab loading near the edge. The results for the case with two axles on the slab and a nighttime temperature gradient indicate that tensile stresses can be reduced by dowels. It is interesting to note that the case with two axles on the slab resulted in maximum tensile stresses at the midslab, which would contribute to classical transverse fatigue cracking in the slab.

The purpose of these very simple and limited scope fatigue life extension calculations is to provide an indication as to whether it is even possible for DBR to extend the fatigue life of existing PCC pavements. If DBR cannot extend fatigue life, then it is not economically viable compared to the base case of grinding only because, as will be seen in Results, the high initial cost of DBR is not overcome by elimination of later grinding in terms of life-cycle cost if faulting is the only mode of failure considered.

## **RESULTS**

The LCCA was performed using the agency and user cost algorithms developed in the FHWA RealCost software package (16). The data input to the program included the variables and scenarios presented above in previous sections. The rest of the data were taken from recommendations in the *Life-Cycle Cost Analysis Procedures Manual* (9).

It is important to note that the final results of the LCCA are relatively unaffected by the case study's details. The traffic distribution (rural versus urban) affects only the user costs in the model, and that effect is minimal. While AADT and truck traffic would seem to have a significant impact on the results, their influence must be accounted for when estimating the remaining fatigue life of the pavement. The RealCost software uses only the direct input of AADT and truck traffic in calculating user costs, which are only road user delay costs associated with construction work zone traffic closures. As discussed below, the user costs are quite small when compared to the agency costs because of the use of nighttime traffic closures.

### **Variable Initial Remaining Fatigue Lives**

Aside from the initial (before DBR) remaining fatigue lives of 10, 20, and 30 years, results have also been produced at plus/minus 10 percent from the expected DBR initial cost (\$120,000/ lane-mi) and at yearly slot failure rates of 0 percent, 3 percent, and 6 percent. These permutations were analyzed to check the sensitivity of the results to uncertain initial cost and the varying performance history of DBR in California. The results are shown in Table 11. Recall the assumptions in this analysis and their significance as discussed above, that is, a discount rate fixed at 4 percent and an analysis period of 40 years.

**Table 11**  
**LCCA Results for 10, 20, and 30-Year Remaining Fatigue Lives**  
**(Assumes a 10-Year Grinding Life)**

Initial Cost of DBR	DBR Maintenance (Failed Slots/Yr)	Percent Fatigue Life Extension Needed From DBR to Equal Life Cycle Cost of Base Case		
		10-Yr Remaining Fatigue Life	20-Yr Remaining Fatigue Life	30-Yr Remaining Fatigue Life
Base Case Net Present Value →		\$11.21M	\$9.40M	\$6.65M
\$0.108M	0%	71%	35%	20%
	3%	74%	38%	24%
	6%	78%	42%	28%
\$0.120M	0%	80%	41%	25%
	3%	84%	44%	29%
	6%	87%	48%	33%
\$0.132M	0%	90%	47%	30%
	3%	93%	51%	34%
	6%	96%	54%	39%

Note: Dollar values are in millions of dollars per lane mile.

The percentages in Table 11 show that pavements with 30 years of initial remaining fatigue life hold the most promise for the cost-effective implementation of DBR, because they require the least amount of fatigue life extension from the DBR to have the same life cycle cost as the base case (grinding without DBR). It must be kept in mind, for reference, that a 50 percent extension in fatigue life for a pavement with 20 years of remaining life at the time of the DBR means that the eventual crack, seat, and overlay treatment must be deferred to Year 30. Depending on specific aspects of a given project, this may or may not be reasonable.

### Variable Grinding Lives

Table 12 provides a closer examination of the 30-year remaining fatigue life scenario by varying the grinding life in the base case (grinding without DBR) between 10, 12, 15, 17, and 20 years. Since additional grinding is not needed after the initial grinding that is part of the DBR treatment, the variation in years between subsequent grindings affects only the base-case scenarios.

Table 12 shows that the grinding life in the base case (no DBR) is not as important as the number of times grinding has to be performed during the remaining fatigue life. This is exemplified by the significant jump in fatigue life extension needed between the 12-year and 15-year grinding cases. When a grinding activity lasts for only 12 years, it needs to be performed three times (Years 0, 12, and 24) over a 30-year span; when it lasts for 15 years, grinding needs to be performed only twice (Years 0 and 15) before a major rehabilitation occurs. The change in fatigue life extension needed between other grinding lives (for example, 10 years and 12 years, or 15 years and 17 years) ranges between 1 percent and 3 percent, whereas the change between the 12- and 15-year grinding lives is between 8 percent and 16 percent.

**Table 12**  
**LCCA Results for Variable Grinding Lives**  
**(Assumes a 30-Year Remaining Fatigue Life)**

Initial Cost of DBR	DBR Maintenance (Failed Slots/Yr)	Fatigue Life Extension Needed from DBR				
		10-Yr Grinding	12-Yr Grinding	15-Yr Grinding	17-Yr Grinding	20-Yr Grinding
<b>Base Case Net Present Value →</b>		<b>\$6.65M</b>	<b>\$6.56M</b>	<b>\$6.05M</b>	<b>\$6.01M</b>	<b>\$5.90M</b>
\$0.108M	0%	20%	22%	30%	31%	33%
	3%	24%	25%	34%	36%	38%
	6%	28%	29%	40%	41%	44%
\$0.120M	0%	25%	26%	36%	37%	40%
	3%	29%	30%	42%	43%	46%
	6%	33%	34%	47%	49%	52%
\$0.132M	0%	30%	31%	43%	45%	48%
	3%	34%	35%	49%	51%	54%
	6%	39%	41%	55%	57%	60%

Note: Dollar values are in millions of dollars per lane-mile.

### Variable Fatigue Life Lost Due to Grinding

Using data from the MEPDG model, a 0.25-in (6-mm) grind will result in a 10 percent to 30 percent loss in fatigue life for PCC pavement. Applying DBR will aid fatigue life in two ways: (1) reducing deflections and stresses in the slab because of the presence of dowels at the joints, and (2) precluding the need to remove a fraction of the structural material (slab thickness) in future grindings.

Table 13 shows the results for a PCC pavement with an estimated 30-year remaining fatigue life. Because the percentage of life lost will vary between grinding activities, results are for fatigue life-lost increments of 10 percent, 20 percent, and 30 percent. The results are relative to the DBR case, with 30 years of initial remaining fatigue life at a cost of \$120,000 per lane-mile and a 3 percent slot failure rate per year.

Table 13 also includes a case where the grinding has been capped at a maximum of two grinds over its service life. To model this scenario, CSOL is performed at Year 20. This is the year when a third grinding would normally occur but is not allowed due to the two-grind limit.

**Table 13**  
**LCCA Results Modeling Fatigue Life Lost from Grinding (Assumes a 30-Year Remaining Fatigue Life)**

Percent Fatigue Life Lost From Grinding	Fatigue Life Extension Needed From DBR			
	10-Yr Grinding	15-Yr Grinding	20-Yr Grinding	10-Yr Grinding, Maximum Two Grinds
10%	21%	38%	42%	23%
20%	15%	30%	37%	22%
30%	9%	24%	33%	22%

As expected, modeling the fatigue life lost due to grinding will result in smaller percentages of fatigue life extension needed from DBR for DBR to be cost-effective compared to grinding without DBR. An intriguing note is that it is more cost-effective to cap the maximum number of grindings than to continue maintaining the PCC pavement for the rest of its service life. This is because multiple grindings may remove significant material from the surface. This makes future CapM activities (such as CPR) ineffective, due to the reduced life that can be expected from those future activities. For instance, if CPR-A is performed after 0.50 in (12.5 mm) of PCC has been removed, its expected life has been decreased from 5 years to between 2 and 4 years. The shortened life of this CPR activity due to thinning of the slab supports a plan to overlay the section before the pavement's fatigue life is exhausted.

## **CANDIDATE LANE-MILES IN THE CALTRANS NETWORK**

DBR can be applied only to pavements that meet the following three basic conditions: (1) the surface layer is PCC, (2) the PCC is undoweled JPCP, and (3) the PCC has a minimal amount of cracking (discussed below). Identifying the number of lane-miles that meet all these criteria will produce an estimate of the potential benefit that can be obtained from the addition of dowel bars.

The Caltrans Pavement Condition Survey (PCS) database contains records of the surface type and distress level of all pavements in the State's highway network. The lane-miles of PCC and their respective cracking levels can be extracted from this database. The database has to be filtered to keep only those cases where the pavement surface type is PCC and the slabs exhibit low levels of cracking. Requiring a low cracking level ensures that the pavement has enough remaining fatigue life to warrant investing resources to dowel the pavement's joints. If a section has high cracking levels, it is not practical to install dowels because it will likely be reconstructed shortly after their installation. For the purpose of mining the PCS database for candidate lane-miles for this project, the thresholds for first- and third-stage cracking were set at 10 percent and 5 percent, respectively. Pavement sections that exceeded either of those cracking levels were not considered as potential candidates for DBR.

First-stage cracks are transverse, longitudinal, or diagonal cracks that do not intersect but that divide the slab into two or more large pieces. Third-stage cracks are interconnected cracks that divide the slab into three or more large pieces. Some transverse cracks can be retrofitted with dowels at the same time as transverse joints (17, 18), as long as the transverse cracking is not too extensive. Extensive longitudinal cracks and corner cracks make it very difficult to execute DBR. First-stage cracking in the PCS database can be either transverse or longitudinal cracking.

According to the PCS, the total lane-miles of PCC that have less than 10 percent first-stage cracking and 5 percent third-stage cracking is roughly 10,200 lane-mi (16,415 lane-km). This estimate has been filtered from an original total pool of 12,800 lane-mi (20,600 lane-km) of PCC pavement across the State as of the 2006 survey.

The 10,200 lane-mi (16,415 lane-km) include the three different types of PCC pavements used in California: undoweled JPCP, doweled JPCP, and CRCP. Undoweled JPCP is the only concrete pavement that can be considered for DBR. Therefore, the lane-miles of CRCP and doweled pavements must be subtracted from the 10,200 lane-mi (16,415 lane-km) of PCC pavements identified in the condition survey database. CRCP is extremely rare in California. The only known occurrence of this on the State highway network is a short test section on westbound I-80 in Fairfield. Another section near Tracy was overlaid with asphalt some years ago, but because

there is such a small amount its length is negligible when compared to the total lane-miles of PCC pavement across the State.

California began using dowel bars in new construction in 1999. It became mandatory in 2004. According to the Caltrans Office of Construction Engineering, approximately 1,500 to 2,000 lane-mi (2,414 to 3,219 lane-km) of new PCC, full-depth pavement have been constructed since 1999. Unfortunately, since there is no comprehensive pavement structure inventory information in the current Caltrans database system, the only method of determining how many of those lane-miles contain doweled joints is to examine them on a case-by-case basis, which is too labor-intensive for this project. According to the Office of Construction Engineering, it is safe to assume that most PCC construction projects from 1999 to 2004 have dowels. For that reason, the value on the high end of the range is adopted: 2,000 lane-mi (3,219 lane-km) of doweled pavements.

The result of the above analysis is that there are approximately 8,200 lane-mi (13,197 lane-km) of pavement that are possible candidates for DBR. A further refinement to this value would be to stratify it by traffic levels because pavements with higher levels of truck traffic would benefit more from a doweled structure. This, however, is not easily accomplished using the available databases, therefore stratifying was only done in terms of truck and nontruck lanes. According to the PCS, 37 percent of the PCC lanes are truck lanes. Therefore, roughly 3,000 lane-mi (4,828 lane-km) of PCC are in truck lanes and currently do not contain dowels. These sections are likely to be good candidates for DBR. Because this represents roughly 25 percent of the total PCC pavement in the State, DBR could have a significant impact as a maintenance strategy, provided those lane-miles have sufficient remaining fatigue life to make DBR potentially economically competitive.

The construction dates of existing PCC pavements in the State are not available in any Caltrans databases at this time, and estimation of remaining fatigue life of PCC lane-miles across the network based on past truck traffic is outside the scope of this project. This analysis should be performed on a project-by-project basis if DBR is to be evaluated for LCCA versus grinding alone.

## **CONCLUSIONS AND RECOMMENDATIONS**

### **Conclusions**

The question that this study has tried to answer is whether DBR is an economically competitive option for rehabilitation of concrete pavements. Although a single, simple answer cannot be given, some important conclusions are drawn from the work presented here.

#### *Conditions That Make DBR Economically Effective*

The benefits of DBR will not be immediately realized, which means that the avoidance and delay of future M&R activities only occurs after significant time has passed. But the LCCA results show that DBR *can* be a cost-effective solution under certain conditions. On rigid pavements with relatively long remaining fatigue lives (~30 years), the investment in DBR is recaptured through avoiding future grindings and extending the initial fatigue life. Alternatively, on pavement sections with short remaining fatigue lives (~10 years), much of the capital investment in

DBR will probably not be fully recovered. When a PCC pavement will need to undergo a major rehabilitation in the near future, the DBR benefits will not have sufficient time to materialize.

#### *Extension in Fatigue Life Expected From DBR*

Under the currently typical California case analyzed (initial cost of \$120,000 per lane-mile, 3 percent failed DBR slots per year, 10-year grinding life, 20 years remaining fatigue life—see Table 11, DBR is the best solution if it provides at least a 44 percent increase in fatigue life. That is almost gaining 9 more years as a result of the DBR. The 44 percent is for a typical case, but Table 11 shows a sensitivity analysis and that, depending on the assumptions, this value can range from 20 percent to 96 percent. Some State agencies, like those of Kansas, Washington, Oklahoma, and South Dakota, seem to believe that such fatigue life extensions are possible, and have conducted extensive rehabilitation with DBR (see Reference 2).

If DBR is limited to pavement with 30 years of remaining fatigue life (Table 12, which is to say pavements with almost no cracking [could be less than 5 percent slabs with first-stage cracking]), then an increase in fatigue life from DBR between 20 percent and 60 percent (depending on life of the grinding-only treatment and the initial cost) would be sufficient. If the slab thickness reduction from grinding and associated fatigue life reduction is taken into account (see Table 13, which assumes 30 years of remaining fatigue life), then the results show that using a 40-year planning horizon and a 4 percent discount rate, DBR must increase the fatigue life of the pavement between 9 percent and 42 percent to be a cost-effective solution, depending on grinding life and effect of thickness reduction.

A simple, quick analysis indicated that DBR could potentially increase the fatigue cracking life of PCC slabs. A reasonable upper bound might be 50 percent, primarily based on judgment and the fact that for many cases, such as the single-axle case analyzed, DBR provides no fatigue cracking benefit, while for others, such as the two axles-on-the-slab case analyzed, DBR can reduce tensile stresses and therefore increase fatigue cracking life. Determining the actual extent of a potential fatigue cracking life increase would require a detailed study as the increase depends on the pavement structure, subgrade, climate, load locations, axle types, and other factors.

#### *Agency Versus Users Costs*

Although this analysis considered both user costs and agency costs, the latter drove the results. The user costs, while substantially higher for the DBR cases, accounted for only a very small percentage of the total costs. In this analysis, agency costs were generally two orders of magnitude larger than the accompanying user costs for a given activity. For user costs to make a significant impact in the results, long traffic queues would have to develop as a result of the construction. Queues such as these can be produced by using high traffic growth rates and/or peak-time construction closures, both of which are contrary to the assumptions used in this analysis. However, it is realistic to assume that Caltrans will take measures to mitigate the inconvenience posed to users through the Traffic Management Plan when performing the retrofit. This includes using nighttime and off-peak closure schedules whenever possible.

#### *Sensitivity to Construction and Maintenance Costs*

The only initial construction cost that had a notable effect on the results was that of the DBR. A 10 percent change in the initial cost of DBR resulted in a 5 percent to 10 percent change in the

fatigue life extension needed to be life-cycle cost competitive; this can be seen by the permutations included in Table 11 and Table 12. Economies of scale (which arise when DBR is implemented as a statewide rigid pavement capital maintenance activity) or other economic efficiency gains can reduce DBR construction costs.

The initial cost of grinding and the longevity of grinding activities are much less important to the results than the number of grinds that need to be performed over the remaining fatigue life of the JPCP. When the base case requires multiple grindings, the equivalent DBR case can avoid these future activities, thus saving the costs associated with them. The resulting loss of fatigue life is exacerbated when multiple grindings are necessary over the service life of the pavement.

Therefore, when faulting is the anticipated trigger distress, DBR becomes a more feasible alternative because of its ability to improve load transfer efficiency and therefore avoid future grindings.

The annual maintenance costs also play a key role. Although California has encountered varied maintenance demand on its finished DBR projects, DBR has performed much better on a nationwide scale. It is not unreasonable to expect that the maintenance demand can approach zero failed slots per year if experienced contractors are selected and if underperforming backfill materials are ruled out. Closer inspection during the construction process would also help to ensure a quality initial product, as would considering warranty contracts for DBR projects.

## **Recommendations**

In general, DBR is best suited for JPCP sections that have a substantial fatigue life remaining and that are susceptible to faulting. Because external inputs are highly sensitive, it is impossible to claim that DBR should (or should not) be used under all circumstances.

The first recommendation is that the decision to use DBR should be made on a project-by-project basis using the tables generated in this study. However, the results are interpreted, greatly simplified, as showing that:

DBR should not be performed on pavements containing more than 5 percent cracked slabs (third-stage cracking, either with slabs presenting interconnected cracks or having previously been replaced). The 5 percent cracking limit is subjective, but it was selected to reflect shorter fatigue life remaining in the candidate pavement. Even if slabs have been replaced and the current cracking level has therefore been reduced below 5 percent, the fact that slabs had been replaced should be interpreted as an indicator that the fatigue life of the original remaining slabs is about to be exhausted.

Another greatly simplified interpretation is that *DBR should be performed on pavements exhibiting less than 2 percent cracked slabs*. This will prevent faulting and will help in delaying slab cracking.

There is a grey area between the 2 percent and 5 percent where greater reliance on engineering judgment is required as part of a special project review. The primary factor to consider is the rate of faulting development. If there is sufficient fatigue life, then DBR is more likely to be life-cycle cost-effective where the rate of faulting development is high.

Factors increasing the rate of faulting development are:

- *Erodability of the base*: DBR will be more effective if the base is susceptible to erosion, which would lead to a faster rate of faulting development.
- *Poor aggregate interlock*: Smaller aggregates and lower strength aggregates will tend to lose load transfer efficiency faster, leading to faster faulting development. Loss of aggregate interlock also occurs where slabs have had greater shrinkage or have higher coefficients of thermal expansion, leading to opening of the joint.
- *Traffic forecasting*: DBR will be more cost-effective in situations with heavier traffic.

Each of these factors contributes to load transfer efficiency, which, combined with historical measurements of faulting development (current fault height divided by cumulative truck traffic or age), will give an indication of fault development rate.

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# Load Transfer Restoration—A Survey of Current Practice and Experience

Linda M. Pierce<sup>1</sup>

## ABSTRACT

The concept of restoring load transfer in existing concrete pavements through the installation of mechanical devices at transverse joints or cracks began in the United States in the early 1980s. A number of devices have been evaluated for their effectiveness in restoring load transfer and reducing the return of joint faulting. Dowel bar retrofit has been shown to be effective in restoring load transfer and minimizing the return of faulting. This paper focuses on the specifications, construction, and performance aspects of dowel bar retrofit.

## INTRODUCTION

Load transfer restoration of existing concrete pavements began in the United States in the early 1980s. Studies (1–36) evaluated a variety of mechanical devices that included Figure Eight, Vee, Double Vee, miniature I-beam, Georgia Split Pipe, and dowel bars. Many of the devices had varying degrees of success. The one load transfer restoration process that has gained widespread use, in at least the United States, has been dowel bar retrofit. The following provides a summary of current state highway agency dowel bar retrofit practices and performance.

## IMPORTANCE OF LOAD TRANSFER

Adequate load transfer is essential for the long-term performance of jointed plain concrete pavements. Sufficient load transfer will reduce tensile stress and deflections, which reduces the potential for joint spalling, base/subbase pumping, transverse joint faulting, and cracking. Depending on the level of truck traffic, load transfer can be obtained through aggregate interlock, treated bases, and/or the use of dowel bars placed at transverse joints. In general, adequate load transfer efficiencies occur between 70 and 100 percent. Load transfer efficiencies below 50 percent often times result in faulting, cracking, and poor ride quality (1).

## LOAD TRANSFER RESTORATION TECHNIQUES

The following briefly describes and summarizes each of the various load transfer restoration techniques.

### Figure Eight

The Figure Eight Device (Figure 1) consists of a cylindrical metal shell formed in the shape of the numeral eight (1). Installation includes coring 4-in. (102-mm) holes (two per wheelpath), centered over the transverse joint or crack (1). The device is secured to the existing to the sides

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of the core hole with epoxy. This device did not receive widespread use due to premature debonding failures and ineffectiveness in improving load transfer efficiencies (1).

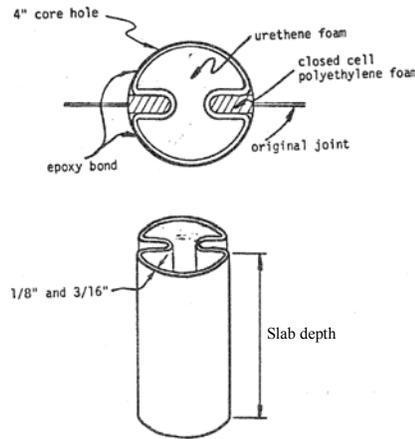


Figure 1. Figure Eight device (2).

### Vee and Double Vee

The Vee Device (Figure 2) consisted of a steel plate bent into the shape of a V (1). The Vee Device required coring two holes for each device, with two devices per wheelpath, centered over the transverse joint or crack. As with the Figure Eight Device, the Vee Device failed after the first winter due to bond failure and was ineffective in improving load transfer efficiencies (1).

The Double Vee device (Figure 3) consists of two Vee devices placed back to back and reduced in size to fit into a six inch core hole. Typically installation included drilling two core holes per wheelpath (1), centered over the transverse joint or crack. The Double Vee device is currently available and marketed by American Highway Technology. The Double Vee device had varied success (1, 3, 4, 5, 6, 7, 8, 9, 10) with the most common failures including debonding and consolidation issues with the polymer concrete patching material.

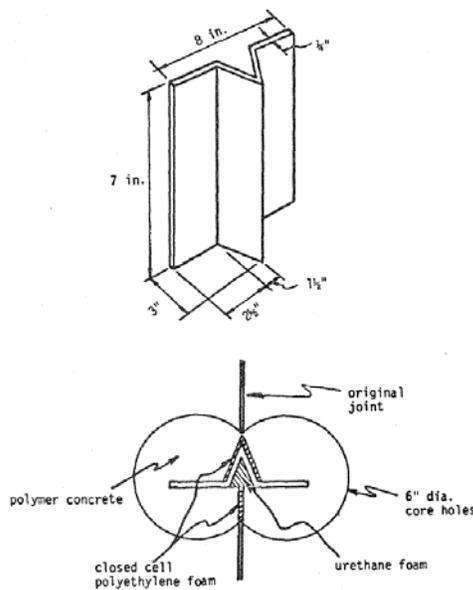


Figure 2. Vee device (2).

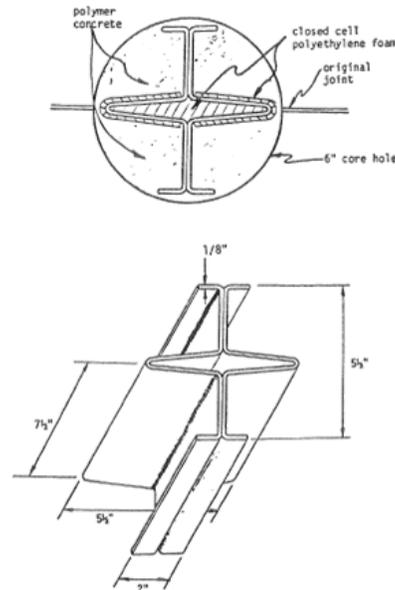


Figure 3. Double Vee device (3).

## Miniature I-Beam

The miniature I-beam (Figure 4) is installed in the same manner as dowel bar retrofit, which is discussed below. Study results indicate that the I-beam has no effect on reducing faulting (11, 12, 13).

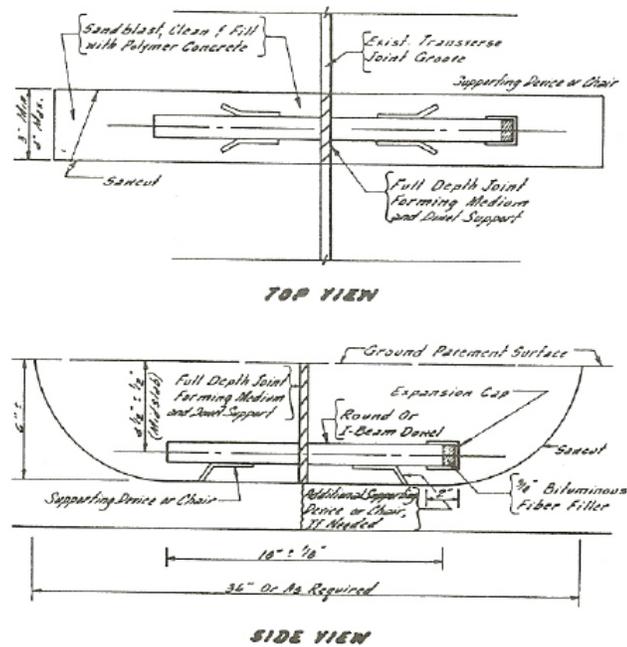


Figure 4. Miniature I-beam (12).

## Georgia Split Pipe

The Georgia Split Pipe device consists of two sides of a split pipe (Figure 5) epoxied to either side and placed into a 4-in. (102-mm) core hole centered over the transverse joint or crack. The one study (1) that evaluated this device indicated that it was difficult to construct, and its use was discontinued.

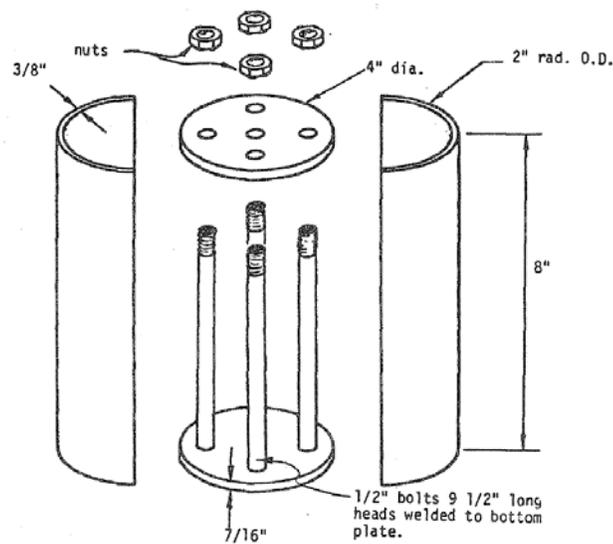


Figure 5. Georgia Split Pipe device (2).

## Freyssinet Connector

The Freyssinet Connector (Figure 6) was developed in coordination with the Laboratoire Central des Ponts et Chaussées (LCPC), the Service d'Études Techniques des Routes et Autoroutes (SETRA), and the Services Techniques de l'Aéroport de Paris, Freyssinet International. The connector consists of two symmetrical cast iron half shells, a steel key that slides in a housing machined in the half-shells and a central elastomeric sleeve that bonds the half shells and which makes the unit watertight (14). A total of four connectors are placed at each joint or crack. This device is currently available, but studies that evaluated its performance have not been identified.

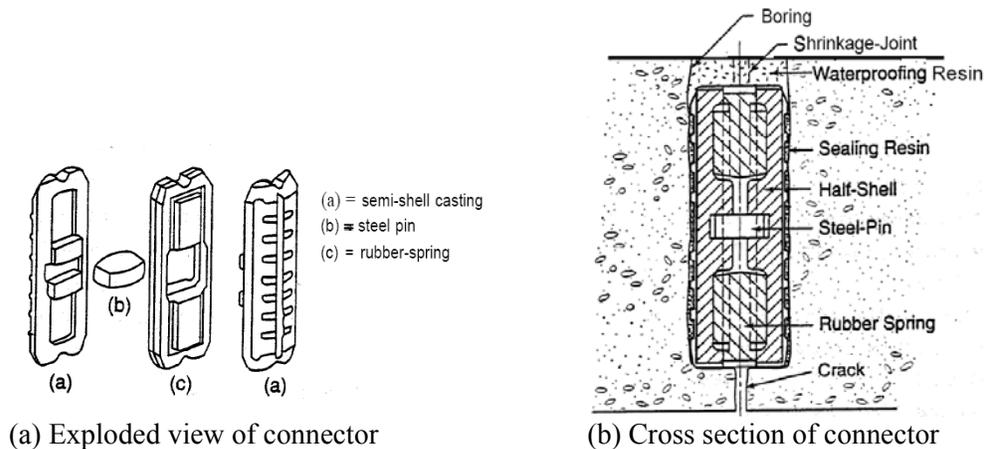


Figure 6. Freyssinet Connector (10).

## Dowel Bar Retrofit

Dowel bar retrofit includes the placement of smooth, round dowel bars at the transverse joints and/or cracks of jointed plain concrete pavements or at the transverse cracks of jointed reinforced concrete pavements. Slots are cut into the existing concrete pavement, of sufficient width and length to accommodate placement of the dowel bar (parallel and perpendicular to the centerline joint for non-skewed transverse joints) at mid-depth of the concrete slab. Prior to being inserted into the dowel bar slot, the dowel bars are placed on nonmetallic chairs to allow placement of the patching material around and beneath the dowel bar. In addition, caps are placed on both ends of the dowel bar to allow for dowel bar movement during concrete expansion and contraction. Typical dowel bar retrofit dimensions are shown in Figure 7.

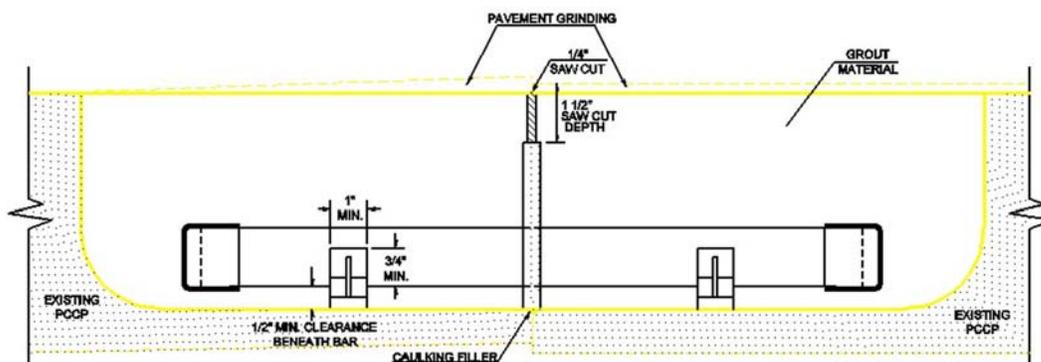


Figure 7. Dowel bar retrofit elevation and plan view.

## STATE PRACTICES AND SPECIFICATIONS

Currently, 23 State departments of transportation have placed over 5,000,000 dowel bars as part of the dowel bar retrofit process (Figure 8).

A summary of State specifications (15) for dowel bar retrofit designs is shown in Table 1, and patching material specifications are shown in Table 2. The majority of State highway agencies specify the use of a smooth steel dowel bar, 1 1/2 in. (38 mm) in diameter, placing three bars per wheelpath on 12-in. (305-mm) centers. Only the States of California, New York, Oklahoma, and Pennsylvania specify the use of four bars per wheelpath, and the States of Indiana, Minnesota, Mississippi, and Pennsylvania (also allows 1 1/4-in. [38-mm] dowel bars) require the use of 1 1/4-in. (32-mm) dowel bars. Specifications for patching material vary from State to State. In general, patching materials either listed on a State-qualified product list (QPL), in accordance with the National Transportation Product Evaluation Program (NTPEP) and/or ASTM C 928. The majority of States require the patching material to obtain a compressive strength (2,000 to 4,000 lbf/in<sup>2</sup> [13.79 to 27.58 MPa]) within a specified number of hours (2 to 6 hours) prior to opening to traffic.

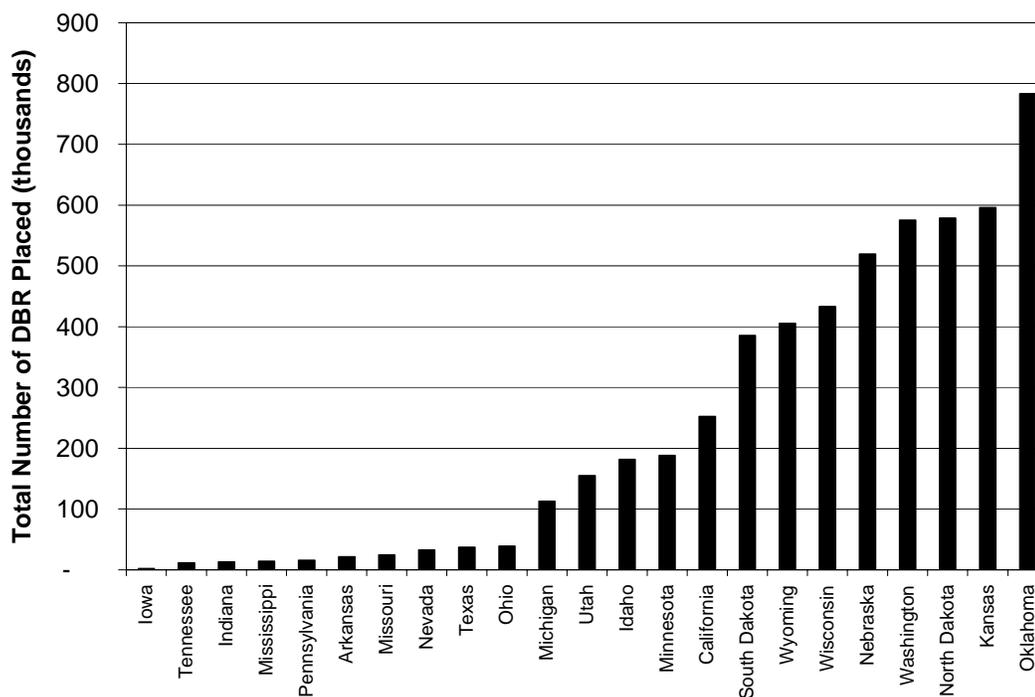


Figure 8. Dowel bar retrofit use (16).

**Table 1**  
**Summary of Dowel Bar Retrofit Specifications (15)**

<b>State</b>	<b>Bars</b>	<b>Dowel Bar Size (in.)</b>	<b>Bond Breaker</b>	<b>Caulking Materials</b>	<b>Curing Compound</b>
California	4	1½	Petroleum paraffin or white-pigmented	Silicone	Section 90-7.01B
Delaware	3	1½	White-pigmented	Compatible	Liquid membrane
Idaho	3	1½	Liquid membrane	Silicone	Patching supplier recommendations
Indiana	3	1¼	Bond breaking material	Silicone	Patching supplier recommendations
Kansas	3	1½	Pull out resistance less than 3400 lbs	Silicone	Liquid membrane
Louisiana	3	1½	Oil or grease	Silicone	White-pigmented
Michigan	3	1½	Qualified Products List	Qualified Products List	White membrane
Minnesota	3	1¼	Liquid membrane	Silicone	Modified membrane
Mississippi	3	1¼	Liquid membrane	Silicone	Liquid membrane, burlap, polyethylene sheeting
Missouri	3	1½	Graphite grease or approved equal	Caulking sealant	Liquid membrane
Nebraska	3	1½	Petroleum oil or grease	Non-sag sealant	White pigmented
Nevada	3	1½	Recommended by the coating manufacturer	As shown in plans	White-pigmented or wax-based
New York	4	1½	As approved by Materials Bureau	Silicone	White pigmented
North Dakota	3	1½	Liquid membrane	Compatible	Wax based
Ohio	3	1½	Oil or other bond-breaking material	Silicone	White pigmented
Oklahoma	4	1½	Form release oil	Sealant material	ODOT Spec Section 701.07
Pennsylvania	4	1¼ - 1½	Graphite Type B	Silicone	Section 705.8
South Dakota	3	1½	Form oil, white pigmented, asphaltic	Silicone	Liquid membrane
Tennessee	3	1½	Qualified Products List	Silicone	Liquid membrane
Utah	3	1½	Approved by Engineer	Silicone	Liquid membrane
Washington	3	1½	Curing compound or grease	Silicone	Liquid membrane
Wisconsin	3	1½	Manufacturer recommendation	Compatible	Poly-alpha-methylstyrene
Wyoming	3	1½	Liquid membrane	Silicone	Patching supplier recommendations

**Table 2**  
**Summary of Patching Material Specifications (15)**

State	General	Extender Aggregate Size	Compressive Strength (lbf/in <sup>2</sup> )	Shrinkage
California	Mg-phosphate; high alumina; PCC	< 3/8"	> 3000 @ 3 hrs; > 5000 @ 24 hrs	< 0.13 @ 4 day)
Delaware	---	Manufacturer's recommendation	> 2000 @ 6 hrs	< 0.13% @ 4 days
Idaho	Approved products	Manufacturer's recommendation	---	---
Indiana	Approved products	< 3/8" or 1/2"	Variable	< 0.03% @ 28 days
Kansas	ASTM C 928 and NTPEP <sup>1</sup>	< 3/8"	---	---
Louisiana	QPL <sup>2</sup>	Manufacturer's recommendation	> 3000 @ 3 hrs; > 5000 @ 24 hrs	< 0.13% @ 4 days
Michigan	QPL	< 1/2"	> 2000 @ 2 hrs; > 2500 @ 4 hrs; > 4500 @ 28 day	---
Minnesota	QPL	< 3/8"	---	---
Mississippi	Approved products	Manufacturer's recommendation	---	---
Missouri	QPL and NTPEP	Manufacturer's recommendation	---	---
Nebraska	Approved products	< 3/8"	> 3000 @ 4 hrs; > 4500 @ 24 hrs	---
Nevada	QPL	Manufacturer's recommendation	> 3000 @ 3 hrs	---
New York	Approved products	< 1/2"	---	---
North Dakota	Approved products	Manufacturer's recommendation	> 4000 @ 6 hrs	---
Ohio	ASTM C 928 and QPL	100% passing 1/2"; > 85% passing 3/8"	---	< 0.13% @ 4 days
Oklahoma	Section 701	Manufacturer's recommendation	> 4000 @ 6 hrs	---
Pennsylvania	Approved products	Manufacturer's recommendation	---	---
South Dakota	Approved products	Manufacturer's recommendation	> 3000 @ 3 hrs; > 5000 @ 24 hrs	< 0.13% @ 4 days
Tennessee	Approved products	Manufacturer's recommendation	> 4000 @ 6 hrs	---
Utah	Approved products	< 1/4"	---	---
Washington	---	< 3/8"	> 3000 @ 3 hrs; > 5000 @ 24 hrs	< 0.15% @ 28 days
Wisconsin	ASTM C 928	> 95% pass. 3/8"; < 25% pass. No. 4	> 3000 @ 3 hrs	---
Wyoming	Approved products	Manufacturer's recommendation	> 4000 @ 24 hrs	---

<sup>1</sup> National Transportation Product Evaluation Program; <sup>2</sup> Qualified Products List; 1 lbf/in<sup>2</sup> = 6.89 kPa

## DOWEL BAR RETROFIT PERFORMANCE

A number of States have reported various performance aspects of dowel bar retrofit. The following summarizes this information.

Puerto Rico was one of the first highway agencies to retrofit concrete pavements with dowel bars. Puerto Rico's first dowel bar retrofit project occurred in 1980, and to date excellent load transfer performance with less than 0.5 percent dowel bar slot failure has been reported. (17, 18).

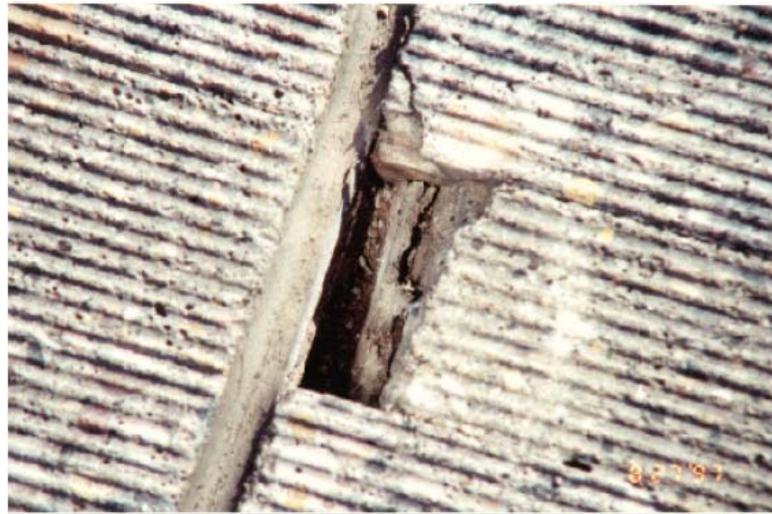
From 1994 to 1996, Minnesota Department of Transportation (DOT) constructed test sections to evaluate dowel bar length (15 and 18 in. [381 and 457 mm]), dowel bar configuration (two versus three dowel bars per wheelpath), and patching materials (MNDOT 3U18 and a proprietary patching material). After 6 years of performance, load transfer has remained above 80 percent on all sections, with no visible patching material failures and very little additional faulting (19).

Minnesota DOT determined that there was no performance difference between the 15-in. (381-mm) and 18-in. (457-mm) dowel bar length. The Minnesota studies also determined a large deflection difference with two dowel bars per wheelpath and recommend the use of three dowel bars per wheelpath. Studies also provide construction-related issues that include:

- Control of water content during batching of the patching material is critical to minimize shrinkage cracking and to enhance bonding of the patching material to the existing concrete.
- Sandblasting or other means of cleaning the dowel bar slots is required to guarantee bonding of the patching material to the existing concrete.

North Dakota has constructed several dowel bar retrofit test sections primarily for the evaluation of concrete patching materials (Minnesota DOT 3U18 and FOSROC Patchroc 10-60). Study conclusions (5) determined that dowel bar retrofit was effective in restoring load transfer, and, in general, distress within the dowel bar slot appears to be related to shrinkage cracking (3U18 mix), lack of bond, movement of the foam core board (Figure 9), or lack of consolidation of the patching materials (Figure 10). Due to contractor challenges in keeping the foam core board vertical within the dowel bar slot, North Dakota requires the use of a notched foam core board insert (Figure 11). The notched foam core board is also required by the States of California and Idaho and allowed (but not required) by Oklahoma and Wisconsin.

From 1997 to 2000, the Michigan DOT dowel bar retrofitted transverse cracks at seven different locations across the State. Annual monitoring indicates that all projects are performing as expected with limited spalling of the patching material and a few locations with dowel bar slot cracking.

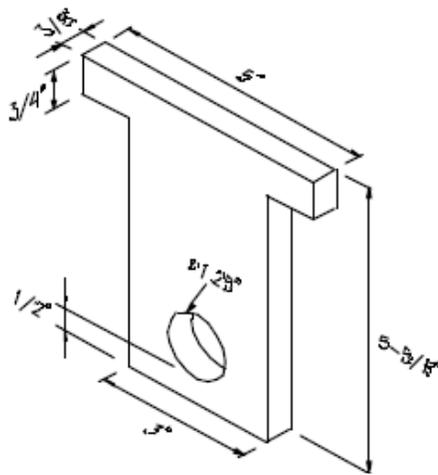


**Figure 9. Movement of foam core board (20).**

Wisconsin DOT constructed its first dowel bar retrofit test section in 1999, and 2 years after construction noted good load transfer results. Significant patching material failure was noted on several projects. Due to patching material failure, Wisconsin DOT placed a 1-year moratorium on dowel bar retrofit until further investigation could be conducted. In 2002, Wisconsin DOT lifted the moratorium, determining that the patching material failure was due to freeze–thaw durability issues. Today, Wisconsin DOT requires a 3-year warranty (warranties are also required in Nevada, Michigan, and Wisconsin) on dowel bar retrofit workmanship and materials.



**Figure 10. Lack of patching material consolidation (20).**



**Figure 11. Notched foam core board insert (20).**

California DOT (Caltrans) initiated studies (21, 22) to determine the applicability of dowel bar retrofit as a concrete rehabilitation treatment. What makes these studies unique was the use of a heavy vehicle simulator. The result from these studies indicated that dowel bar retrofit improved load transfer efficiencies and is promising as a rehabilitation strategy for faulted concrete pavements (21, 22). At the same time (1998 to 2003), Caltrans was actively dowel bar retrofitting approximately 100 lane-mi (161 lane-km) of existing concrete pavements. In 2001, Caltrans noted several locations of patching material failure (Figure 13) and initiated a study to investigate the cause of the distress.



**Figure 13. Failure of dowel bar patching material (23).**

The Caltrans study included the review of all 12 dowel bar retrofit projects with an in-depth investigation of six representative (good to poor performance) sites. Conclusions from this study included the following (23):

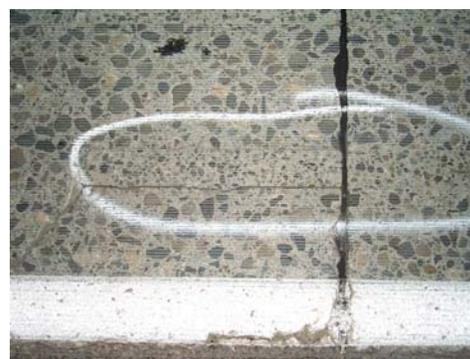
- Several locations of misaligned dowel bars; however, very few of these locations had any type of patching material distress.

- Magnesium phosphate patching material showed less adhesion and lower bond strengths than the high alumina backfill material.
- Variation of dowel bar slot distress exists within a given project.
- On several projects, dowel bar retrofit installations were not conducted according to specification (24, 25). On one project, dowels were not placed at the correct depth, patching material was not adequately consolidated and the slot was not properly cleaned prior to placement of the dowel bar assembly and the patching material. On a second project, 60-lb (specification required 30 lb) jackhammers were used to remove the existing concrete from the dowel bar slot, slots were cut too short to accommodate placement of the dowel bar, and there was misalignment of foam core boards.

Finally, the Washington State DOT has been dowel bar retrofitting concrete pavements since 1992, and to date has rehabilitated approximately 280 lane-mi (451 lane-km). In a detailed pavement condition review of approximately 180 lane-mi (290 lane-km) (or approximately 380,000 dowel bar retrofit slots) of dowel bar retrofit, it was determined that the majority of dowel bar slots have very little distress (15). Noted distresses include dowel bar slot cracking, debonding, spalling, and misaligned foam core board (Figure 14).



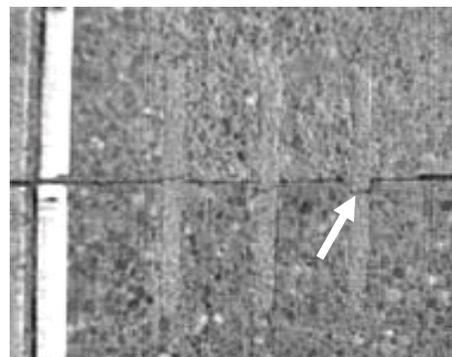
a. Cracking



b. Debonding



c. Spalling



d. Misaligned foam core board

**Figure 14. Dowel bar slot distress (15).**



**Table 3**  
**Summary of Future Use of Dowel Bar Retrofit**

<b>State</b>	<b>Primary Application</b>	<b>Estimated Future Use of Dowel Bar Retrofit (lane-mi)</b>
Arkansas	Transverse joints	None at this time
California	Transverse joints	500
Idaho	Transverse joints	20–120
Iowa	Transverse cracks	10–20
Kansas	Transverse joints	None at this time
Michigan	Transverse joints	Minimal
Minnesota	Transverse cracks	Unknown
Mississippi	Transverse joints	Unknown
Missouri	Transverse crack	2,000
Nevada	Transverse joints	None at this time
Ohio	Transverse cracks	Minimal
Oklahoma	Transverse joints	500–600
Pennsylvania	Transverse cracks	Unknown
South Dakota	Transverse joints	Minimal
Tennessee	Transverse joints	Minimal
Texas	Transverse joints	200–300
Utah	Transverse joints	Unknown
Washington	Transverse joints	600–800
Wisconsin	Transverse joints	2,000–2,200
Wyoming	Transverse joints	100
Total Estimate		5,900–9,900

## **SUMMARY**

Dowel bar retrofit has gained widespread use in the rehabilitation of faulted concrete pavements. The studies described above indicate that load transfer efficiencies are improved, reducing the potential for the return faulting. Based on a summary of State reports on dowel bar retrofit performance, many provide specifications for patching material (e.g., nonshrink, high early strength) selection and support the importance of quality control during construction.

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# Highway Panel Replacement—CSA Concrete in California

Chris Ramseyer<sup>1</sup> and Vincent Perez<sup>2</sup>

In general, the U.S. highway system dates from the 1940s and 1950s. Some of these original pavements exist 60 years later. Needless to say, our Nation's transportation infrastructure is in great need of improvements. According to the American Society of Civil Engineers' "2009 Report Card for America's Infrastructure," our roads earn a D-minus, with 33 percent of the nation's major roads in "poor or mediocre condition." Repair solutions must be cost-efficient, easy to implement, and long-lasting.

The last 10 years have seen considerable growth in the use of proprietary and special repair cements for concrete pavements. Many of these products lend themselves to "fast track" construction techniques that allow reopening to traffic within 12 hours or less. These products achieve high early strengths by accelerating the portland cement hydration process for both Type I and Type III cements or through alternative cementitious reactions that include alkali-activated aluminosilicate cements, sulfoaluminate based cements, or magnesium phosphate cements. These products are typically labeled as "cementitious" because their chemical reactions are inorganic, unlike the organic chemical reactions fundamental to epoxies and polymeric concretes. Unfortunately, most of these products are difficult to work with or uneconomical.

The perfect material for highway panel replacement would be (1) cost effective, (2) easy to work with, and (3) have very early strength for early opening to traffic. The time required for a concrete mixture to achieve a minimum compressive strength influences the timing of opening a repaired road to service. Zia et al. (1) applied a criterion for a minimum compressive strength of 13.8 MPa (2,000 lbf/in<sup>2</sup>) in 6 hours for very early strength (VES) high-performance concrete. This paper discusses the use of a VES calcium sulfoaluminate concrete to meet these challenges and its use in the State of California.

## PROPRIETARY TESTING PROGRAM

In the last 10 years, more than 20 proprietary very early strength (VES) materials have been tested at Donald G. Fears Structural Engineering Laboratory at the University of Oklahoma. These materials included aluminosilicate cements, sulfoaluminate based cements, magnesium phosphate cements, epoxy based cements and gypsum based cements. In general the materials tested compare poorly in comparison to VES I and VES III. These are state-of-the-art, Type I and Type III, very early strength (VES) concrete mix designs developed at the University of Oklahoma and adopted by the Oklahoma Department of Transportation (2,3,4).

One of the few exceptions is a VES mix built around a calcium-sulfoaluminate-based (CSA-based) cement. Table 1 presents the VES CSA mix proportions. Figure 1 illustrates the early age

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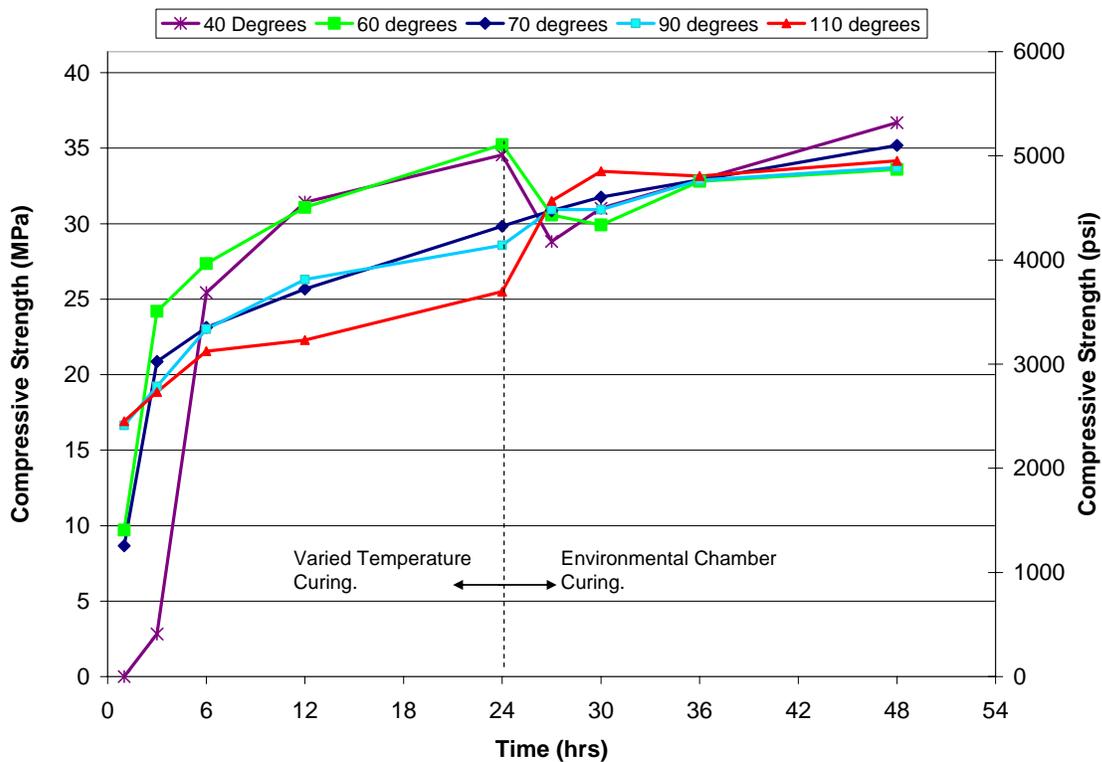
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compressive strength gain for this VES concrete with variable initial temperature conditions. In this test, for the first 24 hours the samples were subjected to a water bath with temperatures ranging from 4.4 °C to 43.3 °C (40 °F to 110 °F). After 24 hours, the samples were dry-cured in an environmental chamber at 23°Celsius (73.4°F) and 50 percent humidity. With the exception of the 4.4 °C (40° F) mix and 43.3 °C (110° F) mix, all of the samples met the strength requirement to open a road to traffic of 20.7 MPa (3000 lbf/in<sup>2</sup>) at 3 hours. These two exceptions met the requirement by 6 hours. The ASTM C 666 freeze–thaw durability is essentially 100 at 300 cycles and shows negligible visual damage. Performing the ASTM C 1202 Rapid Chloride Permeability Test on this material at 28 days rates this material’s permeability as “very low,” with 760 coulombs passing.

**Table 1**  
**Very Early Strength Mix Proportions**

Ingredient	Amount for 1 m <sup>3</sup> (1 yd <sup>3</sup> )
Calcium sulfoaluminate cement	390 kg (658 lb)
#57, Coarse aggregate	989 kg (1667 lb)
River sand, fine aggregate	867 kg (1462 lb)
Water	156 L (262 lb)
High range water reducer (polycarboxylate)	2.5 L (66 ozs)

Note: Water amount assumes aggregate is at saturated surface dry condition.



**Figure 1. Early age strength gain for very early strength calcium sulfoaluminate concrete with changes in temperature.**

The CSA VES mix typically used in California gains structural strength in approximately 1 hour at standard placement temperatures and has superior durability and low shrinkage. Due to its chemistry, it is resistant to sulphate and alkali–silica reactivity attack. The 1-year flexural strength of this typical pavement mix design is 7.7 MPa (1,120 lbf/in<sup>2</sup>), while the 1-year compressive strength is 71.6 MPa (10,390 lbf/in<sup>2</sup>).

## **USE OF CALCIUM SULFOALUMINATE CEMENTS BY CALTRANS**

California's Department of Transportation (Caltrans) has been using a VES CSA-based concrete for highway panel replacement since 1994. The average concrete panel in the State of California is 3.7 m wide, 4.6 m long, and 230 mm thick (12 ft wide, 15 ft long, and 9 in. thick). From 1994 to 2008 Caltrans replaced approximately 70,000 highway panels, an equivalent of approximately 322 lane-km (200 lane-mi), using this VES material. In total, over 267,600 m<sup>3</sup> (350,000 yd<sup>3</sup>) of VES CSA-based concrete has been used for highway panel replacement in California since 1994.

The replacement process is fairly simple. In California, perimeter sawcutting is allowed up to 2 days prior to the removal process. At the time of replacement, the panels can then be sawed into smaller sections for removal by a nonimpact method in an attempt to preserve the base material. In southern California, dowel bars, tie bars, or dowel baskets are not generally encountered during the removal and replacement process. The Special Provisions do mention sawing through tie bars or dowel bars before the concrete slabs are removed.

A bond breaker is required and must be one of the following:

1. Curing paper ASTM C-171, white.
2. Polyethylene film ASTM C-171 minimum thickness 6mm, white opaque.
3. Paving asphalt, Grade PG 64-10.
4. Curing compound.

Joints for pavement 8 in. (203 mm) thick shall have a minimum depth of 2.75 in. (70 mm) to D/3 in. (76 mm) (D = pavement thickness). Commercial quality polyethylene flexible expansion foam, 0.25 in. (6 mm) thick is placed to full depth along all joint faces.

Generally an ASTM C-309, Type II, Class B, resin based, white pigmented curing compound is used.

The highway panel is lifted out with minimal disturbance to the base. If the base needs repair, it is simple to remove the questionable material and then move on to the next step, placing the very early strength concrete. Filling the void left by the questionable base with high-quality VES CSA-based concrete. This process solves a poor base problem in a timely manner, maintaining the high production quantities required. It also achieves a superior pavement in this troubled area since the section properties of the pavement are improved due to the pavement's increased thickness. The remaining steps are similar to any concrete process, finishing and opening to traffic. Figures 2 through 5 outline this process.



**Figure 2. Damaged highway panel being removed.**



**Figure 3. Mobile Mixer placing very early strength (VES) concrete.**



**Figure 4. Finishing the panel.**



**Figure 5. Panels open to traffic.**

The average volume of VES CSA concrete produced in an 8-hour lane closure is 134 m<sup>3</sup> (175 yd<sup>3</sup>). This is enough to replace a total of 35 typical highway panels. Placing this material can be achieved by batching in a ready-mix plant or a mobile (Volumetric) mixer. The highest yardage of VES CSA produced in a single night using a ready-mix plant is 776 m<sup>3</sup> (1015 yd<sup>3</sup>) in a 10-hour production run. This is enough to replace a total of 203 typical highway panels. The highest volume of VES CSA produced in a single night using mobile (Volumetric) mixers is 381 m<sup>3</sup> (498 yd<sup>3</sup>) in a 6-hour production run. This is enough to almost replace 100 typical highway panels.

## COMPARISON TO PRECAST SYSTEMS

In general there are a number of additional requirements for recast panels that the VES CSA system does not have to deal with:

1. Transporting two panels per flat bed to the job site.
2. Crane and crane operator to unload panels.
3. Base sand that has been laser-screeded and compacted prior to receiving panel.
4. Tying panels together and grouting tie cable pockets.
5. Waiting for grout to obtain strength.
6. Limited production capacity.

Additionally, Smith (5) and Kohler et al. (6) mention the use of nonplanar warped slabs cast to the three-dimensional geometry required at the location they are placed. This type of panel increases the complexity and cost of the planning, manufacturing, coordination, and installation phases.

In general, the precast systems to date are require more time and are less cost efficient than the VES CSA system. The shortest time from production of the concrete to opening to traffic for the VES CSA system is 48 minutes, with the average being a little over an hour. The average production per shift is 35 typical highway panels, with approximately 1 panel completed every 14 minutes.

While the average cost of a VES CSA system highway panels in 2007 was \$2,716, the reported cost of precast panels have run between \$7,000 and \$24,000 dollars per highway panel. Table 2 by Muench et al. (7) lists the panel replacement projects in Washington (both rapid and not). Muench et al. mention that the costs in Table 2 “indicate that for a longer construction schedule (the Spokane project involved just over 20 days of lane closure for panel replacement), replacement costs can be between \$2,500 and \$10,00 per panel, while for rapid construction those costs can increase to the \$18,000 to \$25,000 range.” They went on to mention that their paper “assumes that a reasonable cost per panel replaced is \$20,000” because “future panel replacement projects are likely to use rapid construction.”

**Table 2**  
**Comparison of Panel Replacement Costs (Muench et al. (7))**

Panel Replacements	Rapid?	Cost (\$) <sup>a</sup>	Lane-mi	Panels Replaced		Rounded Cost/Panel (\$)
				No.	Percent	
Tacoma, WA (2006)	Yes	735,000	10.4	29	0.79	25,300
Federal Way, WA (2006)	Yes	1,000,000	11.4	54	1.35	18,500
Bellingham, WA (2003) <sup>b</sup>	No	660,000	13.2	265	5.70	2,500
Vancouver, WA (2006)	No	1,600,000	16.5	233	4.01	7,000
Spokane, WA (2007)	No	300,000	11.1	36	0.92	8,000

1 mi = 1.61 Km

a. Costs are approximate because of difficulties in separating panel replacement work from other work on single contracts, differences in costs between geographic locations, and differences in complexity and urgency of work.

b. The contractor lost a substantial amount of money on the Bellingham job. Therefore, the rounded cost per panel, while reasonably accurate for this job, is probably too low to use for future estimating.

## COMPARISON TO ASPHALT PANEL REPLACEMENT

For years, the prevailing mindset was that asphalt is cheaper than concrete. However, the reverse is actually true in today's changing marketplace. The days of inexpensive asphalt have come to an end, in part due to rising petroleum prices and an overall shortage of asphalt in the United States. The U.S. asphalt shortage is so severe that currently the country is undersupplied by about 24,000 barrels of asphalt a day, a figure that is expected to jump to 257,000 barrels a day by 2012, according to San Antonio-based NuStar Energy L.P., a producer of asphalt (8).

Table 3 is a direct comparison of panel replacement in Southern California districts where asphalt was used to make repairs in 2007. The eight Caltrans contracts include a total of 2,249 panels. The average price for the VES CSA system was \$2,703.39 per panel, and the average price for the asphaltic concrete was \$2,789.37. In a direct comparison of initial cost, asphalt was found to cost more than VES CSA concrete, which was found to offer an average initial cost savings of \$85.98 per panel or \$193,369.02 dollars overall on these contracts. When the contracts are compared within their district, the average initial cost savings is approximately \$300 per panel.

The longevity of the VES CSA concrete accounts for few, if any, repairs over the lifespan of the roadway. If life-cycle costs are compared, and the longevity of VES CSA concrete when compared to asphalt is factored into the economic question, the advantage of CSA concrete is even more evident.

Many people hold a perception that VES concrete is difficult to use. In this case, the reverse is true. Research at the University of Oklahoma has shown that the VES CSA system is much more forgiving than typical VES concretes. CSA has a high theoretical optimum water-to-cementitious materials ratio of 0.47, which means it is extremely pourable and placeable, easier in some cases to install than asphalt. And since all the water is used in the hydration process, shrinkage is extremely small. In addition, fast-setting materials such as VES CSA cement can be extremely user-friendly due to (1) the advances in admixtures available today, which have helped to increase the material's working time, and (2) advancements in the clinkering process

and production quality control of CSA cements available in the United States, which have helped improve batching characteristics.

**Table 3**  
**Initial Cost Comparison of 2007 Caltrans Contracts**

<b>Contract #</b>	<b>Location</b>	<b>Material</b>	<b># Panels</b>	<b>Cost / Panel (\$)</b>
03-0C7704	03-PLA-80-R66.3/68.5	Rapid Setting Concrete	272	2,295.00
01-457704	01-MEN-101-R31.6/R33.7	Rapid Setting Concrete	133	2,715.75
05-491904	05-SLO-1-L16.7/18.1	Rapid Setting Concrete	209	3,155.63
06-0G6204	06-KER-5-62.0/82.5	Rapid Setting Concrete	198	2,640.00
04-269604	04-CC-24-R0.1/R8.3	Rapid Setting Concrete	993	2,731.05
		Total	1,805	
		Average Cost per Panel		\$ 2,703.39
06-0F5004	06-KER-14-R17.9/T22.0	Asphaltic Concrete	128	3,387.75
04-447204	04=ALA-92-6.8/8.2	Asphaltic Concrete	34	3,407.63
04-4A6104	04-SOL-80-22.0/30.6	Asphaltic Concrete	282	2,443.22
		Total	444	
		Average Cost per Panel		\$2,789.37

***Environmental Considerations***

Environmental sensitivities are among the forefront of today’s building and construction topics, and here VES CSA again excels over asphalt for numerous reasons. First, the production process uses less fuel and is able to utilize alternative fuel sources such as industrial byproducts. By relying on waste materials for production, VES CSA actually helps reduce the carbon footprint of other production processes.

Like other concrete products, VES CSA does not have the rain or water runoff concerns that asphalt has. After a rain, no oils come to the surface of VES CSA that could then seep into the water supply.

Another environmental benefit is that as a concrete product, VES CSA helps minimize the urban heat island effect. This phenomenon occurs in cities, where asphalt is the primary building material for roads and parking lots. The black surfaces absorb heat and raise temperatures in these areas, so that the city experiences an increased demand for electricity to cool homes and buildings. Brownouts and blackouts then become more likely as energy demand soars. On the other hand, concrete is reflective, helping to keep temperatures down; concrete roadways help minimize such an urban heat island effect.

The durability of VES CSA makes it a greener choice in that fewer, if any, repairs are required over the course of the pavement’s lifespan. Concrete, in general, is an extremely durable product, and VES CSA is no exception. According to the American Concrete Pavement Association (ACPA), most pavements are placed with a targeted design life of 20 years, but

concrete pavements typically last much longer. There are well-documented cases of heavily trafficked concrete pavements that have performed for more than 40 years. On the other hand, most asphalt pavements last less than 20 years, and recent modifications to asphalt mixes due to the increased efficiency of the cat-cracking process, which has reduced the amount of heavy molecular weight organic compounds in asphaltic oils, have shortened the product's life span even more.

What is more, the cement industry has reduced the amount of energy required to produce the product by 30 percent and is committed to reducing this figure by another 10 percent by the year 2020. All materials are readily available in the United States, and the production does not rely on the importing of oil. Adding to concrete's green appeal is the material's recyclability and reusability. According to the ACPA, routinely old concrete is crushed, steel components are removed and recycled, and the crushed concrete is used for roadbed materials, stormwater management, aggregate in new concrete mixtures, and also some nonpaving applications. Concrete is 100 percent recyclable and reusable.

As contractors look for ways to create a competitive advantage and transportation departments look for long-term, high-value solutions, the VES CSA system proves to be one such means of doing so. This CSA-based cement allows paving costs to be kept at a minimum, while roads can open in shorter times.

## CONCLUSIONS

Caltrans has placed over 70,000 highway panels using CSA concrete, an equivalent of approximately 322 lane-km (200 lane-mi) since 1994. It is easy to work with and requires a minimum of equipment. It can be successfully produced at a batch plant or Volumetric mixer. It provides VES, which minimizes a project's impact on traffic. CSA concrete provides a cost-effective solution for highway panel replacement.

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## **Part 4**

### **Concrete Pavement Surface Texture**



# Effect of Diamond Grinding on Noise Characteristics of Concrete Pavements in California

Shubham Rawool<sup>1</sup> and Richard Stubstad<sup>2</sup>

## ABSTRACT

The construction of sound walls along highways has been the primary noise mitigation strategy in California and in many other western States. Sound walls cost approximately \$1.5 million per mile and are effective only in close proximity to the highway, on the “far” side of the sound wall, so to speak.

In its efforts to explore other noise mitigation strategies, the California Department of Transportation (Caltrans) recently conducted a study to determine the effect of diamond grinding on the noise characteristics of existing concrete pavements. Since the noise generated at the tire–pavement interface is the greatest contributor to highway noise, quieter pavement surfaces can reduce overall noise levels for both road users and neighborhoods—whether sound walls are used or not.

On-board sound intensity (OBSI) measurements were conducted on six routes in California, for a total of 42 evaluation sections; each evaluation section was 440 ft (136.8 m) long. OBSI measurements before and after diamond grinding were recorded. Following are the overall conclusions that were reached after the pre- and post-grinding OBSI levels were measured:

- There is a significant and readily audible reduction in OBSI levels (and hence in tire–pavement noise) after grinding.
- An average 2.7 dBA reduction in OBSI levels was observed for all test sites.
- Among the six routes, the highest average reduction of 4.4 dBA was observed on I-5 near Richards Boulevard in Sacramento County, and the lowest reduction of 1.2 dBA was observed on State Route 60 (on a single test section) in San Bernardino County.
- The highest reductions in sound intensity levels on a 1/3-octave band basis occurred in the 1600 Hz band, while the lowest reductions occurred in the 1000 Hz bandwidth.

## INTRODUCTION

The construction of sound walls along highways has been the primary noise mitigation strategy in California and many other States. Since the sound walls have their own limitations, other strategies which could be used in conjunction with the sound walls are investigated. Among the strategies that can reduce noise levels and sustain them while maintaining durability, maintain-

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ability, and friction is improving pavement surface characteristics during construction or diamond grinding the concrete surface post-construction. Efforts are underway to determine the optimum surface characteristics that would simultaneously address tire pavement noise, texture, smoothness, and friction.

In its effort to develop newer noise mitigation strategies, the California Department of Transportation (Caltrans) is evaluating various alternatives that can reduce highway noise, even in existing portland cement concrete pavement (PCCP). Since the noise generated at the tire–pavement interface is the greatest contributor to highway noise, quieter pavement surfaces can reduce the overall noise levels substantially for road users and neighborhoods alike. The noise-reducing capabilities of open-graded asphaltic mixtures, porous concrete, and diamond-ground pavement surfaces are some of the alternative approaches that are being considered to reduce tire–pavement interface noise.

Grinding on bridge decks and elevated structures has been found to reduce tire–pavement source levels by 3 to 10 “average weighted decibels” (dBAs), with relatively comparable reductions in wayside noise measurements (Ref. 1). Also, in Arizona, diamond grinding of transversely tined concrete surfaces has been found to reduce pavement interface source levels by up to 9 dBAs (Ref. 1). Similar results were also obtained in a study conducted on Route 101 in California. Average noise-level measurements and single-vehicle pass-by (not pavement interface) values were used to determine the reduction in noise levels after diamond grinding. Although the results of the average noise levels were found to be inconclusive, the results of the pass-by noise-level measurements showed an average noise drop of approximately 6 dB at 25 ft and 4 dB at 50 ft (Ref. 2). Considering this potential level of noise reduction, it can be concluded that diamond grinding of concrete surfaces may be a feasible means of reducing noise levels on existing PCCP.

### **Objective of the Study**

The main objective of the Caltrans study was to determine the effect of diamond grinding on the tire–pavement noise characteristics of PCCP. Existing tire–pavement noise characteristics of in-service pavements were determined both before and after grinding. The study had the following specific objectives:

- Conduct on-board sound intensity (OBSI) measurements to determine tire–pavement noise levels on PCCP sections before and after diamond grinding.
- Compare these before-and-after OBSI results to determine the change in sound intensity (SI) levels after grinding.
- Select candidate PCCP sections to monitor long-term noise characteristics.

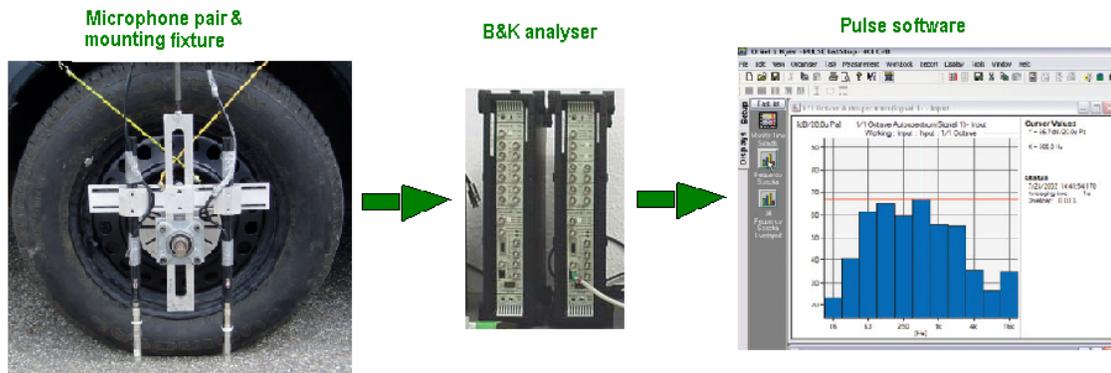


Figure 1. OBSI equipment setup.

## DATA COLLECTION

### OBSI Equipment

The OBSI method was used to measure tire–pavement interface noise. The equipment setup used to conduct measurements consisted of a Bruel and Kjaer front-end analyzer and the associated “Pulse” software package, two probes (each consisting of a microphone pair), a mounting fixture, and a Michelin Standard Reference Test Tire (SRTT—see Figure 1).

Each probe has two 0.5 in. (13 mm) diameter, phase-matched condenser microphones spaced 0.625 in. (16 mm) apart and fitted with a wind screen (Figure 2).

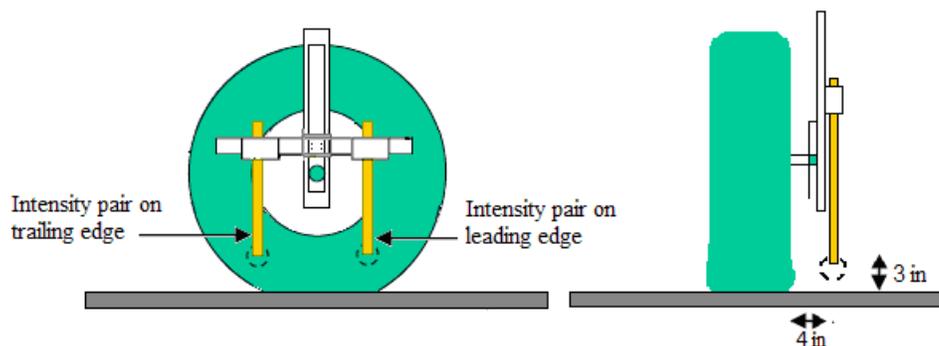


Figure 2. Schematic diagram depicting the microphone setup.

The two probes are placed 3 in. (76 mm) above the pavement and 4 in. (102 mm) away from the tire sidewall and are positioned to capture SI at the leading and trailing edges of the tire contact patch (Figure 2). The SI at the two probes is captured simultaneously by the front analyzer in real time and can be viewed on an onboard computer using the Pulse software. The intensity values at the leading and trailing edges of the tire contact area are averaged together on an energy basis to determine the SI for a given pavement section.

## Testing Procedure

The test vehicle was driven at a constant speed using the cruise control. For each route, test sections with minimum grade and alignment changes were selected. Tests were conducted at times when traffic was sparse and on days when dry pavement and favorable wind conditions were present. To limit the data to as few variables as possible, a test plan with the following parameters was developed:

- Constant speed of  $60 \pm 2$  mi/h ( $97 \pm$  km/h).
- Michelin Standard Reference Test Tire (SRTT).
- Cold tire pressure  $30$  lbf/in<sup>2</sup> (206.8 kPa).
- No significant grade.
- Dry pavement.

To reduce any bias caused by the equipment; microphones, preamplifiers, and cords were numbered and placed in the same location for each test. Before and after every test, each of the four microphones was calibrated.

## Test Sections

All pavement sections considered for this project are listed in Table 1. Caltrans originally identified 15 routes on which pre-grind OBSI measurements were conducted. A total of 81 evaluation sections, each of 5-second duration (440 ft [134 m]), were measured using the OBSI testing equipment.

Since some of the pavements remained unground by the time this study ended, only 6 of the original 15 routes (consisting of several test sections on each route) could be tested for post-grind noise levels. However, these included over half (42 out of 81) of the evaluation sections originally surveyed in the pre-grind phase of the project. All the evaluation sections except one (along I-60 southbound) showed a reduction in SI after grinding. The section on SR 60 beginning at postmile (PM) 7.9 was excluded from analysis because it was not considered typical.

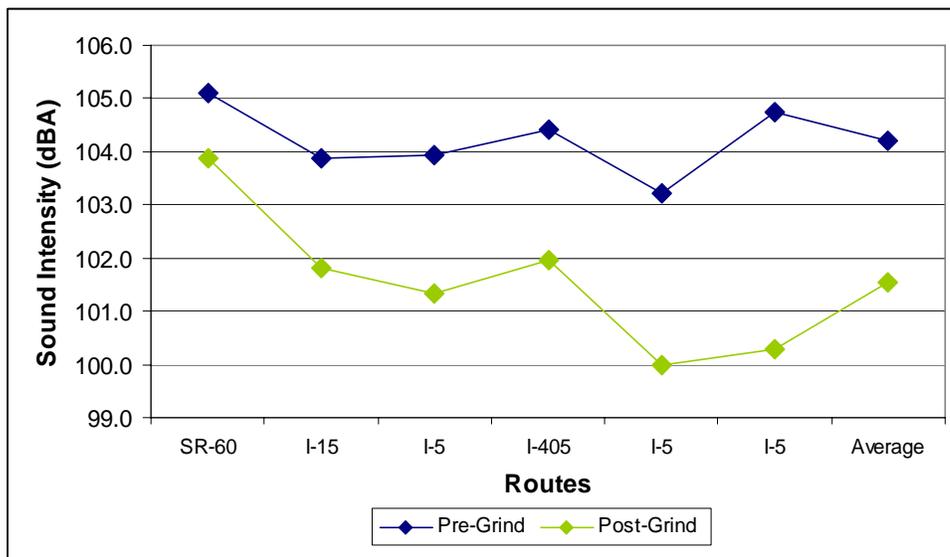
Table 2 and Figure 3 show pre- and post-grind SI values for the six routes. There was a reduction in SI on all the routes after grinding. The greatest reduction in SI value—4.4 dBA—was observed on I-5 in Sacramento County, and the lowest reduction (1.2 dBA) was observed on State Route 60 in San Bernardino County. On average, the reduction for all six sites was 2.7 dBA—a significant and audibly noticeable improvement.

**Table 1**  
**Details of Tested Routes**

Site No.	Route	County	Starting Postmile	Ending Postmile	No. of Tested Sections (440 ft)	Date of Pre-grind Testing	Date of Post-grind Testing
1	I-5	LA	32.3	44.3	7	5/22/2007	Not ground
2	I-10	LA	18.3	32.7	7	5/22/2007	Not ground
3	SR-60	LA	23.9	30.4	3	5/8/2007	Not ground
4	SR-60	SBD	0	9.9	2	5/8/2007	6/4/2008
5	I-15	SBD	0	3.8	4	5/9/2007	Not ground
6	I-15	RIV	51.4	52.3	2	5/9/2007	Not ground
7	I-10	RIV	0	8.2	4	5/9/2007	Not ground
8	I-15	RIV	8.1	23.9	6	5/23/2007	6/5/2008
9	SR-91	RIV	0	9.0	4	5/10/2007	Not ground
10	I-5	ORA	14.5	21.3	4	5/23/2007	6/5/2008
11	I-405	ORA	16.9	24.2	3	5/22/2007	6/5/2008
12	SR-101	SFO	0	4.2	1	5/06/2007	Not ground
13	I-280	SCL	5.1	7.8	4	5/06/2007	Not ground
14	I-5	KER	10.2	15.8	14	4/16/2008	5/17/2008
15	I-5	SAC	24.1	24.8	16	5/19/2008	7/24/2008

**Table 2**  
**Pre- and Post-grind SI Values From All Six Routes**

Site No.	Route Name	County	Pre-Grind SI (dBA)	Post-Grind SI (dBA)	After Grind Reduction (dBA)
1	SR-60	SBD	105.1	103.9	1.2
2	I-15	RIV	103.9	101.8	2.1
3	I-5	ORA	104.0	101.3	2.6
4	I-405	ORA	104.4	102.0	2.5
5	I-5	KER	103.2	100.0	3.2
6	I-5	SAC	104.7	100.3	4.4
<b>Average</b>			<b>104.2</b>	<b>101.5</b>	<b>2.7</b>



**Figure 3. Pre- and post-grind SI values for all six routes.**

### Detailed Data Analysis for I-5 in Kern County

A detailed analysis was conducted on the I-5 project in Kern County. This site was chosen for detailed analysis since it had the most evaluation sections among all sites tested. Also, the data on this site were considered to be most representative to study the detailed effect of grinding on SI levels because pre- and post-grind data were collected within a 1-month period of one another.

On this route, data were collected in both the northbound and southbound directions. In the northbound direction, five consecutive 440-ft (134-m) sections starting at PM 11.8 were measured. Similarly, in the southbound direction, nine consecutive 440-ft (134-m) sections starting at PM 12.3 were measured.

#### *SI Levels for I-5 in Kern County*

A comparison of the pre- and post-grind SI levels is shown in Figure 4 and Figure 5 for the southbound and northbound directions of traffic, respectively. The average pre-grind noise level in the southbound direction was 102.8 dBA, whereas in the northbound direction it was 104.0 dBA. Correspondingly, the average post-grind noise levels were 100.1 dBA and 100.0 dBA in the southbound and northbound directions, respectively.

Note that the average pre-grind noise level was approximately 1 dBA higher in the northbound direction compared to southbound direction. After grinding, a reduction of 3.9 dBA northbound and 2.9 dBA southbound was observed (Figure 6). The average post-grind SI level for this site was 100 dBA. This means that the results in terms after-grind dBA were similar, regardless of the initial noise level.

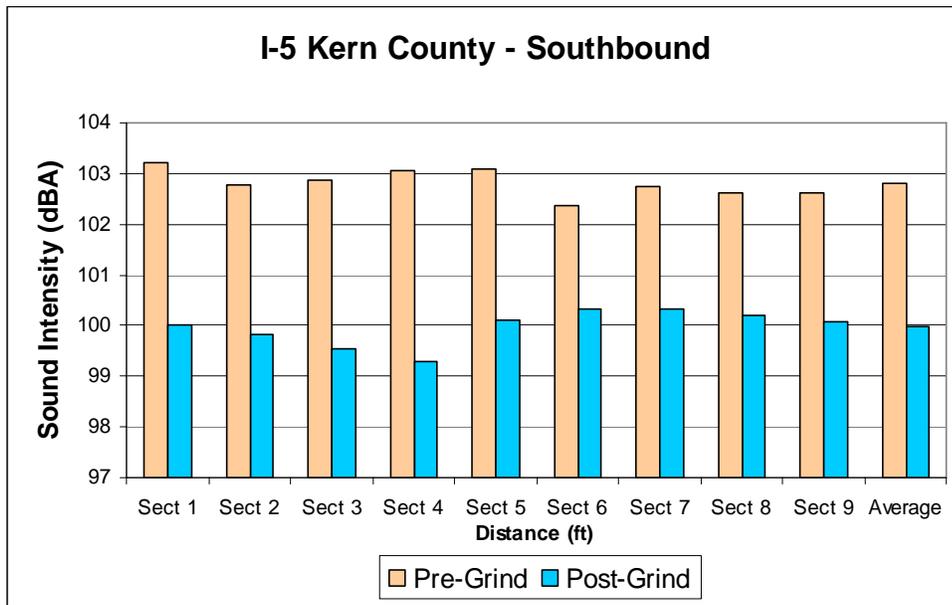


Figure 4. A-weighted SI levels averaged over four runs for southbound I-5 in Kern County.

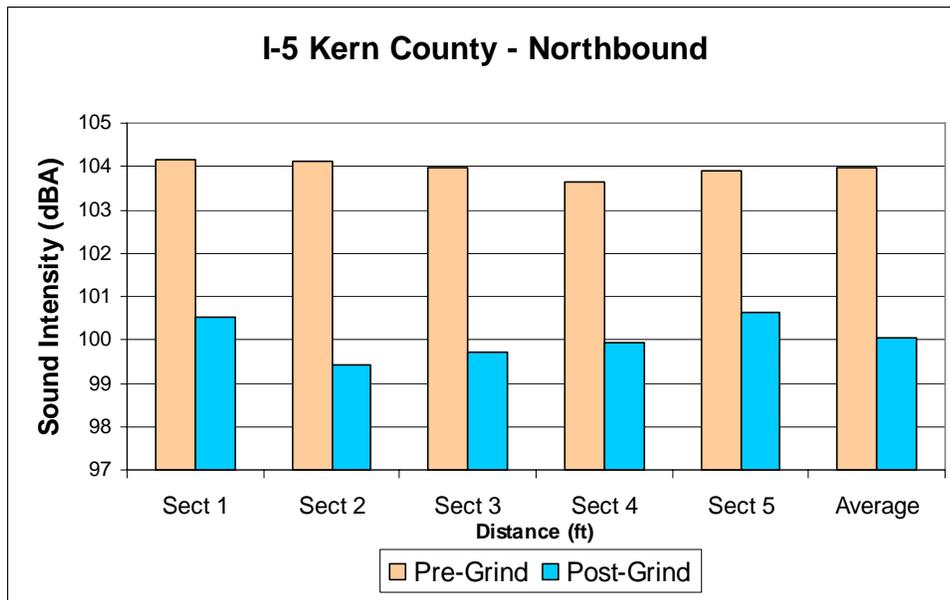


Figure 5. A-weighted SI levels averaged over four runs for northbound I-5 in Kern County.

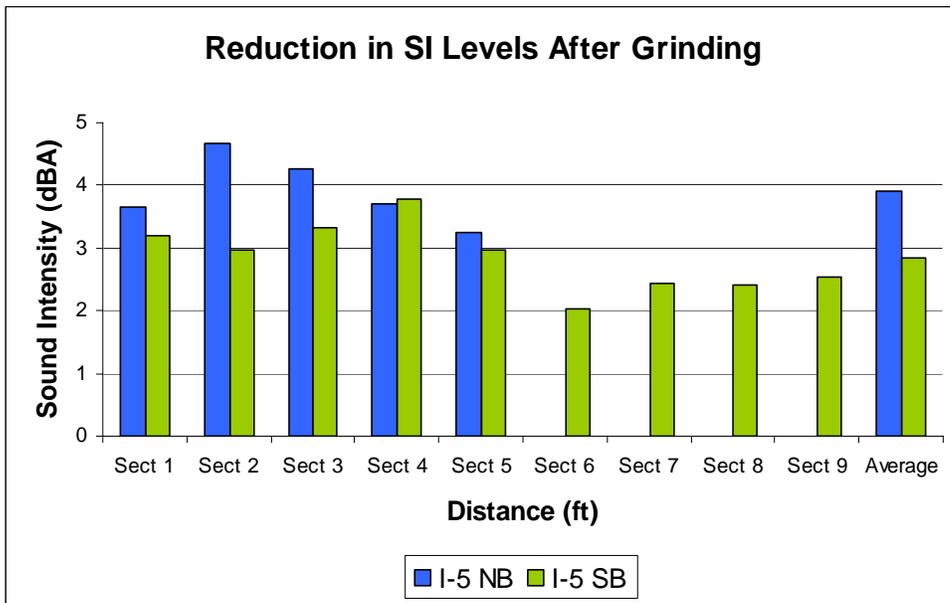


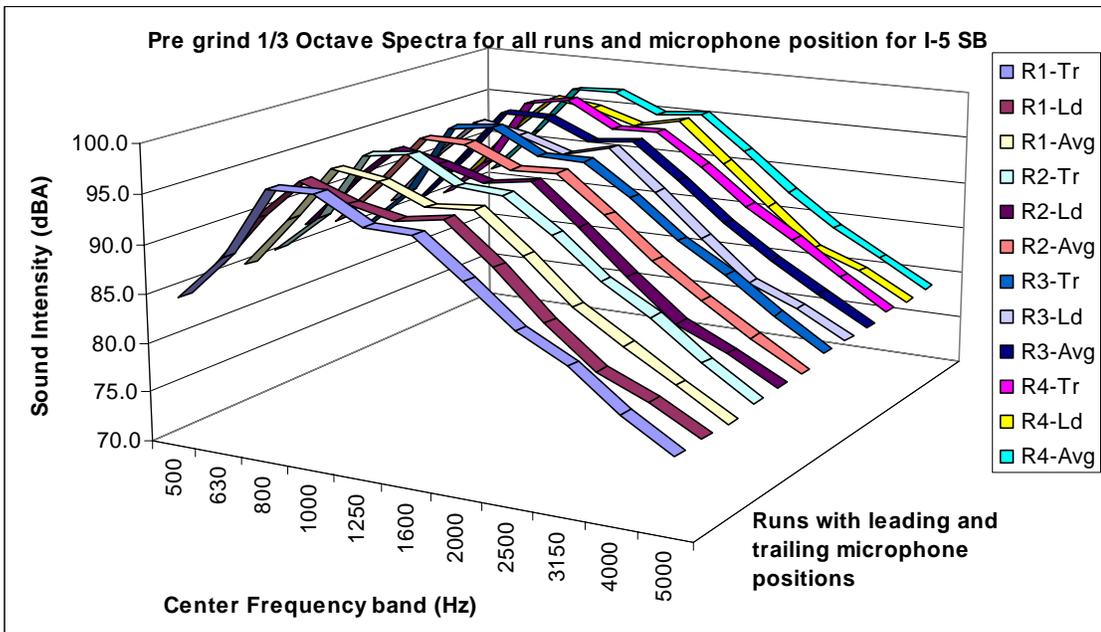
Figure 6. Reduction in noise levels after grinding on I-5 in Kern County.

#### *1/3-Octave Analyses for I-5 in Kern County*

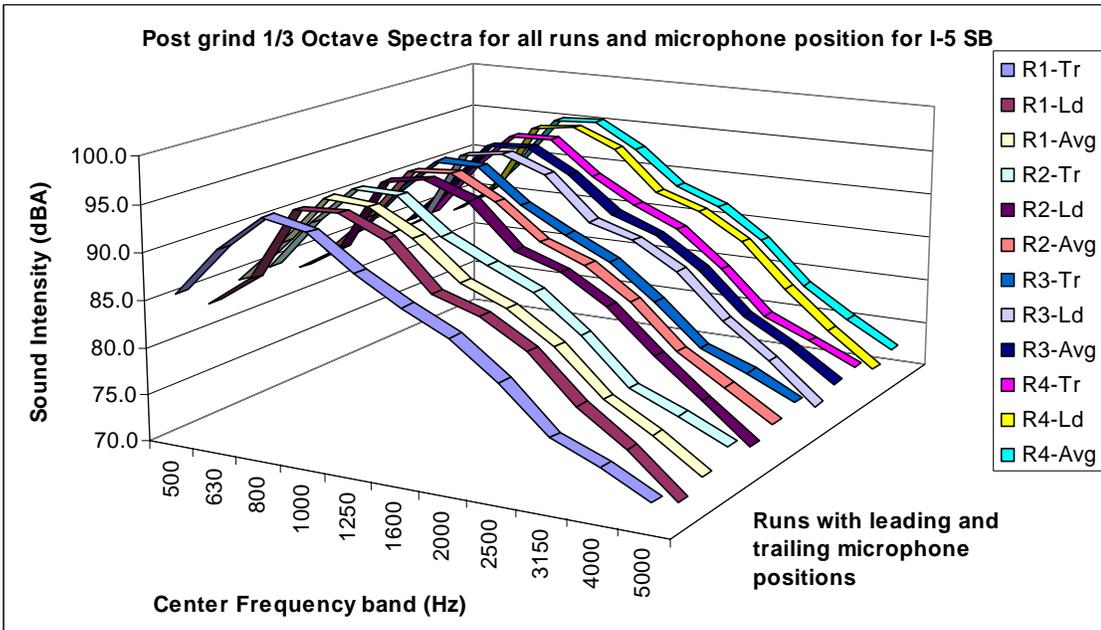
One-third-octave band analyses of SI levels were also conducted to determine the consistency among runs and delineate the effect of grinding on the individual 1/3-octave bands.

**Southbound Direction:** Figure 7 and Figure 8 show pre- and post-grind center frequency bands from 500 Hz to 5000 Hz for all four runs on southbound I-5. In the figure, each run (Rx) is represented by three SI curves, the first curve for the microphone at the trailing edge (Tr) of the tire, the second for the microphone placed at the leading edge (Ld), and the third representing the average for the leading and trailing edge microphones. As shown in these figures, the consistency of SI values among runs is very clear in the 1/3-octave band analysis.

Pre- and post-grind SI values at 1/3-octave bands were compared to determine the effect of grinding on the various individual octave bands. Table 3 and Figure 9 show pre- and post-grind 1/3-octave band spectra comparisons for the southbound direction. The greatest reductions for the 1/3-octave bands occurred in the 1600 Hz band, while the lowest reduction was for bands between 500 and 1000 Hz. However, all octave bands showed significant reduction levels within audible frequencies.



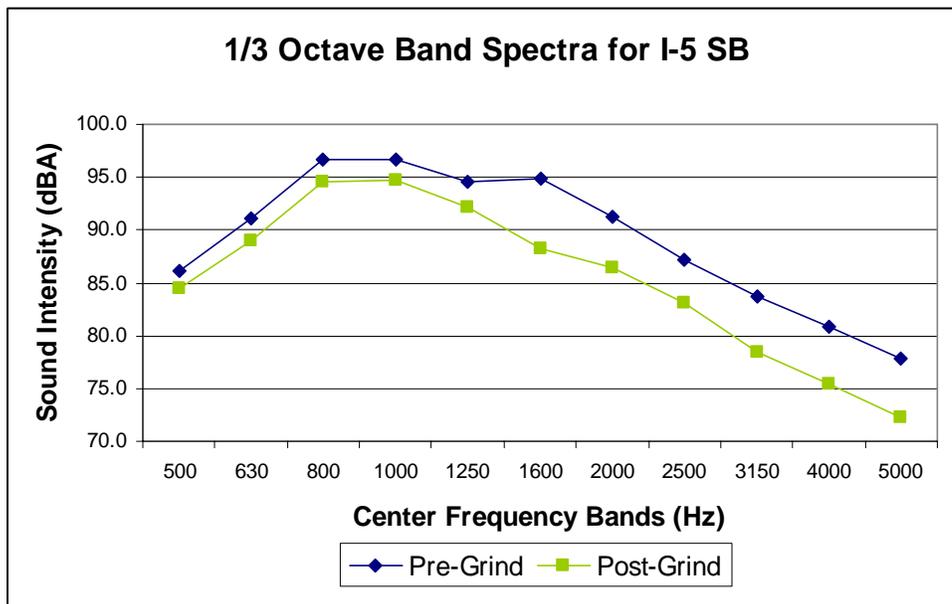
**Figure 7. Pre-grind 1/3-octave band spectra for Southbound I-5 in Kern County.**



**Figure 8. Post-grind 1/3-octave band spectra for Southbound I-5 in Kern County.**

**Table 3**  
**SI Values for Various Center Frequency Bands on**  
**Southbound I-5 in Kern County**

Center Frequency Band (Hz)	Pre-Grind SI (dBA)	Post-Grind SI (dBA)	Difference in SI (dBA)
500	86.2	84.4	1.8
630	91.2	89.0	2.1
800	96.7	94.6	2.1
1000	96.6	94.8	1.8
1250	94.5	92.1	2.4
1600	94.9	88.3	6.6
2000	91.2	86.4	4.8
2500	87.1	83.2	4.0
3150	83.7	78.5	5.3
4000	80.8	75.4	5.4
5000	77.9	72.3	5.6



**Figure 9. Pre- and post-grind 1/3-octave band spectra for northbound I-5 in Kern County.**

**Northbound Direction:** Figure 10 and Figure 11 show pre- and post-grind frequency bands from 500 to 5000 Hz for all the runs on northbound I-5. Again, each run (Rx) is represented by three SI curves, one for the microphone at the trailing edge (Tr) of the tire, another for the microphone placed at the leading edge (Ld), and the third one representing the average of the leading and trailing edge microphones. The consistency of SI values among runs is clear in the 1/3-octave band analyses.

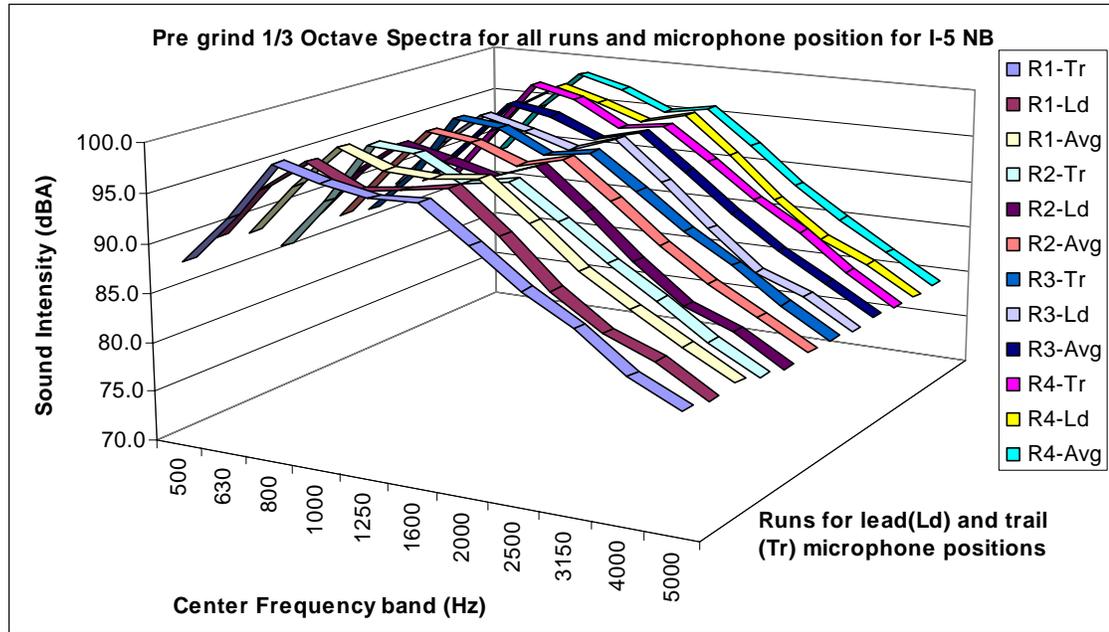


Figure 10. Pre-grind 1/3-octave band spectra for northbound I-5 in Kern County.

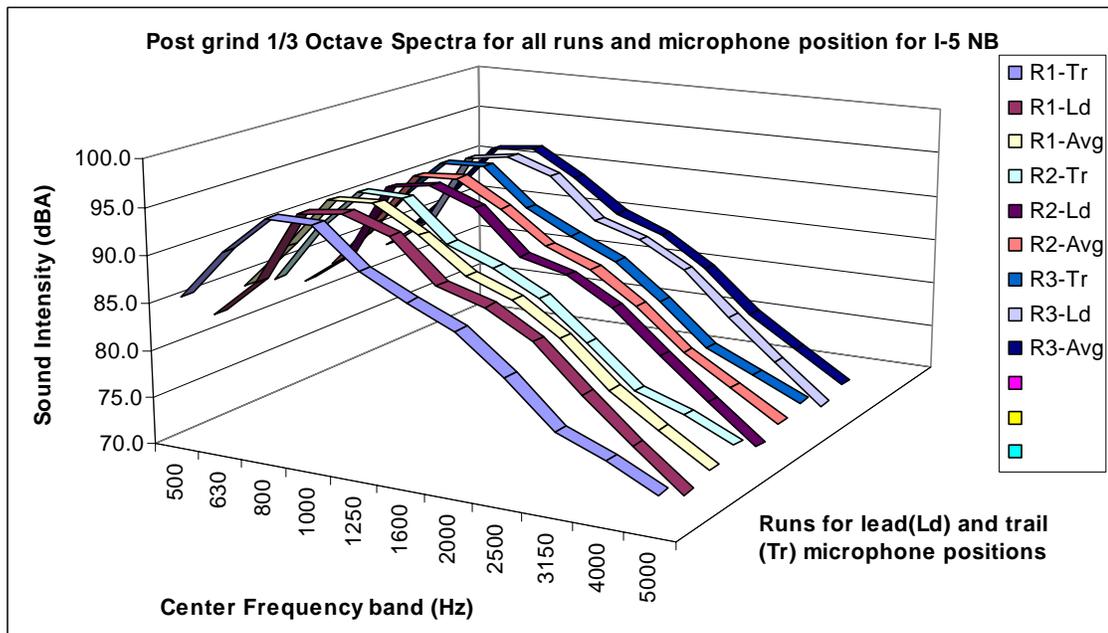
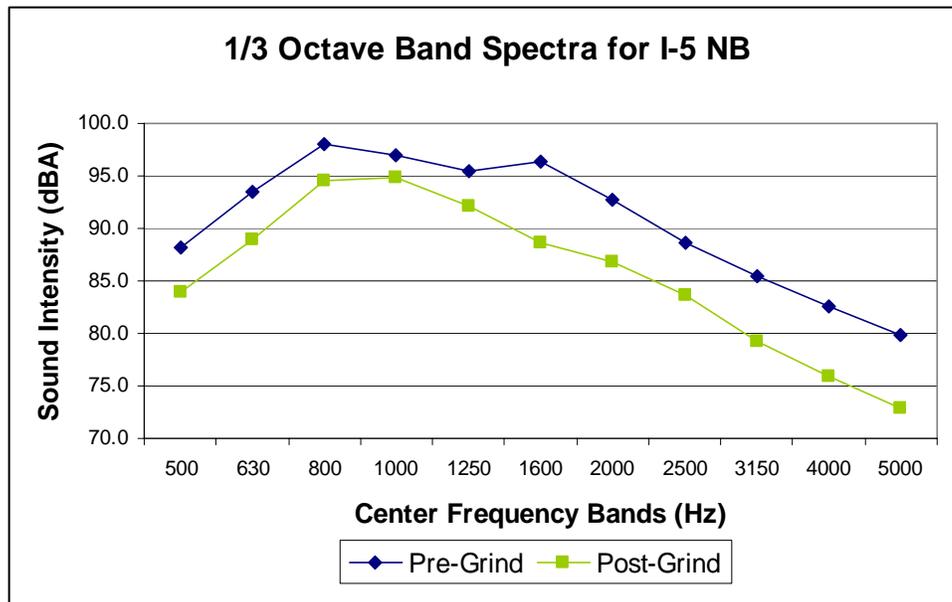


Figure 11. Post-grind 1/3-octave band spectra for northbound I-5 in Kern County.

Pre- and post-grind SI values at the 1/3-octave bands were compared to determine the effect of grinding on the respective octave bands. Table 4 and Figure 12 show pre- and post-grind 1/3-octave band spectra comparisons for the southbound direction of traffic. The greatest reductions on the basis of these 1/3-octave bands occurred in the 1600 Hz band, while the lowest reduction was in the 1000 Hz octave band. However, all octave bands showed significant reduction levels within audible frequencies.

**Table 4**  
**SI for Various Center Frequency Bands on Northbound I-5 in Kern County**

Center Frequency Band (Hz)	Pre-Grind SI (dBA)	Post-Grind SI (dBA)	Difference in SI (dBA)
500	88.2	84.0	4.2
630	93.5	88.9	4.5
800	98.1	94.6	3.5
1000	96.9	94.9	2.0
1250	95.4	92.1	3.3
1600	96.4	88.6	7.8
2000	92.7	86.8	5.9
2500	88.7	83.6	5.1
3150	85.5	79.2	6.3
4000	82.6	76.0	6.7
5000	79.8	72.9	6.9



**Figure 12. Pre- and post-grind 1/3-octave band spectra for southbound I-5.**

## CONCLUSIONS AND RECOMMENDATIONS

- There is a significant and easily audible reduction in SI levels (and hence in tire–pavement noise) after diamond grinding of PCC pavements.
- For the six routes on which pre- and post-grind data were available, the highest average reduction of 4.4 dBA was observed on I-5 near Richards Boulevard in Sacramento County, and the lowest reduction of 1.2 dBA was observed on State Route 60 (on a single test section) in San Bernardino County.
- An average 2.7 dBA reduction in SI levels was observed for all test sites.
- The highest reduction of 6.7 dBA, after grinding, for a single 440-ft (134-m) section was observed on I-5 in Sacramento County.
- The highest reductions in SI levels on a 1/3-octave band basis occurred in the 1600 Hz band, while the lowest reductions occurred in the 1000 Hz bandwidth. All octave bands showed significant reduction levels within audible frequencies.

It is therefore recommended:

- The long-term effect of grinding on tire–pavement noise should be monitored to determine the effectiveness of diamond grinding as a long-term strategy for noise mitigation in concrete pavements.
- All six routes, which consist of forty-two 440-ft (134-m) individual evaluation sections, should continue to be monitored to determine the progression of tire–pavement noise over a longer analysis period.

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## DISCLAIMER

This report reflects the observations, findings, conclusions, and recommendations of the authors. The contents do not necessarily reflect the official views or policies of the California Department of Transportation or the State of California.



# **Finding Buried Treasure With Diamond Grinding of a Concrete Pavement After Removal of an Asphalt Overlay**

Daniel P. Frentress<sup>1</sup>

## **ABSTRACT**

Many times in the past, an agency has covered a sound concrete pavement with an asphalt overlay to improve the ride which may have been a cost-effective solution, but with the recent increase in asphalt prices, the mill-and-overlay option is becoming too expensive. Current diamond-grinding prices can be half the cost of an asphalt overlay and if the old concrete is still structurally functional, then diamond grinding becomes a cost-effective solution and allows for the recycling of the asphalt millings for future asphalt projects.

This paper presents a case study of an actual project under construction that is scheduled to be completed in spring 2009. The area in question is a diamond-grinding project of underlying concrete pavement that took place after the removal of the existing asphalt overlay. The paper describes the selection process that the New Jersey Department of Transportation used to design and undertake this project as well as the construction issues related to completing the work on a night-only (Monday through Saturday) construction schedule.

The diamond-grinding contractor asked that the asphalt milling machines not cut into the concrete pavement, which means that some asphalt is being removed by the diamond-grinding equipment while profiling the old concrete pavement to a satisfactory ride.

The project is located on Highway 21 on the north side of Newark, New Jersey. Crisdel Group, Inc., of South Plainfield, New Jersey, is the prime contractor and Interstate Improvement, Inc., of Faribault, Minnesota, is the diamond-grinding subcontractor.

## **PROJECT SCOPE**

Highway 21 in Newark, New Jersey, known locally as the McCarter Highway, is an urban freeway. It has a combination of 50 percent elevated roadway using curbs and gutters for storm water drainage and about 50 percent rural Interstate design with paved shoulders and open ditch drainage into the Passaic River. The existing concrete pavement was constructed in three phases. First, in 1931 (1.4 mi [2.3 km]), again in 1958 (3.5 mi [5.6 km]), and completed in 1970 (1.0 mi [1.6 km]) using an old design with panels 73-ft (23.3 m) in length and steel mesh that was placed in each panel. This pavement is 9 in. (229 mm) thick and was placed on a 12-in. (305-mm) aggregate base. The majority of this roadway was placed over a 4-year period from 1958 to 1961 utilizing a long-panel design with stainless steel dowels on a 73-ft (23.3 m) joint spacing. The mesh was purposely placed in the top one-third of the pavement depth to help control top-down cracking from shrinkage of the concrete. This location of the mesh led to

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many of the partial-depth repairs (overrun by twice the bid quantity) and the desire to not mill too deeply into the old concrete pavement with the asphalt milling machines.

The contract was designed as a nighttime only construction schedule from Monday through Saturday nights between the hours of 8 p.m. and 6 a.m. One of the challenges on the project was the proximity of the steel mesh near the surface of the concrete, which was sometimes less than 2 in. in depth. This required all of the milling operations to be exact in order to leave as much of the old concrete as possible.

For the majority of this project, the roadway was three lanes with a 10-ft (3.1 m) outside shoulder and a 1-ft (0.3-m) inside shoulder. The area is 5.5 mi (8.9 km) long and extends northbound from milepost 5.01 to 10.57. In the southbound direction, the project extends from milepost 6.20 to 10.50 and is 4.3 mi (6.9 km) long.

This section of Route 21 received two microsurfacing treatments since it became a safety issue in the early 1990s with a high number of wet-weather accidents and less than desirable skid numbers. Because the 9-in. (229-mm) concrete pavement was structurally sound, only the concrete surface needed attention. A microsurfacing application was placed in 1993 that improved the skid numbers. But a second micro treatment of Nova Chip was placed in 2001 because of delamination of the 1993 microsurfacing. The total thickness removed in 2008 was about 1 in. (25 mm), with some localized areas of 2-in. (50-mm) maximum thickness of microsurfacing.

Because the second microsurfacing was suffering from delamination, Route 21 became a project again in 2008. It was the second of three projects on which the New Jersey Department of Transportation (New Jersey DOT) decided to try using precast panels for full-depth concrete repairs. I-295 in Burlington County was let on September 1, 2006, with 62,000 ft<sup>2</sup> (5,760 m<sup>2</sup>) of precast panels and diamond grinding as the final surface. Route 21 in Passaic County was let on January 31, 2008, with 44,000 ft<sup>2</sup> (4,088 m<sup>2</sup>) of precast panels and diamond grinding as the final surface. I-280 in Essex County was let on June 5, 2008, with 35,000 ft<sup>2</sup> (3,252 m<sup>2</sup>) of precast panels and an asphalt friction course as the final surface.

The roadway also utilized curb and gutters with inlet drainage structures that needed to be replaced or repaired.

It was determined that the old concrete was sound and in good condition, so the New Jersey DOT decided to try an experimental project on this roadway. They decided to remove the asphalt overlay and diamond grind the underlying concrete pavement. All new ideas for concrete pavement repair techniques including precast panels for the full-depth repair areas were used. The partial-depth patches were also done with a fairly new material, Crafcoc's TechCrete, a heated and flowable partial-depth repair material that stays flexible after placement.

## **DESIGN AND BID INFORMATION**

The project was designed as a simple asphalt overlay removal and a diamond grinding of the old concrete pavement underneath. Items included asphalt milling, catch basin reconstruction, slab stabilization, precast panels for full-depth repairs, and diamond grinding for the final ride and surface texture.

The underlying soils are silty sandy in nature and are susceptible to washouts under the transverse concrete joints. The roadway follows the river, and a concern for the designers was to fill any voids under the joints. Polyurethane grout was chosen for the slab stabilization work that was done at each joint along the 9.8-mi (15.8 km) of roadway, which meant that 400 joints were done in the northbound lane and 300 were done in the southbound lane.

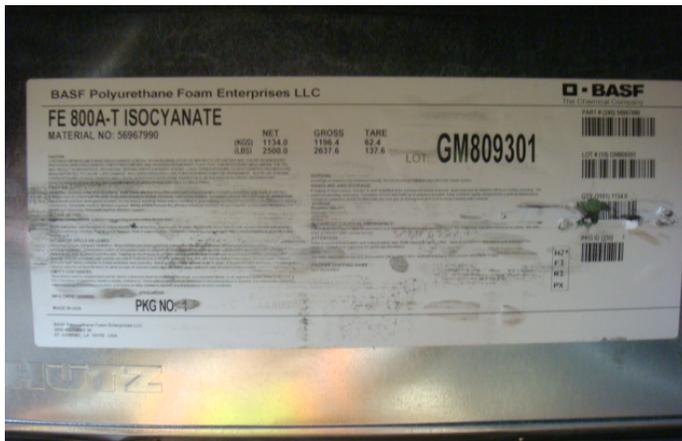
Letting was done on January 31, 2008, with four bidders participating. Crisdel Group, Inc., had the low bid of \$8,996,288.10, and the second lowest bidder was Della Pello Paving, Inc., at \$10,268,428.50, 14 percent greater than the low bidder.

The project is still under construction at the time of this writing with the final stages of striping and signing being completed soon. It is anticipated that the project will experience approximately 20 percent overrun due to the experimental nature of the project, the challenge of rehabilitating an old roadway, its proximity to the Passaic River, and the poor soils in the subgrade. The two main items of overruns were the slab stabilization (80 percent over) and the partial-depth repairs (100 percent over); most other quantities were much closer to the bid quantities.

## **REPAIRS**

### **Slab Stabilization**

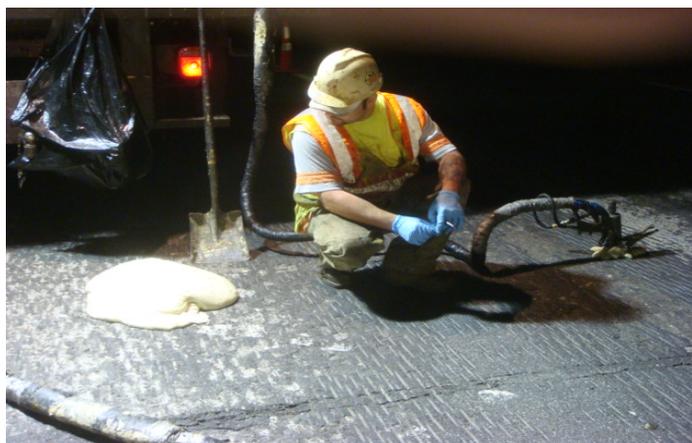
Slab stabilization was done by using a polyurethane grout supplied by BASF Polyurethane Foam Enterprises, LLC. The actual grout was FE-800A-T-Isocyanate (Figure 1), and the operation was done at each transverse joint to stabilize the joint area. The process for each 12-ft (3.7-m) lane involved drilling four small (less than 0.75-in. [19-mm]) holes at each concrete pavement transverse joint (every 73 ft [22.3 m] along the centerline) and then pumping polyurethane grout to fill voids under the concrete pavement (Figure 2). Also, it took more material than was included in the bid amount due to the soil in the area that was pumped out from under the transverse concrete pavement joints (Figure 3). Geo Tech Services, Inc., completed this part of the project in about 4 weeks. The bid called for 150,000 lb (68,039 kg) of material equaling a total cost of \$684,000. This item did overrun by at least 80 percent, which indicates a lot of void area under each joint location, with the worst voids located on the oldest concrete pavement which was built in 1931. This was probably due to the base aggregate, which had the largest maximum stone size above 1 in. (25 mm), as was used in 1931. The newer pavement section built in 1971 had a much better gradation (top size 0.75 in. [19 mm]) for the base aggregate and does not pump as easily as the 1931 base material. The largest amount of grout material was used in the older pavement section due to the base and the fact these pavements received the greatest number of total wheel loads due to the age of the 9-in. (229-mm) concrete pavement.



**Figure 1. Company name of polyurethane grout.**



**Figure 2. Grout being placed on both sides of a joint.**



**Figure 3. Grout leaving check hole.**

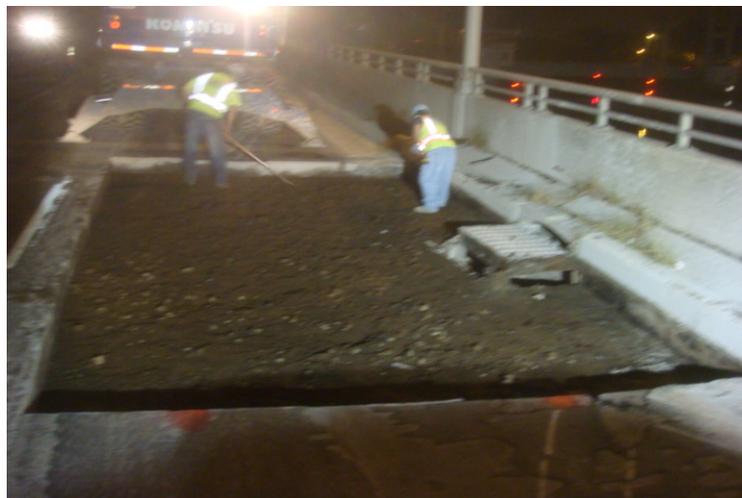
## Catch Basin Repair

A major problem for this project was the need to lower drainage inlet structures after removal of the asphalt overlay (Figure 4). A great deal of this roadway, including elevated sections, uses curbs, gutters, and catch basin inlets for storm water drainage (Figure 5). While many of the structures were old and in need of repair, more were repaired than were allotted for in the original estimate.

The other item that changed the nature of the project was the narrow inside shoulder at 1 ft (305 mm) or less. The diamond-grinding machines need a minimum of side clearance between 18 and 28 in. (457 and 711 mm) In the areas of narrow shoulder, New Jersey DOT decided to use an asphalt overlay. These areas included around 2 mi (3.2 km) of the northbound lanes and approximately 1 mi (1.6 km) of the southbound roadway.



**Figure 5. Failing catch basin.**



**Figure 5. Deteriorated catch basin after removal of pavement.**

## Precast Panels for Full-Depth Repairs

The Fort Miller Co., Inc., of Schuylerville, New York, built the 4,900 sq yd<sup>2</sup> (4,097 m<sup>2</sup>) of precast panels. The cost was \$472.89 per sq yd<sup>2</sup> for a total bid amount of \$2,317,161. Standard precast panel sizes were constructed as follows: 12 ft by 9 ft (3.7 m by 2.7 m), 12 ft by 10 ft (3.7 m by 3.0 m), and 12 ft by 14 ft (3.7 m by 4.3 m). If a cut in the field was made too large by the contractor's forces, then the contractor had to remove enough extra pavement to allow placement of a second precast panel. This did not happen too often in the field, but the same process can be used by engineers to allow some flexibility in patch area sizes during nighttime construction schedules. Extra precast panels were always available to provide this flexibility on this project.

Using the standard sizes as guides, full-depth repairs were marked in the field, and then the Fort Miller Company made each precast panel for every full-depth repair area. The field process involved the following steps:

1. Measure and order the precast panel.
2. Remove the old pavement (Figure 6).
3. Rough-grade and backfill the area with black recycled granite sand (Figure 7).
4. Check the depth using a simple template for depth off the edge of remaining pavement (Figure 8).
5. Tamp with a plate compactor sized 12 in. by 12 in. (305 mm by 305 mm).
6. Final grading is done with a metal screed for accuracy, and the grade was left 0.25 in. (6 mm) high (Figure 9).
7. Drill and grout dowel bars on transverse joints (four in each wheel path) and, when needed, longitudinal joints as well with No. 19 metric reinforcing bars (Figures 10 and 11), which was done with an HD-50 fast setting grout.
8. Place precast panels and grout into place with a bedding grout that sets in 2 to 3 hours (Figure 12).
9. Finally, diamond grinding was accomplished for final riding surface and friction.

The subgrade backfill material was obtained locally from the waste in the production of gravel for roads. This material has the ability to achieve 75 percent of its compaction upon placement and is easily compacted to a suitable level with plate compactors for the small or large patch areas.

The precast panels have strips of foam that are about 0.25 in. (6 mm) thick and 4 in. (102 mm) wide cast in the bottom of the panel to be embedded into the recycled granite sand subbase material (Figure 13).

The panels are lowered by a crane, and the eye hooks are removed after placement of the panel (Figure 14). To lock the panel in place, a grout was pumped through the holes near the dowels and under the panel to level for the slope of the roadway (Figure 15).



**Figure 6. Removal of old pavement.**



**Figure 7. Placement of sand backfill material.**



**Figure 8. Template for checking depth of sand backfill material.**



**Figure 9. Rail screeds in place for final subgrade check before placement of precast panel.**



**Figure 10. Grouting of dowels into existing pavement.**



**Figure 11. Full-depth area ready to receive precast panel.**



**Figure 12. Staging area on roadway for precast panels.**



**Figure 13. Placement of precast panel (notice black styrofoam on underside of panel).**



**Figure 14. Final placement of precast panel using four dowels per wheelpath.**



**Figure 15. Holes for grouting dowels and a section of damaged concrete that was filled with grout.**

## Partial-Depth Repairs

Partial-depth repairs had to be able to carry traffic in a very short amount of time, so the design called for the use of proprietary patching material poured into the partial-depth patch area. The product used by the contractor was a Crafcro, Inc., product called TechCrete (Figure 16). This product is a hot-pour repair solution that is different from conventional rigid repair methods. TechCrete remains flexible with a high tensile strength and has excellent adhesion to concrete surface. The final nature of this material is a rubber-like compound that bonds to the concrete surface to allow for movement but does not crack or de-bond. Normal cleaning and sandblasting operations have to take place ahead of the patching material (Figure 17). The material requires that the patch area be primed before placement of the TechCrete into the patch area (Figure 18). A friction layer of the black granite sand was added to the top of the patch material and the diamond-grinding equipment was able to cut through the tops of these patches without damaging any material. The challenge was with the diamond-grinding equipment picking up rubber debris from the cutting action, which congested the blades and pumps.



Figure 16. Crafcro product TechCrete used as partial-depth patching material.



Figure 17. Partial area after removal and sandblasting (note the height of the steel mesh).



**Figure 18. Area primed for placement of patch material.**

### **Diamond Grinding and Removal of Asphalt Overlay**

The diamond-grinding contractor, Northern Improvement Company (Figure 19), asked the asphalt contractor to leave a little of the asphalt on the concrete surface. If the asphalt milling machine went too deep, it would leave low pockets where the aggregate particles would break off. If these low areas would be greater than 0.75 in. (19 mm), it would take at least two passes of the diamond-grinding machine to cut to the bottom of the low spot. This left the project with an unfinished look, and some thin layers of asphalt were left after the diamond grinding was completed (Figures 20 and 21). A close-up side view of the concrete pavement, using a quarter for scale, after some heavy removal of the asphalt overlay is shown in Figure 22. The next picture shows the final diamond ground surface after removal by the diamond grinding machine (Figure 23). There are some areas of the project that have no asphalt left on the surface after diamond grinding (Figures 24 and 25).

The diamond-grinding contractor said that the material used for the partial-depth patches caused some problems with his equipment. When the patches were small (less than 2 ft by 2 ft [0.6 m by 0.6 m]), the rubberized material removed by the diamond-grinding machine was washed out of the blades and caused no problems. When the partial-depth patches were large (greater than 2-ft by 2-ft [0.6 m by 0.6 m]) the extra rubberized material got stuck between the blades, preventing cooling water from reaching the diamond blades correctly. Also, there was one incident when a bunch of material washed out and broke a trash pump on the collection system.

Although there was not a specification for ride on this project, the final ride is excellent and provides the public with an improvement over the old asphalt surface. The diamond-grinding contractor stated that a lot of the old concrete pavement material was removed at every other joint. This indicates a high degree of curl at the joints, which is to be expected with long panel lengths (73 ft [22.2 m]). Cut measurements at most joints were at the 1-in. (25-mm) level at every joint and as high as 1.25 in. (32 mm) at every other joint. The concern by the engineers was the depth of steel mesh from the final surface elevation after diamond grinding. To date, no problems have occurred.

The bid price for the removal of the hot-mix asphalt overlay was \$784,245 for 266,750 yd<sup>2</sup> (223,037 m<sup>2</sup>) at a unit price of \$2.94 per yd<sup>2</sup>. The bid price for the diamond grinding was \$1,963,280 for 266,720 yd<sup>2</sup> (223 012 m<sup>2</sup>) at a unit price of \$7.63 per yd<sup>2</sup>.



**Figure 19. Diamond-grinding equipment.**



**Figure 20. Asphalt left after diamond grinding was completed.**



**Figure 21. Some asphalt left in the wheelpath low areas.**



**Figure 22. Cut left after pass of asphalt milling machine.**



**Figure 23. Final surface after diamond grinding.**



**Figure 24. Showing normal diamond-ground surface with complete removal of asphalt.**



**Figure 25. Final completed project.**

## Joint Resealing

The joints were resealed with a hot-pour material after all repairs and diamond grinding were completed (Figure 26.) Both transverse and longitudinal joints were re-sealed.



**Figure 26. Joint resealing at night.**

## Removal of Diamond Grinding Slurry and Asphalt Millings

The prime contractor, Crisdel Construction, Inc., built a retention dike system near the project for collection of the diamond-grinding slurry waste. By placing a layer of plastic on the grade and using the asphalt millings to make a dike around a retention pond, the diamond-grinding contractor was able to unload his slurry quickly and safely. After the slurry dried via evaporation, the remaining aggregate chips minus 200 materials can be recycled into a gravel base. The asphalt millings can be either added to the gravel base or sold by the ton as recycled asphalt product into a future asphalt project.

## RIDE INFORMATION

The ride data shown in Table 1 shows that the ride on the existing microsurfacing was an average of 160.94 in/mi (2,543 mm/km) as measured by the New Jersey DOT in December 2007.

After completion of the removal of the asphalt layer, the ride increased due to the rough texture of the milled concrete surface. The average was 222.80 in/mi (3,489 mm/km) as shown in Table 2 in July 2008.

After diamond grinding was completed the ride was improved by an average of 30 percent over the initial International Roughness Index of 160.94 in/mi (2,543 mm/km). The final ride was an average of 112.00 in/mi (1,770 mm/km) as measured by the New Jersey DOT on February 11, 2009, as shown in Table 3.

**Table 1**  
**Ride Information for Microsurfacing Done in 2007**  
**(Tested December 18, 2007; Average International Roughness Index [IRI] = 160.94)**

Route	Direction	From-To MP-MP	Pavement Type	IRI Score (in/mi)		
				Left Wheelpath	Right Wheelpath	Average
021	N	7.20-7.30	Microsurfacing on RC	141.56	146.52	144.04
021	N	7.30-7.40	Microsurfacing on RC	133.34	139.62	136.48
021	N	7.40-7.50	Microsurfacing on RC	144.63	149.51	147.07
021	N	7.50-7.60	Microsurfacing on RC	120.16	121.11	120.63
021	N	7.60-7.70	Microsurfacing on RC	98.80	134.83	116.82
021	N	7.70-7.80	Microsurfacing on RC	128.82	146.52	137.67
021	N	7.80-7.90	Microsurfacing on RC	113.04	109.67	111.36
021	N	7.90-8.00	Microsurfacing on RC	130.69	118.70	124.69
021	N	8.00-8.10	Microsurfacing on RC	107.77	105.21	106.49
021	N	8.10-8.20	Microsurfacing on RC	177.91	233.98	205.95
021	N	8.20-8.30	Microsurfacing on RC	128.76	182.23	155.50
021	N	8.30-8.40	Microsurfacing on RC	120.24	139.95	130.09
021	N	8.40-8.50	Microsurfacing on RC	106.37	164.61	135.49
021	N	8.50-8.60	Microsurfacing on RC	123.96	169.56	146.76
021	N	8.60-8.70	Microsurfacing on RC	97.56	132.90	115.23
021	N	8.70-8.80	Microsurfacing on RC	200.07	277.11	238.59
021	N	8.80-8.90	Microsurfacing on RC	170.05	158.16	164.11
021	N	8.90-9.00	Microsurfacing on RC	170.65	178.25	174.45
021	N	9.00-9.10	Microsurfacing on RC	208.59	224.92	216.75
021	N	9.10-9.20	Microsurfacing on RC	173.76	143.12	158.44
021	N	9.20-9.30	Microsurfacing on RC	146.01	185.00	165.50
021	N	9.30-9.40	Microsurfacing on RC	140.01	143.86	141.93
021	N	9.40-9.50	Microsurfacing on RC	136.71	124.23	130.47
021	N	9.50-9.60	Microsurfacing on RC	119.21	123.48	121.34
021	N	9.60-9.70	Microsurfacing on RC	129.94	152.75	141.34
021	N	9.70-9.80	Microsurfacing on RC	148.27	151.51	149.89
021	N	9.80-9.90	Microsurfacing on RC	174.74	244.76	209.75
021	N	9.90-10.00	Microsurfacing on RC	202.73	183.57	193.15
021	N	10.00-10.10	Microsurfacing on RC	131.73	176.72	154.22
021	N	10.10-10.20	Microsurfacing on RC	133.65	156.69	145.17
021	N	10.20-10.30	Microsurfacing on RC	157.11	178.38	167.75
021	N	10.30-10.40	Microsurfacing on RC	196.52	247.53	222.03
Average IRI						160.94

RC = reinforced concrete

**Table 2**  
**Ride Information for Concrete Surface After Asphalt Removal**  
**(Tested July 28, 2008; Average IRI = 222.80)**

Route	Direction	From-To MP-MP	Pavement Type	IRI (in/m)		
				Left Wheelpath	Right Wheelpath	Average
021	N	7.20-7.30	Milled Micro-s/RC	160.17	174.47	167.32
021	N	7.30-7.40	Milled Micro-s/RC	186.95	237.28	212.12
021	N	7.40-7.50	Milled Micro-s/RC	187.02	242.93	214.97
021	N	7.50-7.60	Milled Micro-s/RC	189.43	234.06	211.75
021	N	7.60-7.70	Milled Micro-s/RC	195.73	260.36	228.04
021	N	7.70-7.80	Milled Micro-s/RC	244.40	274.81	259.61
021	N	7.80-7.90	Milled Micro-s/RC	192.15	282.84	237.50
021	N	7.90-8.00	Milled Micro-s/RC	202.26	185.43	193.84
021	N	8.00-8.10	Milled Micro-s/RC	169.48	223.38	196.43
021	N	8.10-8.20	Milled Micro-s/RC	295.89	375.34	335.62
021	N	8.20-8.30	Milled Micro-s/RC	173.84	252.54	213.19
021	N	8.30-8.40	Milled Micro-s/RC	150.29	273.63	211.96
021	N	8.40-8.50	Milled Micro-s/RC	137.03	168.14	152.58
021	N	8.50-8.60	Milled Micro-s/RC	115.40	205.44	160.42
021	N	8.60-8.70	Milled Micro-s/RC	138.14	187.42	162.78
021	N	8.70-8.80	Milled Micro-s/RC	209.89	243.05	226.47
021	N	8.80-8.90	Milled -Micro-s/RC	223.32	253.19	238.26
021	N	8.90-9.00	Milled Micro-s/RC	234.02	177.82	205.92
021	N	9.00-9.10	Milled Micro-s/RC	228.96	247.14	238.05
021	N	9.10-9.20	Milled Micro-s/RC	274.86	186.66	230.76
021	N	9.20-9.30	Milled Micro-s/RC	151.07	194.30	172.69
021	N	9.30-9.40	Milled Micro-s/RC	119.91	103.90	111.90
021	N	9.40-9.50	Milled Micro-s/RC	132.09	123.54	127.82
021	N	9.50-9.60	Milled Micro-s/RC	111.08	101.87	106.48
021	N	9.60-9.70	Milled Micro-s/RC	120.92	126.74	123.83
021	N	9.70-9.80	Milled Micro-s/RC	101.98	125.24	113.61
021	N	9.80-9.90	Milled Micro-s/RC	179.61	185.29	182.45
021	N	9.90-10.00	Milled Micro-s/RC	188.65	124.59	156.62
021	N	10.00-10.10	Milled Micro-s/RC	175.26	153.29	164.28
021	N	10.10-10.20	Milled Micro-s/RC	137.82	171.05	154.43
021	N	10.20-10.30	Milled Micro-s/RC	127.12	204.92	166.02
021	N	10.30-10.40	Milled Micro-s/RC	169.36	244.25	206.80
Average IRI						222.80

RC = reinforced concrete

**Table 3**  
**Ride Information for Final Surface After Diamond Grinding**  
**(Tested February 11, 2009; Average IRI = 112.00)**

Route	Direction	From-To MP-MP	Pavement Type	IRI Score (in/mi)		
				Left Wheelpath	Right Wheelpath	Average
021	N	7.20-7.30	Diamond-ground RC	98	88	93
021	N	7.30-7.40	Diamond-ground RC	103	113	108
021	N	7.40-7.50	Diamond-ground RC	100	95	98
021	N	7.50-7.60	Diamond-ground RC	112	106	109
021	N	7.60-7.70	Diamond-ground RC	123	118	121
021	N	7.70-7.80	Diamond-ground RC	95	95	95
021	N	7.80-7.90	Diamond-ground RC	109	80	95
021	N	7.90-8.00	Diamond-ground RC	126	89	108
021	N	8.00-8.10	Diamond-ground RC	223	179	201
021	N	8.10-8.20	Diamond-ground RC	100	91	96
021	N	8.20-8.30	Diamond-ground RC	131	127	129
021	N	8.30-8.40	Diamond-ground RC	94	90	92
021	N	8.40-8.50	Diamond-ground RC	85	101	93
021	N	8.50-8.60	Diamond-ground RC	66	99	83
021	N	8.60-8.70	Diamond-ground RC	100	85	92
021	N	8.70-8.80	Diamond-ground RC	107	91	99
021	N	8.80-8.90	Diamond-ground RC	137	122	129
021	N	8.90-9.00	Diamond-ground RC	161	186	174
021	N	9.00-9.10	Diamond-ground RC	98	86	92
021	N	9.10-9.20	Diamond-ground RC	126	107	117
021	N	9.20-9.30	Diamond-ground RC	137	88	112
021	N	9.30-9.40	Diamond-ground RC	100	89	95
021	N	9.40-9.50	Diamond-ground RC	128	124	126
021	N	9.50-9.60	Diamond-ground RC	106	105	105
021	N	9.60-9.70	Diamond-ground RC	103	110	106
021	N	9.70-9.80	Diamond-ground RC	134	131	132
021	N	9.80-9.90	Diamond-ground RC	110	100	105
021	N	9.90-10.00	Diamond-ground RC	108	101	104
021	N	10.00-10.10	Diamond-ground RC	119	113	116
021	N	10.10-10.20	Diamond-ground RC	157	128	143
021	N	10.20-10.30	Diamond-ground RC	101	110	105
021	N	10.30-10.40	Diamond-ground RC	120	116	118
Average IRI						112

RC = reinforced concrete

## **SUMMARY AND POTENTIAL FOR OTHER PROJECTS**

This project proved that you can effectively remove an asphalt overlay and diamond grind the old concrete pavement. The ride is superior to the old microsurface treatments (at 6 years), and the concrete pavement still has the structural ability to carry today's heavy traffic load. While this was a difficult project, it proved that an urban freeway with curb and gutter drainage can be rehabilitated with concrete repair techniques under a night-only construction schedule.

With the advancements in ground penetrating radar it should be easier today to determine if the old concrete pavements under an asphalt overlay are experiencing voids problems. If cores are obtained for concrete strength, it is common practice today to use dynamic cone penetrometers for subgrade strength through the core hole, which can help determine the structural ability of the subbase. Falling-weight deflector data can help pavement engineers determine the strength of composite pavement systems and the benefit from the old concrete pavement if values can be obtained before and after placement of the asphalt overlay. The use of high-speed vans for ride measurements can help engineers determine when to schedule rehabilitation work.

If the price of asphalt cement remains at its present high values, then the value of the asphalt millings has to be considered in the overall cost of a concrete rehabilitation project with an existing asphalt overlay. If, by taking cores, an agency can determine the soundness of the concrete, then concrete pavement restoration and diamond grinding can be cost-effective tools to restore ride and longevity to an old concrete pavement. The value of the old asphalt pavement can be used to offset the cost of removal and possibly some of the diamond grinding cost as well, depending on the thickness of the asphalt layer and the value of recycled asphalt product (RAP) in the local market. Using the bid tabs on this project, it was determined that a 1-in. microsurfacing overlay generated approximately 0.05 tons of RAP per sq yd<sup>2</sup>. A value of \$20 per ton for asphalt millings could generate \$1 per sq yd<sup>2</sup> per in. of asphalt overlay. This value can be taken into account at the time of bids by the private industry the same as it has been done in the past for asphalt mill and overlay projects. Unfortunately these asphalt millings in this project were used as a dike system to contain the diamond-grinding slurry and as such got too contaminated to be of value to asphalt contractors. The material will be used as recycled base material.

### **List of Owners and Contractors for This Project**

New Jersey DOT, District N1, Newark New Jersey

Michael Orłowski, Resident Engineer, HNTB Corporation, [www.hntb.com](http://www.hntb.com)

Ralph Hagy, Inspector, HNTB Corporation, [www.hntb.com](http://www.hntb.com)

Prime Contractor: Crisdel Group Inc., S Plainfield, NJ, [www.crisdel.com/](http://www.crisdel.com/)

Precast Subcontractor: Fort Miller Co. Inc., Schuylerville, NY, [www.fortmiller.com/](http://www.fortmiller.com/)

Stabilization Subcontractor: Geotechnical Services Inc., [www.geotechserve.com/](http://www.geotechserve.com/)

Diamond Grinding Subcontractor: Interstate Improvement Inc., Faribault, MN, [interstateimprovement.com/index.html](http://interstateimprovement.com/index.html)

Matleola Saw Concrete Inc.

# Development of the Next-Generation, Low-Maintenance Concrete Surface

Larry Scofield<sup>1</sup>

## ABSTRACT

In 2005, the Portland Cement Association, through the American Concrete Pavement Association, funded research to improve the noise performance of concrete pavements. The International Grooving and Grinding Association, through its affiliated contractors, supported the research effort through equipment development and test section construction.

The research was undertaken by Purdue University's Herrick Laboratories using their Tire Pavement Test Apparatus (TPTA). The TPTA is capable of testing any pavement texture that can be produced. This allows evaluation of texture designs that are not constrained by current construction capabilities or costs associated with construction and evaluation of field test sections. More importantly, the TPTA allows evaluation of textures without causing traffic control or safety issues.

Purdue's concrete pavement research was targeted on both new construction and pavement rehabilitation. Purdue's preliminary efforts focused on evaluation of the variables affecting tire-pavement noise generation characteristics of diamond-ground surfaces. This paper reports on the development and findings of that work.

The Purdue work evaluated the variables affecting construction of diamond-ground textures and the joint-slab effect associated with transverse joint noise generation. The findings of the Purdue work indicated that the geometric configuration of the blades and spacers used to construct diamond-ground textures was not the controlling factor in noise generation; rather the resulting fin profile was the most important factor. To produce a low-noise, diamond-ground surface required producing uniform and consistent fin profiles.

To verify this finding, a new surface was produced that consisted of a uniform fin profile design with essentially only negative texture. This surface texture produced the lowest tire-pavement noise levels in the research. The surface was then constructed in the field using actual diamond-grinding equipment to confirm the laboratory based study. A new surface, now called the Next Generation Concrete Surface (NGCS), was essentially implemented and is being constructed in test sections to evaluate its long-term performance.

NGCS is a term used to describe a category of textures that have evolved or will evolve through current research. The term may apply to several textures that evolve for both new construction and rehabilitation of existing surfaces. The desirable characteristics of such textures will be a very smooth profile coupled with good micro texture and excellent macro texture.

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To date, three field trials have been constructed and one competitively bid construction project. Friction testing and on-board sound intensity levels have been obtained at two of the sites and are reported herein.

## **INTRODUCTION**

Recognizing the national interest in quiet pavements, in 2004 the Portland Cement Association (PCA) and the American Concrete Pavement Association (ACPA) developed a program to research the surface characteristics of concrete pavements. This program, funded by the PCA and administered by the ACPA, was designed as a 3-year program that began in 2005. The International Grooving and Grinding Association (IGGA) and several of its members also provided financial and creative support for the program. The program had three main objectives: evaluation and development of a quieter diamond-ground surface, evaluation and development of new construction surfaces, and evaluation of the joint slap effect. Only the development of a quieter concrete diamond-ground surface is discussed in this paper.

## **PURDUE TIRE PAVEMENT TEST APPARATUS**

Purdue University's Herrick Laboratory conducted the research using their Tire Pavement Test Apparatus (TPTA). The TPTA (shown in the right side of Figure 1) consists of a 38,000-lb (17,237 kg) drum, 12 ft (3.7 m) in diameter, that makes it possible to test numerous types of pavement textures and compositions in combination with various tire designs. Six curved test-pavement sections fit together to form a circle. Two tires, mounted on opposite ends of a beam, are then rolled over the test samples at varying speeds while microphones and other sensors record data. As indicated in Figure 1, two wheel tracks were constructed on each of the six curved test panels allowing 12 surface textures to be tested in one setup. Speeds of 0 to 30 mi/h (0 to 48 km/h) can be tested, and test temperatures ranging between 60 °F and 80 °F (15.5 °C and 26.7 °C) are possible.

### **TPTA Equipment**

The left side of Figure 1 shows the diamond-grinding head that was constructed by Diamond Surfaces, Inc., of IGGA. This head was used to grind all the surfaces studied. It constructs a diamond-ground surface 8 in. (203 mm) wide. Typical diamond-grinding units grind paths 3 ft and 4 ft (0.9 and 1.2 m) wide and use 50–60 blades per ft (0.3 m). To fully “stack” a head can cost \$50,000 and take 6 to 8 hours of stacking time. The use of a small head, 8 in. (203 mm) wide, tremendously reduces the blade cost and setup time. When comparing different grinding blade/spacer configurations, these savings are an important consideration. The grinding unit replaces one of the wheel setups, as indicated in Figure 1. Once the surfaces are diamond-ground, the unit is removed, the test wheel apparatus is re-installed, and testing is conducted.

The Purdue TPTA was the innovative workhorse for the PCA/ACPA/IGGA surface texture efforts. This device allows textures to be produced and tested that may not currently be possible to construct with present day equipment. Additionally, testing can be accomplished without requiring traffic control or endangering workers or travelers.

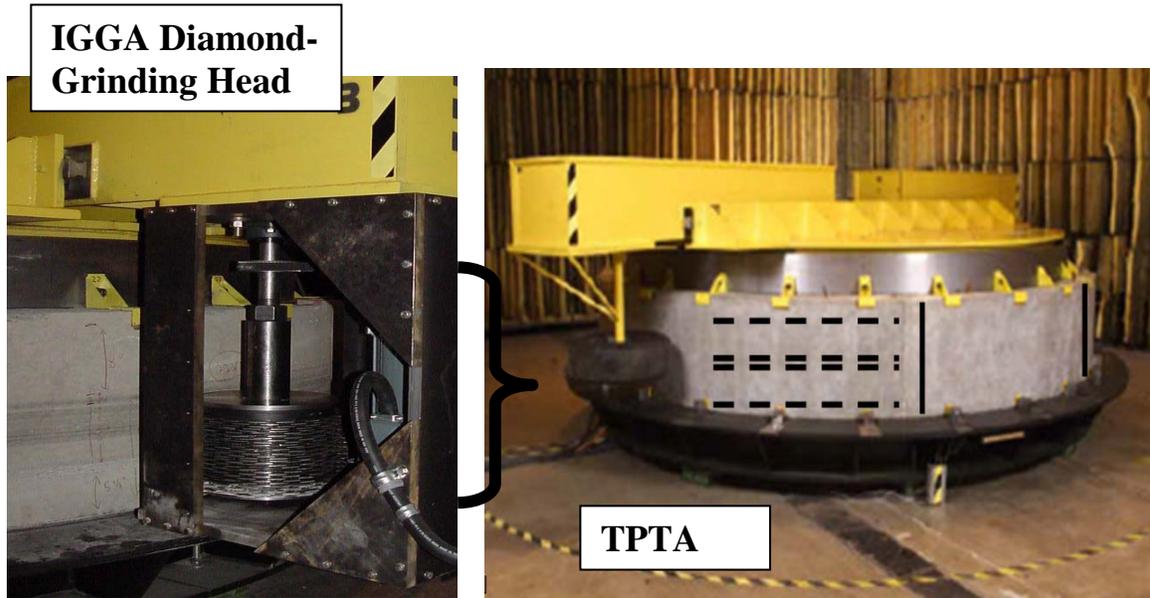


Figure 1. IGGA diamond-grinding head and Purdue Tire Pavement Test Apparatus (TPTA).

### TPTA Measurement Systems

Figure 2 shows the OBSI equipment used to measure tire-pavement noise and the RoLine laser used to measure texture profiles. As indicated in the left side of Figure 2, the OBSI equipment was mounted to the test-tire support frame. Since two tires are used during testing, it was possible to test with two different tire types at the same time.

The right-hand side of Figure 2 indicates the texture measurement system. Texture measurement was accomplished by removing one of the tire support frames and installing an arm to support the RoLine sensor.



Figure 2. OBSI and RoLine test equipment mounted to TPTA.

## Purdue Test Plan

The Purdue diamond grinding research was predicated upon varying the blade width and spacer width to develop the optimum grinding configuration. It was recognized that only one concrete mixture was to be used for all specimen preparation and that this would limit the optimization to the given mix and aggregate type. However, the belief was that any findings would be useful in understanding other mixes as well and that additional research could be pursued if necessary. Historically, the industry has constructed field test sections for research purposes, almost always varying the spacer and blade types. As such, the industry requested that Purdue approach their research by varying the blade and spacer widths and configurations.

After evaluating the range of blade and spacer widths requested by the industry, Purdue advised that no unique relationship could be found between spacer width, blade width, and spacer/blade configuration. Instead, it appeared that the controlling variable was the variability in the fin profile height resulting from the grinding process.

Figure 3 is a close-up photograph of the fin profile just after grinding and before any fins are knocked down due to traffic and winter maintenance operations. As evident in the photo, the harder aggregate stand “proud” in relationship to adjacent areas. Purdue indicated that it was probably this variability in fin profile that affected the tire–pavement noise generation. Textures with low variability were quieter than textures with high variability. In conventional diamond grinding, the resulting fin variability is affected by the blade/spacer configuration, the concrete mixture, aggregate type, pavement condition, equipment setup, and other influences, making it very difficult to control from an experimental standpoint.

To evaluate this hypothesis, it was decided to produce a texture with essentially no positive texture. That is, the surface would be diamond-ground smooth, and additional texture would then be imparted by grooving. In this way, the exact fin profile could be anticipated and controlled at the time of production, unlike conventional diamond-ground (CDG) surfaces, which are affected by many variables. Figure 4 shows one of these surfaces. It should be noted that the CDG surface shown in Figure 3 produces texture in the upward or positive direction, while the Purdue surface produces texture in the downward or negative direction. The Purdue texture, later called the Next Generation Concrete Surface (NGCS), was desirable from the standpoint that it was more of a “manufactured surface” and thus could be controlled as necessary from an experimental basis.



**Figure 3. Variability of fin profile on conventional diamond-ground surface on MnROAD I-94.**

When these new NGCS textures were tested on the TPTA, they produced the quietest of all surfaces tested to date. This was an epiphany in the research. It verified, for the first time, what the controlling factor was for tire–pavement noise generation of diamond-ground surfaces.



**Figure 4. Photo of Purdue negative texture profile (NGCS) and land area.**

## **TEST SECTION CONSTRUCTION**

### **Proof-of-Concept Field Testing**

The epiphany was soon confronted by reality. The Purdue grinding consisted of grinding a wheelpath 8 in. (203 mm) wide and 6 ft (1.8 m) long for each of the specimens. When grinding such small areas, the heat generated by the head is not excessive. However, when diamond grinding a pavement with a conventional machine, with a 3-ft or 4-ft (0.9 or 1.2 m) head, this is not the case. The typical 0.125-in. (3.2-mm) opening provided by a spacer between the grinding blades allows water to circulate between them, cooling them and removing grinding debris. This is an important consideration in production grinding. In addition, flush-grinding the surface prior to grooving requires approximately twice as many blades. For an 8-in. (203-mm) head such as Purdue's, this is not prohibitively expensive. To do it with a 3- or 4-ft (0.9-m or 1.2-m) grinding head could cost upwards of \$60,000, a risky (or investment) for an unproven strategy. The Purdue research indicated that the flush-ground/grooved texture could produce a quieter texture, but it could not verify whether it could be constructed with conventional equipment in the field.

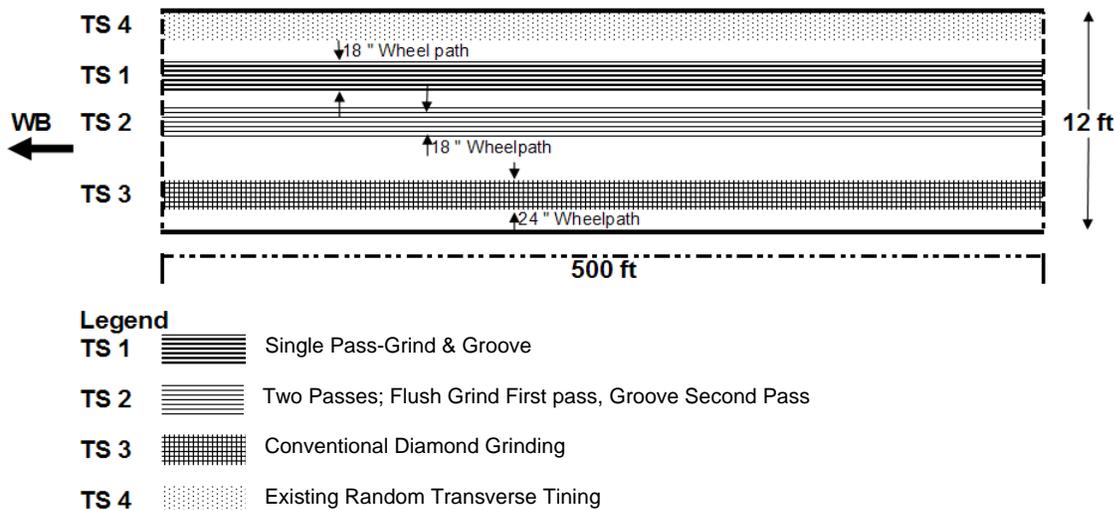
Prior to attempting field validation, two grinding/grooving configurations were developed and tested in the laboratory at Purdue. The first was a grinding configuration that used three smaller diameter blades stacked between two taller blades, and the pattern repeated across the grinding head. The taller blades were approximately 2 mm (0.079 in.) larger. This arrangement provided a single-pass operation that could grind the surface smooth and also groove it on approximate 0.5-in. (13-mm) centers in one pass of the machine. The smaller blades were used to flush-grind the roadway and provide micro texture, while the taller blades were used to create grooves. The Purdue work had also demonstrated the advantage of micro texture in reducing noise levels.

The second grinding configuration used the same smaller blades to “flush” grind the pavement in the first pass over the surface. A second pass was then made using the same taller blades with spacers between them to create on-center spacing of approximately 0.5 in. (13 mm). This second pass provided grooves similar to what was constructed with the single-pass configuration.

The purpose for the two different configurations, designed to achieve the same end result, was to allow consideration of either option by contractors. Some industry representatives did not consider the single-pass operation to be a viable option in a production environment due to excessive blade wear and the potential for ruining the head or blades. Many believed the two-stage process would be required. So both options were pursued. Both surfaces produced similar results on the TPTA, so field trials were pursued.

The opportunity to construct field test sections became a reality when the Minnesota Department of Transportation (MnDOT) allowed construction of the test sections shown in Figure 5 at the MnROADS Low Volume Road Test Cell Number 37. At approximately this same location, Diamond Surfaces, Inc., had equipment uniquely designed to construct the proposed sections. The equipment consisted of a diamond grinding unit with a 2-ft (0.6-m) head designed for curb cuts. This device not only allowed for fewer blades to be used but also was designed to allow quick blade changes. A head of blades could be changed in approximately 45–60 minutes versus 6–8 hrs.

The test strips indicated in Figure 5 represented a compromise between the ability to conduct OBSI testing at 60 mi/h (97 km/h) and requiring as few blades to construct a test strip. It was estimated that an 18 in. wheel track was the narrowest that that could be tested at 60 mi/h (97 km/h) and still ensure the test wheel was within the test strip.



**Figure 5. Proof of concept field test strips at MnROADS low-volume roads test area.**

Additionally, the two Purdue surfaces were to be compared to a CDG surface to assist in determining the benefit achieved by controlling fin profile. This resulted in the need to construct three diamond-ground surfaces.

The purpose of the test section construction was twofold: first, to verify the hypothesis that controlling the texture (i.e., fin) profile in contact with the tire could result in lower noise surfaces; and secondly, to verify that the results obtained using the TPTA could be reproduced in the field on real pavements using actual construction procedures.

The standard diamond-grinding wheel track (TS3 in Figure 5) was constructed with the blades and spacers existing on the equipment to eliminate the need to restack the head one more time. This resulted in TS3 being constructed 24 in. (610 mm) wide while TS1 and TS2 were constructed 18 in. (457 mm) in width to reduce the number of blades. The test sections were constructed on a 14-year-old PCCP that had originally been textured with random transverse tinning.

Wheel tracks TS2 and TS1 were constructed with new “flush” grind blades, which had been dressed to ensure they were essentially the same diameter so that a flat surface with micro texture could be produced. Taller new blades were also used which were approximately 2 mm (0.08 in.) larger in radius than the “flush” grind blades. TS 2 was constructed in two operations and TS1 in a single operation. The diamond-grinding unit constructed each wheel track in approximately 40 to 50 minutes. Equipment travel speed was on the order of 10 to 12 ft (3.0 to 3.7 m) per minute. Both the single pass and double pass procedures were successfully constructed in June 2007.

The findings, as discussed later, validated both that the Purdue texture was quieter, at the time of construction, than the CDG texture and that the Purdue TPTA results could be reproduced in the field using conventional equipment. With the validation of the TPTA results, the next step was to construct a full-width test section using a conventional diamond-grinding machine. This would allow trafficking of the test section as well as additional insight into the production side of the Purdue NGCS texture.

### **Mainline Construction**

The first opportunity to construct a full-lane-width test section occurred on the Chicago Tollway on I-355. At this site, both a CDG test section and a Purdue texture (NGCS) were successfully constructed in October 2007. The sections were 1,200 ft (366 m) long and one lane wide. This section of freeway was a newly constructed alignment that had not been open to traffic prior to constructing the test sections. The two-pass process was used to construct the Purdue texture.

The next opportunity to construct test sections occurred at MnROADs on the I-94 section. A section of Purdue NGCS two lanes wide by 500 ft (152 m) long was constructed in a single pass on a 14-year-old random transverse tined pavement in October 2007 on a new roadway. With the successful placement and performance of the two mainline sections, the ACPA officially named the Purdue textures the “Next Generation Concrete Surface” (NGCS). This naming occurred to describe a category of texture(s) that have or will evolve through current research. The term may apply to several textures that evolve for both new construction and rehabilitation of existing surfaces. The desirable characteristics of such textures will be a very flat profile coupled with good micro texture and excellent macro texture.

Two additional NGCS sections were constructed in fall 2008, one in Wisconsin and one in Kansas. The Kansas NGCS was constructed as part of the Kansas two-lift PCCP test project. Other Kansas textures evaluated consisted of drag textures, exposed aggregate, longitudinally tined, and a CDG section.

The Wisconsin project is unique in that it was the only project in which the NGCS section was bid as a normal construction project and not as a change order or section constructed by the industry. The Wisconsin section was constructed too late in the year to obtain meaningful test results. Additionally, the section was constructed through a town that had a posted speed limit of 25 mi/h (40 km/h) in the vicinity of the CDG section and 35 mi/h (56 km/h) in the vicinity of the NGCS section. OBSI testing is typically conducted at 60 mi/h (97 km/h).

All the NGCS sections were successfully constructed and were the quietest textures placed. The OBSI and friction characteristics are discussed in subsequent sections. The NGCS is still in the evaluation phase, and ways of making the surface more cost effective are being considered. Currently, this surface is considerably more expensive than CDG surfaces.

## **ON-BOARD SOUND INTENSITY MEASUREMENTS**

The ACPA has conducted OBSI testing on the test sections to evaluate their long term performance. The testing is conducted at 60 mi/h (97 km/h) using a dual vertical-probe OBSI system mounted to a Chevy Malibu with an ASTM SRTT test tire. This testing is used to represent the tire-pavement noise generation of the respective surfaces.

### **Pre-Opening to Traffic**

#### *Pre-Traffic OBSI Conclusions*

The pre-traffic OBSI results for the MnROADs and Illinois test sections are indicated in Figures 6 and 7. The proof-of-concept test section results conducted on the MnROADs low-volume road test sections are also presented. There are a number of things to note in these Figures: (1) the NGCS results were always lower than the CDG results; (2) the NGCS results are consistent while the CDG are variable; (3) the NGCS has a somewhat different spectral plot than the CDG, as indicated in Figure 7. Testing indicated that the NGCS surface is quieter below 1000 Hz and between 1000 Hz and 1600 Hz. Above 2000 Hz it is noisier. There is a significant drop in the spectrum at the 1600 Hz center band frequency for the NGCS surface.

It should be noted that no adjustments for the effect of temperature on OBSI levels have been made.

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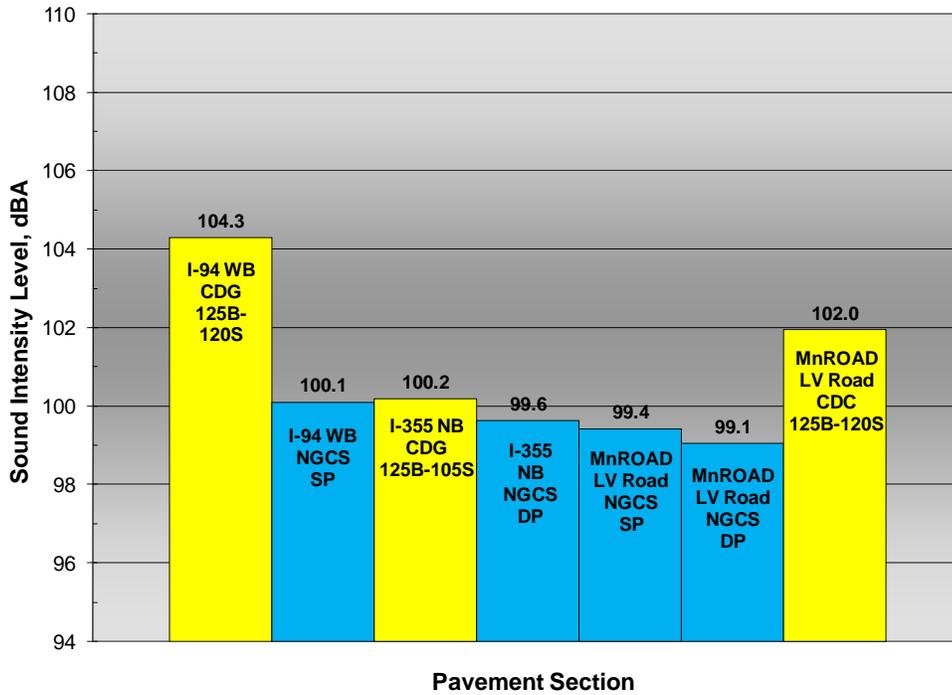


Figure 6. ACPA OBSI results for MN I-94 and IL I-355 sections.

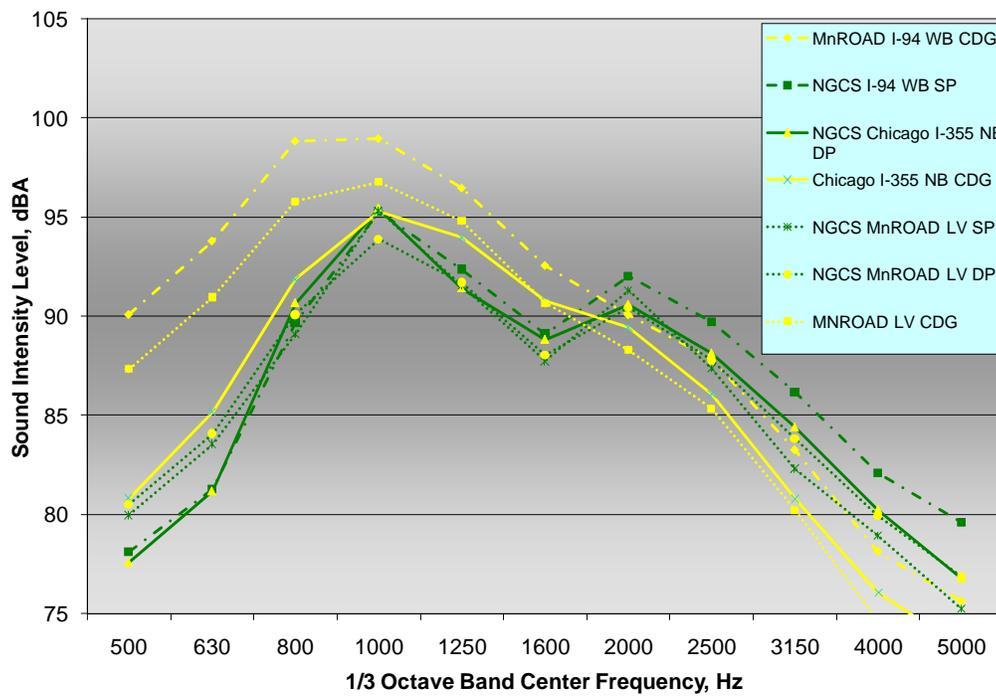
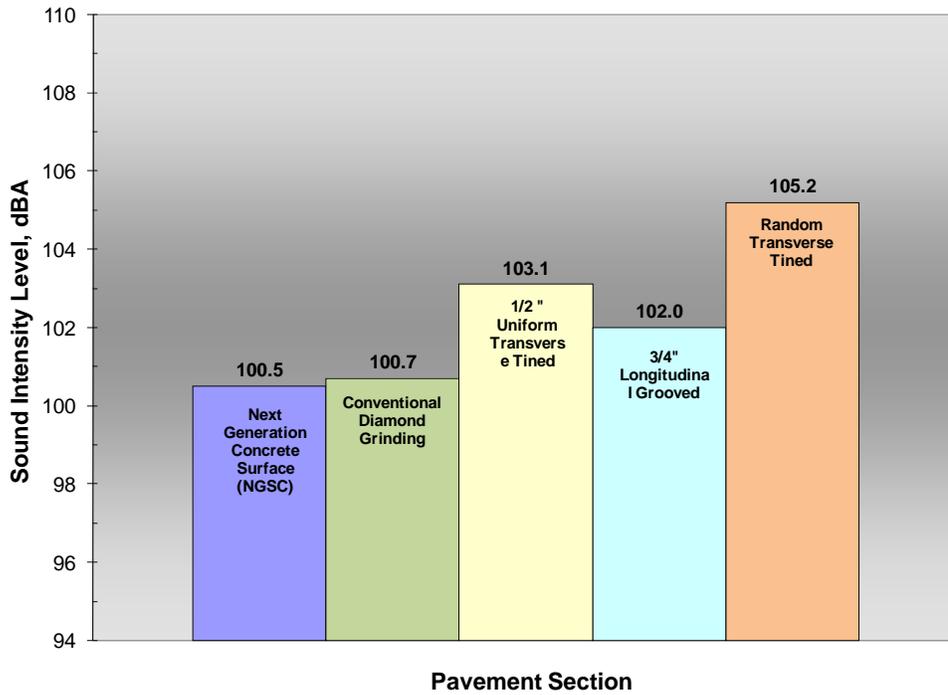


Figure 7. One-third octave spectral plots for MN I-94 and IL I-355 sections.

*Comparison to Other Textures on I-355 on 11-4-07*

OBSI testing of the diamond-ground surfaces was conducted on I-355 just prior to opening, and the results were compared to other textures on this same segment of roadway. The results are indicated in Figures 8 and 9. Figure 8 indicates the overall levels, while Figure 9 indicates the frequency spectrums. OBSI testing was conducted on all sections at the same time and temperature.

Figure 8 indicates that the NGCS surface was almost 5 dBA quieter than the random-transverse-tined surface. Figure 8 also indicates that the CDG and NGCS are similar in noise level. The longitudinal grooved drag texture was almost 2 dBA higher than the NGCS, suggesting that the grooves alone were not the solution.



**Figure 8. Chicago I-355 OBSI overall level results from 11-4-07.**

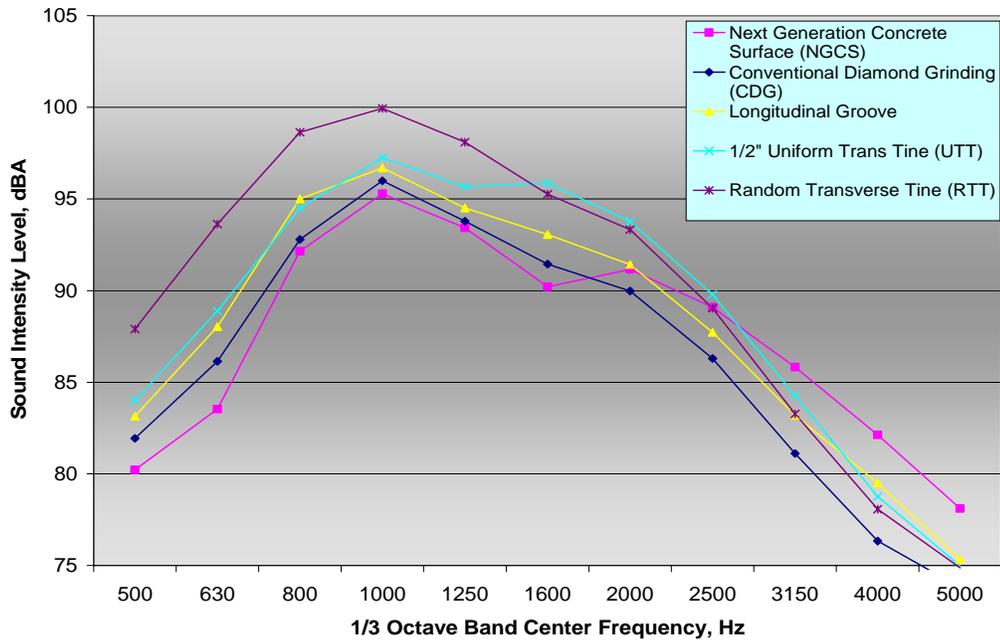


Figure 9. Chicago I-355 OBSI spectrum results of selected textures tested November 4, 2007.

### Post Opening to Traffic Test Results

Both the I-94 and I-355 test sections are located in areas with harsh winters and experience snow plow operations. Test results reported in this section represent performance after one winter of snow and ice control and interstate-level traffic. I-94 in Minnesota has a high percentage of truck traffic, while I-355 has relatively low average daily traffic and truck traffic.

Figure 10 indicates the OBSI levels for each of the two field test locations that are under traffic conditions. Both sites had received approximately 5 months of traffic at the time of testing. For each location and surface type, the oldest test result is displayed to the left of the most current result. The I-94 sections are indicated in the left half of the figure and the I-355 sections in the right half. The results shown are the average of three tests for each surface type. Figure 11 displays the spectral results for both locations.

When reviewing Figures 10 and 11, there are a number of things to note: (1) As evident in Figure 11, the NGCS frequency pattern is slightly different than the CDG. The NGCS has a characteristic dip at 1600 Hz and is typically lower in level at all frequencies below this dip and higher in level at all frequencies above the dip. This is consistent with the pre-traffic results. The NGCS produces a more broadband noise and may be less objectionable. (2) The overall level of the CDG and NGCS are more similar in the trafficked condition than in the as-constructed condition. This is a result of the wearing down of the CDG fins (see Figure 12, and contrast this with Figure 3). (3) The NGCS has produced more consistent results. That is, the textures are essentially at their final noise level at the time of construction and do not have to wear away to a “finished level.”

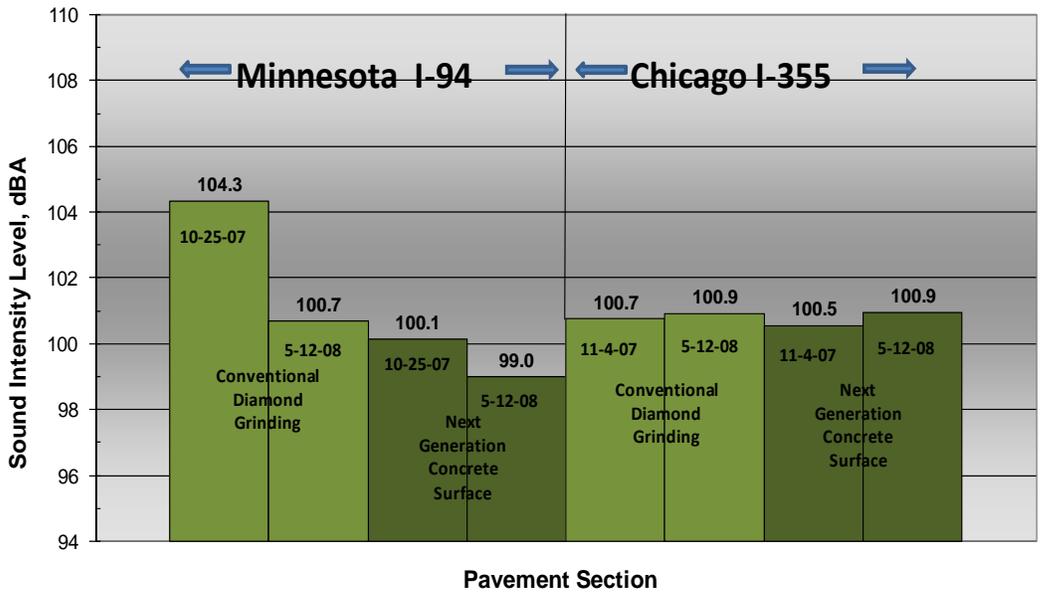


Figure 10. Comparison of CDG to NGCS OBSI levels through May 15, 2008.

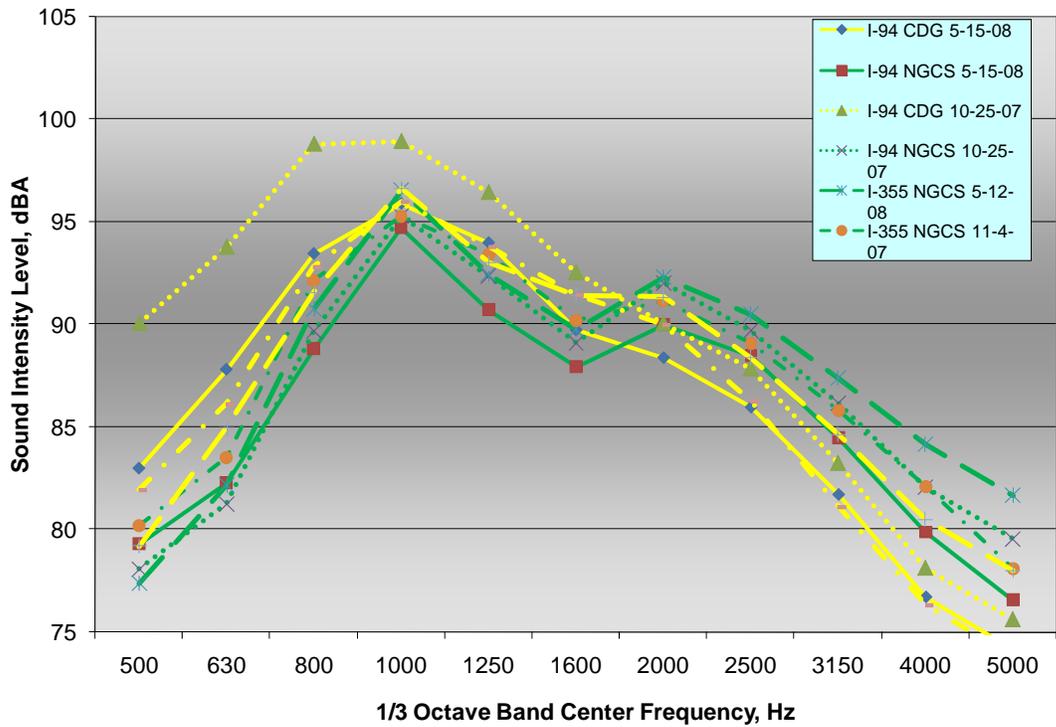


Figure 11. 1/3-octave spectra plots for I-94 and I-355 CDG and NGCS test sections.

As indicated in Figure 10, the CDG and NGCS sections were only 0.2 dBA different in level at the November 2007 testing, with the NGCS being quieter. During the May 2008 testing both surfaces produced similar results. The NGCS tested approximately 0.4 dBA louder in May than the November measurements, while the CDG tested 0.2 dBA louder. These differences are probably within the test repeatability of the OBSI equipment. In addition, this site location exhibits a slight joint slap. To date, the impact of the joint slap or diurnal change on test readings is undetermined.

It should be noted that the I-355 sections used 0.75-in. (19-mm) center-to-center groove spacing instead of the 0.5-in. (13-mm) center-to-center spacing developed at Purdue. This was to accommodate the use of the same grooving equipment that was used to construct the longitudinally grooved turf drag.



**Figure 12. MnROAD I-94 conventional diamond-ground texture after 5 months of traffic.**

## **FRICITION PERFORMANCE TESTING**

The friction performance of the MnROADs I-94 test section has been monitored by MnROADs since the construction of the test sections. Figure 13 indicates the time series behavior of the sections for both the ASTM ribbed (E501) and smooth (E524) tires. It should be noted that I-94 was closed to traffic on April 2, 2008, to allow construction of new test sections on the facility. So there is approximately 5 months of interstate traffic on the test sections. The “traffic” occurring after the measurements obtained on May 28, 2008, would have been construction traffic and winter maintenance operations. Therefore the October 31, 2008, test results should represent changes to the surface subsequent to the May 23, 2008, testing as a result of recent construction-related or winter operations.

The October 23, 2007, measurements reflect the friction of the surfaces just after original diamond grinding and just prior to opening to traffic. The random-transverse-tined section is adjacent to the test diamond-ground sections and is a 14-year-old surface.

One of the more remarkable aspects of data presented in Figure 13 is that the smooth tire results are higher than the ribbed tire results for the diamond-ground surfaces. This is not the case for the random-transverse tining, which exhibits the difference found on most typical surfaces. At this time this finding is not well understood by the author. However, for the NGCS section, the data are essentially identical between the May 28 and October 23, 2008, testing, as would be expected. This would suggest that the repeatability of the MnDOT testing is very good. Since the NGCS has large lands (see Figure 4), it would not be expected to change much due to construction traffic or winter maintenance operations.

The NGCS smooth tire results are essentially the same after 5 months of traffic as at construction. This would seem appropriate as the surface is essentially a “manufactured” surface at the beginning, and little change is expected.

The NGCS LITE is a recently developed surface to provide an economical renewable surface for the NGCS. This surface is intended to develop more micro-texture on the land area. It is a further development of the NGCS concept. The texture produced by the NGCS LITE can be produced in the original NGCS construction or it could be used to “touch up” the texture on the land if it ever became necessary. The touch-up process could be accomplished cost-effectively, since little material is being removed. It is intended to re-establish or improve micro-texture.

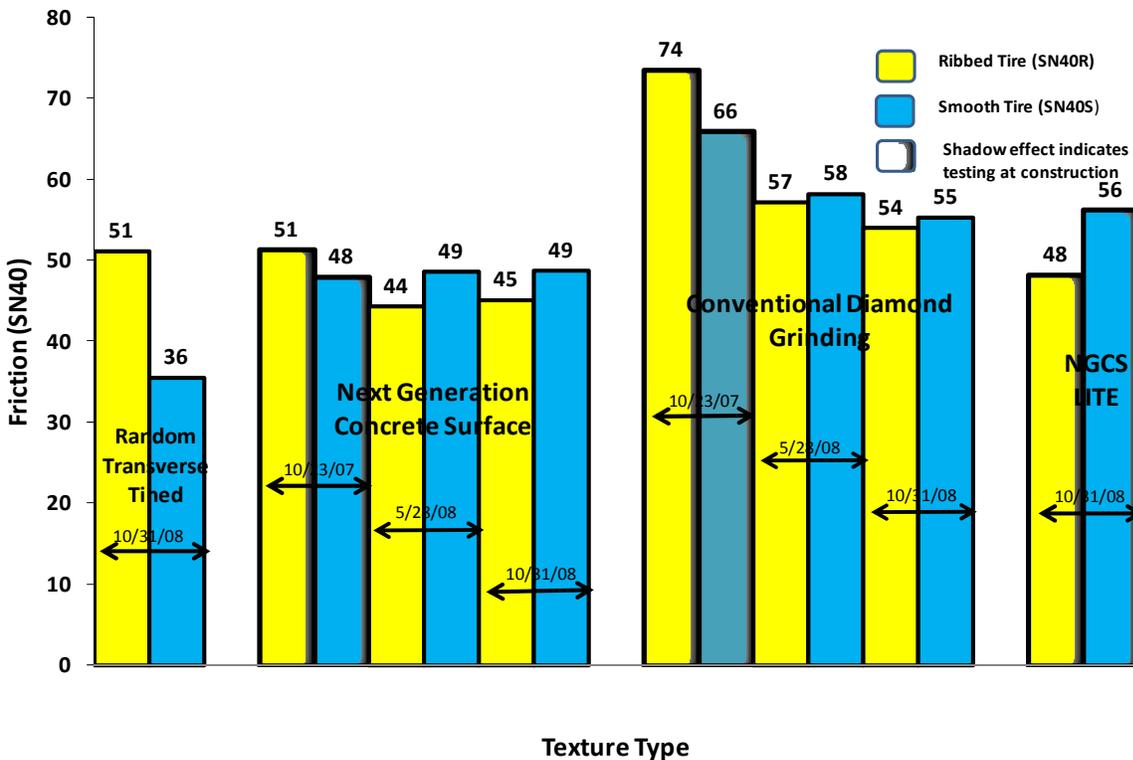


Figure 13. Friction (SN40) as a function of surface texture and time.

## LESSONS LEARNED

### Acoustic Longevity of Diamond-ground Surfaces

One of the questions that arose during evaluation of the NGCS was the expected acoustic longevity. Since it has been less than a year in implementation, the question could not be directly answered. Instead as an alternative, it became of interest to benchmark the existing CDG acoustic longevity. Although this maintenance strategy has been around since the 1960s, acoustic longevity curves were not readily available, so ACPA attempted to establish some findings

With the advent of noise measurement technologies such as OBSI, introduced by Caltrans into the highway industry in 2002, it became possible to develop acoustic longevity curves for selected pavements fairly efficiently. The first ACPA attempt at this occurred during summer 2008 on pavements in Kansas. The Kansas Department of Transportation provided a list of projects and a suggested testing scheme to the ACPA for OBSI testing. The selected pavements were intended to represent pavements of similar type, joint design, environment, and traffic, but of various ages. Pavement ages up to approximately 10 years were evaluated. Limestone is the predominate aggregate type found in Kansas and was used on these pavement sections. The top size aggregate is 0.75 in. (19 mm) to reduce D-cracking potential.

Seventeen pavements were tested. The results indicated in Figure 14. As indicated, there is a poor  $R^2$  with the regression equation, suggesting that the data are randomly associated and no trend exists. This indicates that the acoustic performance of CDG remains almost constant throughout its early life. As noted, the data only include pavements up to 10 years old. The pavements selected and tested were all doweled pavements with little or no faulting, of uniform joint design, and had compression-sealed joints.

The acoustic durability of the NGCS surfaces placed to date has provided experience similar to the CDG surfaces.

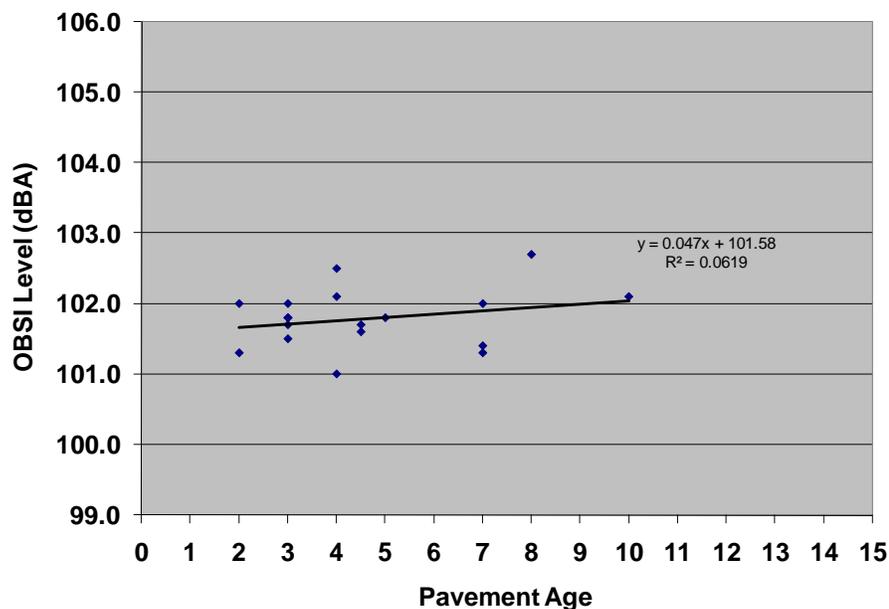


Figure 14. Acoustic durability of conventional diamond-ground projects in Kansas.

## Anisotropic Friction Behavior

The NGCS texture consists of a flush ground surface that has grooves on 0.5-in. (13-mm) centers. Although currently the grooves do not provide a significant benefit from a noise perspective, there is a belief that they would provide additional benefit in regards to wet weather accidents.

Historically, studies have indicated that grooved pavements have demonstrated reductions in wet-weather accidents, often times with little or no change in ribbed tire friction values. The conundrum of the increased safety with little or no additional apparent improvement in friction value has always been difficult to explain.

To investigate this further, the IGGA contracted with MACTEC, Inc., to conduct friction testing using the California Test Method 342. This rather unique device, illustrated in Figure 15, allows friction to be measured at various angles to the centerline direction over a reasonable area. With this approach it is possible to gain insight into the anisotropic friction behavior of selected textures.

One end of the device is attached to the hitch of a pickup truck with a hitch that allows the device to pivot from directly behind the vehicle (e.g., in the direction of traffic) to 90 degrees to the centerline. Testing is conducted by lifting the wheel to 6 mm (0.24 in.) above the pavement, attaining a speed of 50 mi/h (80 km/h), dropping the wheel to the pavement, and measuring the distance the wheel travels along the pavement.

Once the wheel is dropped, there is no additional energy supplied to the wheel, so the kinetic energy of the wheel is transformed into potential energy of the springs that attempt to restrain the wheel. The distance traveled is a function of the friction level and kinetic energy of the wheel. Since the kinetic energy of the wheel is always known and constant, the distance relates directly to the friction level of the surface under investigation.

Testing was conducted at five angles (0, 15, 30, 45, and 90 degrees) at each test location. Three test locations were obtained for each surface type. For each test location, except the Astro-turf, the 0-degree test was repeated upon completion of the 90-degree test to evaluate repeatability of the equipment. During the evaluation, 117 friction tests were conducted.

A shortcoming in the data collection effort was that the test procedure requires testing to be conducted at temperatures above 40 °F (4.4 °C). Because of the time of year in which testing was conducted, this was not possible; testing temperatures were in the range of 33–35 °F (0.5 to 1.6 °C) on pavements of variable wetness.

To minimize the viscosity problem associated with the lower temperatures, friction was expressed in terms of a friction index as indicated in Figure 16. The friction index is derived by dividing the friction value obtained at the specified angle by the friction value obtained at 0 degrees (i.e., the direction of travel). Friction indexes greater than 1 indicate an increase in friction compared to the direction of travel, and lower indices, less friction.

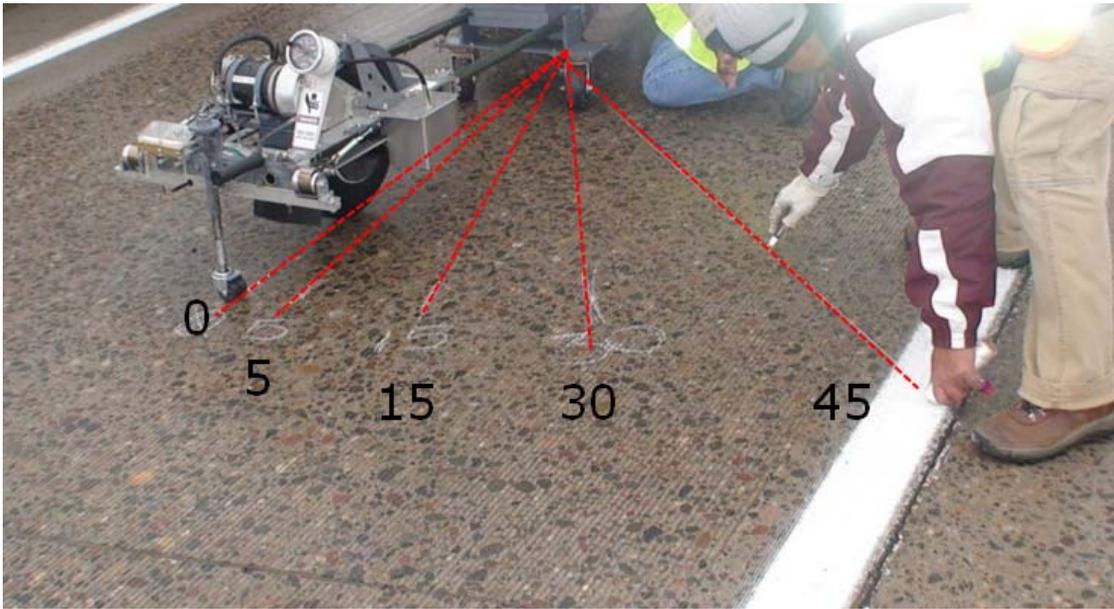


Figure 15. Photo of California CT-342 test device and selected angles of testing.

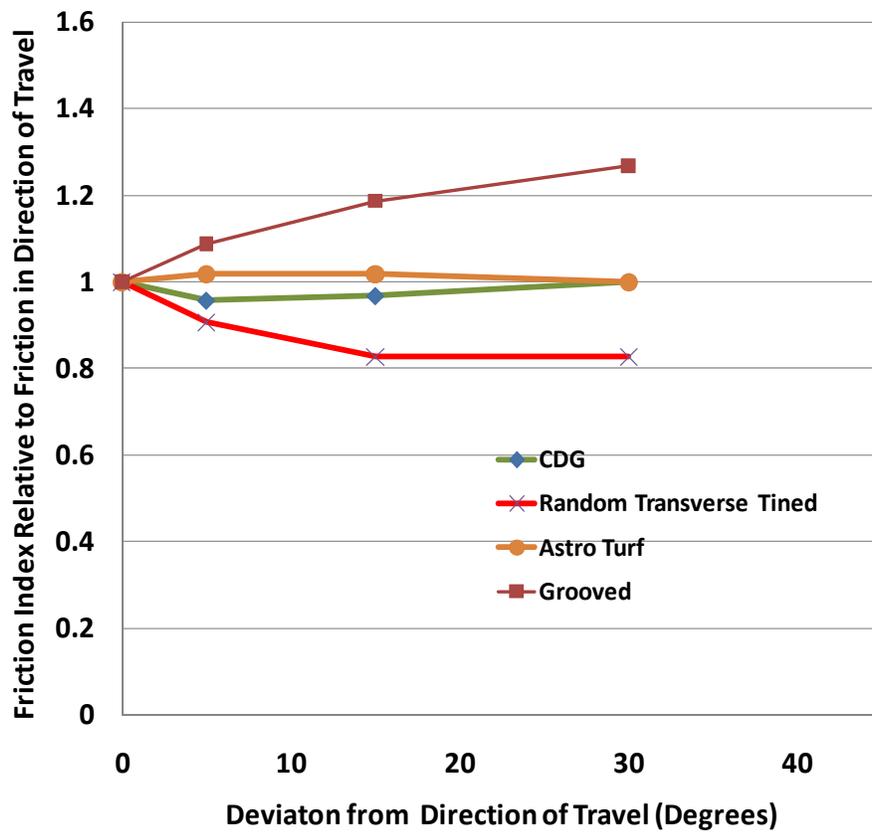


Figure 16. Friction index as a function of deviation from the direction of travel (uncorrected for cross slope).

The grooved textures exhibited an increase in friction as the device was oriented at an angle to the direction of travel (i.e., anisotropic behavior). This would suggest that the grooves are providing additional benefit for vehicles attempting to lose control since the friction increases. The Astro-turf and CDG, on the other hand, appeared to behave more like isotropic surfaces in regards to friction and indicated no apparent difference in direction. The random-transverse-tined pavement decreased in friction in regards to increasing deviation, which is consistent with what would be expected as the tines are already at right angles to the direction of traffic.

At this time these results should be considered preliminary until the experiment can be repeated under more favorable environmental conditions that allow complete adherence to the test procedure and employ replication of the results. It should also be noted that the CT-342 test method uses glycerin on the surface in the test method.

### **NGCS Lite—The Renewable Surface**

As previously mentioned, the NGCS LITE surface was developed to provide additional microtexture on existing NGCS surfaces should it become necessary to do so. With the large land size of the NGCS surface, the texture wear has been assumed to be less than occurs on CDG surfaces. As such it should have extended life in comparison to CDG. The NGCS LITE surface provides an easily renewable surface that can be “touched up” in less time and costs than a CDG surface. Very little material is removed to create this surface, providing a significantly faster operation. It is intended as a perpetual surface strategy.

The first test section of the NGCS LITE surface was constructed in October 2008, and it became too cold to use proper OBSI equipment. Noise results will be available in spring 2009.

### **SUMMARY AND CONCLUSIONS**

The NGCS diamond-ground surface, although only 1 year in implementation, has successfully demonstrated that it is a low-noise concrete surface. The NGCS, resembling a “manufactured surface,” provides its low-noise benefits when initially constructed and does not require a wear-in period to break the fins down. In the test sections constructed to date, the NGCS begins approximately 1 to 4 dBA quieter than a CDG surface and is approximately 0 to 1 dBA quieter after the 1st year. More time is necessary to establish the acoustic performance of the NGCS pavement, but as with CDG surfaces, the acoustic performance is not expected to change within the first 10 years of its construction when implemented on well-designed concrete pavements.

The early friction results of CDG surfaces have been superior to the NGCS surface. The NGCS surface smooth-tire results have not changed since construction, while the CDG, which started out much higher, is decreasing. At 1 year, the CDG still provides excellent friction results, as does the NGCS.

The potential benefit of anisotropic friction behavior of the NGCS (longitudinally grooved) surface needs to be further evaluated and verified as this may provide additional safety to the traveling public.

The ability to improve and maintain the NGCS surface over time is an important advantage, as it provides a renewable maintenance strategy that can be economically constructed. The efficacy of the perpetual surface strategy will require continued evaluation during 2009.

# Tire–Pavement Noise Results From California PCCP and HMA Pavements

Erwin Kohler,<sup>1</sup> Linus Motumah,<sup>2</sup> Bruce Rymer,<sup>3</sup> and John Harvey<sup>4</sup>

## ABSTRACT

Traffic noise generated by tire–pavement interaction is a matter of major concern for the California Department of Transportation (Caltrans). Research is underway in California and other States to evaluate tire–pavement noise characteristics of both concrete and asphalt pavements using the on-board sound intensity (OBSI) method, which allows for detailed characterization of noise levels at the source. In California, both concrete and asphalt pavement research studies are being conducted by the University of California Pavement Research Center (UCPRC) in collaboration with and funding from Caltrans. The concrete pavements and bridge decks study involves a total of 144 sections in different regions throughout the State. The surface textures evaluated in the study comprise of longitudinal tining, diamond grinding, diamond grooving, and burlap drag. Preliminary results indicate that diamond-ground surfaces can be the quietest of the concrete pavement surface textures. With only part of the test sections analyzed, OBSI levels from California concrete pavements range between 101.2 and 107.3 dB(A). The asphalt pavement research evaluates tire–pavement noise characteristics and performance properties of about 70 sections from throughout the State. This study considers acoustic and structural performance of four main asphalt surface types: open-graded asphalt concrete (OGAC), rubberized open-graded asphalt concrete (RAC-O), rubberized gap-graded asphalt concrete (RAC-G) and dense-graded asphalt concrete (DGAC). OBSI measurements indicate that average noise levels increased by 1.3 dB(A) from 100.8 dB(A) to 102.1 dB(A) over the 2-year period. Overall, a noise level of around 100.0 dB(A) measured at 60 mi/h (96 km/h) using the OBSI method appears to be a reasonable goal for both concrete and asphalt quieter pavements, based on the UCPRC data and other studies. Further analysis on the data being collected will answer questions about acoustic durability of different types of concrete and asphalt pavements.

## INTRODUCTION

Considerable developments in the area of tire–pavement noise have occurred in California. Research in this area is managed through the California Department of Transportation (Caltrans) Quieter Pavement Research (QPR) program, under the overall technical direction of the Quieter Pavement Task Group, which includes the Divisions of Pavement Management, Environmental Analysis, Engineering Services, and Research and Innovation. The work started in the late 1990s with the construction of highway noise monitoring sections, and most notably with the development of the on-board sound intensity (OBSI) method. As part of the Caltrans QPR program, the

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University of California Pavement Research Center (UCPRC) adopted and evaluated the OBSI method in 2005, and began testing asphalt pavement sections in early 2006 (1, 2).

There are reports of complaints about traffic noise dating as far back as the time of the Roman Empire. The “*roar of the iron-tired wheels*” in London around 1870 has been documented (3). In the last decade traffic noise has become a growing public concern, while at the same time it has become more obvious that tire–pavement noise now constitutes a major problem in traffic noise in industrialized countries (4). Tire–pavement noise dominates highway noise at speeds as low as 35 mph. In recognition of this concern, the design, construction, and preservation of quieter pavements to reduce noise has received increasing attention in California as well as nationally and internationally. With the short-term benefits of quieter pavements at least partially documented, recent attention has focused on developing a better understanding of their long-term acoustic benefits (2).

### The OBSI Method

The sound intensity method of measuring tire–pavement noise was originally developed in the early 1980s, and began to be adapted around 2002 in California to quantify tire–pavement noise performance of different pavement types (5). Early databases of Arizona and California pavements showed encouraging results regarding the use of the OBSI method. In 2004 Caltrans took the OBSI measurement process to Europe to collect data on a wide variety of quiet pavements in several countries (6).

### Asphalt and Concrete Pavements Tire–Pavement Noise Evaluation

From analysis using the early Arizona and California database it was concluded in 2004 that, “*at least in the US, the absolute level of quiet PCC does not approach that of quiet AC*” (7). The concrete industry responded to this finding, and by 2005 had built test sections in Iowa through the National Concrete Pavement Technology Center (CP Tech Center). The CP Tech Center, with technical assistance from The Transtec Group, had evaluated as of May 2008 nearly 1,500 concrete pavement textures in the United States and Europe (8) with the objective of finding ways to build quieter concrete pavements. Results of this effort, in terms of OBSI levels on four “families” of typical concrete pavements, are shown in Figure 1.

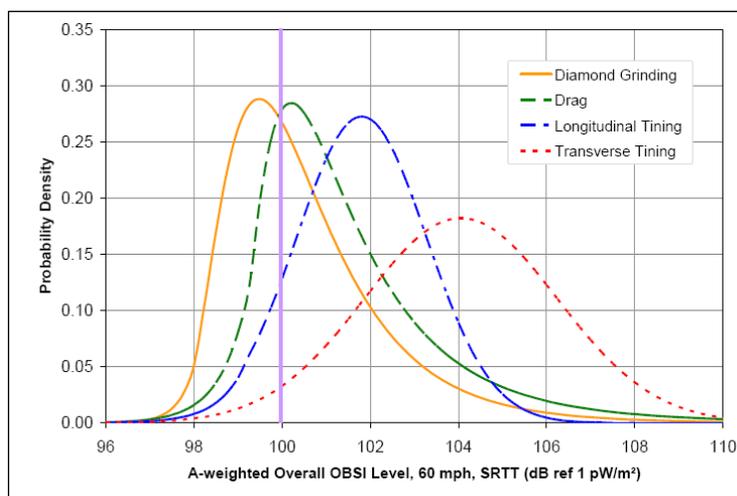


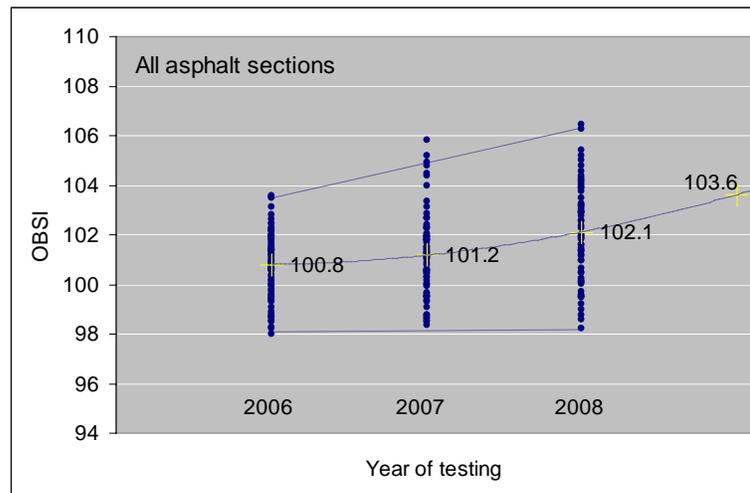
Figure 1. Distributions of OBSI noise levels for conventional concrete pavement textures as reported by the CP Tech Center (8).

## UCPRC ASPHALT PAVEMENT MONITORING

A comprehensive experiment was started by the UCPRC as part of the Caltrans QPR program in 2005 that considers performance of four main asphalt surface types, and the effect of rainfall, traffic, mix parameters, surface properties, and age. The pavement types in the experiment are open-graded asphalt concrete (OGAC), rubberized open-graded asphalt concrete (RAC-O), rubberized gap-graded asphalt concrete (RAC-G) and dense-graded asphalt concrete (DGAC). In addition, special sections placed by various Caltrans pilot and research projects were also included in the field and laboratory plan for monitoring (3,9).

A 2008 report (2) summarizes the OBSI results as well as other performance aspects besides noise, such as permeability, ride quality, distress development, and friction. The report presents the 1st and 2nd years' data of field and laboratory measurements for the main factorial set of the four types of asphalt surface with ages at the start of the study in three age categories: less than 1 year; 1 to 4 years; and 4 to 8 years. The 2008 report also includes results for the Division of Environmental long-term noise monitoring sections of various ages at the start of the study. The data obtained during the 3rd year of testing are being analyzed and indicate, as can be expected, a yearly increase in OBSI levels for the nearly 70 sections being evaluated in the asphalt pavement QPR study.

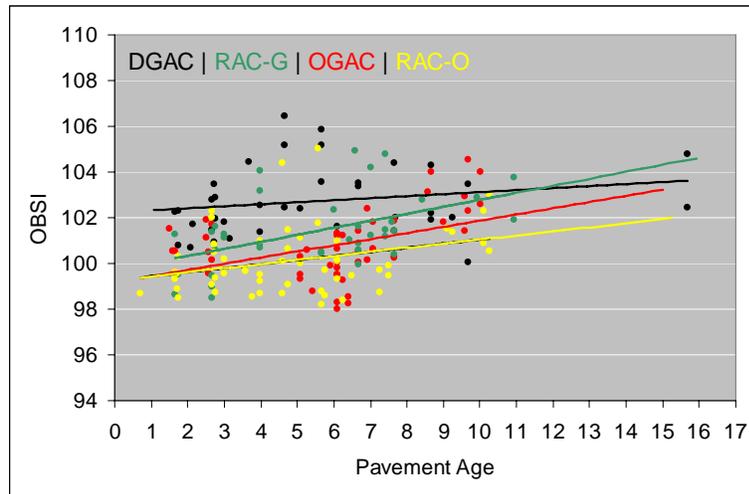
Figure 2 presents the OBSI results on all sections as measured in 2006, 2007, and 2008, along with the average for all sections. A regression line was fitted to the data to predict the 2009 OBSI average. From the chart it is possible to conclude that some sections remain quiet, but others become louder. The sections in this plot could be interpreted as a trend for the entire population of asphalt pavements in California. In 2 years these pavements have become, on average, 1.3 dB(A) louder, changing from 100.8 to 102.1 dB(A).



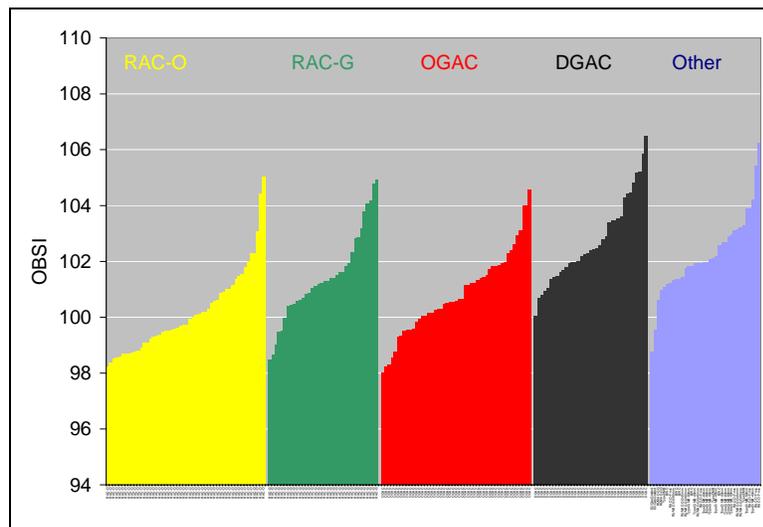
**Figure 2. Trend for the average OBSI over 3 years for the set of asphalt pavement monitoring sections in California.**

Figure 3 presents the data from Figure 1 in a disaggregated manner by pavement type and showing the age of the sections at the time of OBSI measurements. There are sections that were tested about 6 months after construction during the 1st year of the study, and there are sections that were almost 16 years old the last time they were tested. The average section age is

5.5 years. Linear trends have been fitted to the OBSI data of each pavement type. The results indicate that the quietest asphalt pavement surface mixes are the rubberized open graded, followed by conventional open graded, rubberized gap graded, and dense graded in that order. One interpretation from this plot is that no asphalt pavement that has been in place for more than 8 years is producing noise levels lower than 100 dB(A). Another representation of the data is shown in Figure 4 where all the OBSI results of each pavement surface mix type are grouped together and sorted in ascending order. An extra category, labeled as “other,” is included in Figure 4 and includes bonded wearing courses, experimental rubberized mixes, mixes with modified binders, and open-graded mixes with 19-mm (¾-in) aggregates.



**Figure 3. OBSI versus pavement age for four types of asphalt pavements.**



**Figure 4. Comparison of OBSI levels for different types of California asphalt pavements.**

## UCPRC PCC PAVEMENTS EVALUATION

A Caltrans QPR study being performed by the UCPRC since August 2008 is evaluating noise levels on concrete pavement. As in the case for the asphalt pavements, Caltrans requested an initial 2-year study, but this may be extended depending on the results. So far, 120 pavement sections and 24 concrete bridges have been tested, but the data analysis has only begun. The reason to include bridge decks is that California bridge decks very often have transversely tined surface textures, and are a source of public concern due to higher noise levels. Most pavements in California have been built with longitudinal tining. Besides longitudinal tining, other concrete pavement surface textures evaluated in this study include burlap drag, diamond grooving, and diamond grinding. Transversely tined concrete pavements are not included in this study because transverse tining of concrete pavements was discontinued by Caltrans after a study conducted in 1972-1976 (11) that specified that pavements should be given an initial texturing with burlap drag and a final texturing with a “spring steel tine device” to produce longitudinal grooves parallel to the center line. Among other reasons given for discontinuing transverse tining of concrete pavements is that “A greater tire noise is generated by transversely tined concrete pavement and there is belief among some highway engineers that wear rates are also higher” (11).

Table 1 presents the number of sections of each texture type in the study. All these sections were tested between September 2008 and early January 2009, at sites spanning 17 California counties.

**Table 1**  
**Texture Types and Number of Sections for**  
**Pavements and Bridges in The UCPRC PCC Study**

Element	Texture	Sites	Sections	Total
Pavements	Burlap drag	15	43	120
	Diamond grind	15	30	
	Longitudinal tine	8	28	
	Diamond groove	6	18	
	Longitudinal broom	1	1	
Bridges	Transverse tine	5	8	24
	Transverse broom	3	5	
	Diamond grind	2	4	
	Polyester	4	4	
	Longitudinal broom	2	2	
	Longitudinal tine	2	1	

## Methodology for Concrete Pavement Testing

In designing the UCPRC field experiment to test tire–pavement noise on concrete pavements, it was necessary to preserve the same testing protocol used on the asphalt pavements in order to have comparable results. It was also necessary to introduce additional testing to satisfy the special needs of jointed pavements. To preserve the testing method, three runs of 5 seconds at 60 mph with the SRTT tire were made on each section (440 ft sections). Optical triggering was used to ensure that the exact same stretch of road was measured in each of the three runs, and to also ensure that the same section will be tested in subsequent years. Since texturing of fresh concrete pavement is affected by concrete slab placement conditions, it is very possible that texture depth may vary within a few hundred feet. Accordingly, it was decided that three consecutive 5-second sections at 60 mi/h (96 km/h) (1,320 ft [402 m] long) should be tested. A speed correction factor was applied because keeping speed constant in live traffic for such a long stretch is often very difficult to do. The speed correction factor being used for now is  $\pm 0.3$  dB(A) per mile change in testing speed (14). The effect of joints and cracks on tire–pavement noise is being explored because the joint slap noise acts together with the effect of texture (15). To investigate the joint slap effect, the longitudinal profiles were collected at 1-in. (25-mm) intervals, which allows for joint identification, along with joint faulting (using inertial profilometer). Average joint width is also being documented. Tire–pavement noise levels are measured at 15-millisecond intervals using the OBSI equipment to try to relate noise to joint width and faulting.

## Texture Types and Variations

One observation that is worth noting is the wide range of textures encountered in the field in California. The longitudinal tining specification resulted in a variety of textures, ranging from straight lines to very “wavy” lines, with groove depth that varies between deep and shallow. The tining specification only mentions the type of instrument to apply the texture, but the results are highly variable. An analytical complication comes from the fact that pavements are “re-textured”, as in the case of diamond grinding on top of original tined or burlap dragged surfaces that have exposed aggregates due to wearing out of the concrete mortar. Some examples of textures are shown in Figure 5.

Given the fact that nominal texture types show a great deal of overlap in terms of noise levels (a noise level cannot be associated unequivocally to a texture), it is evident that more effort should be placed on engineering characterization of pavement textures that would eventually allow for prediction of OBSI levels.

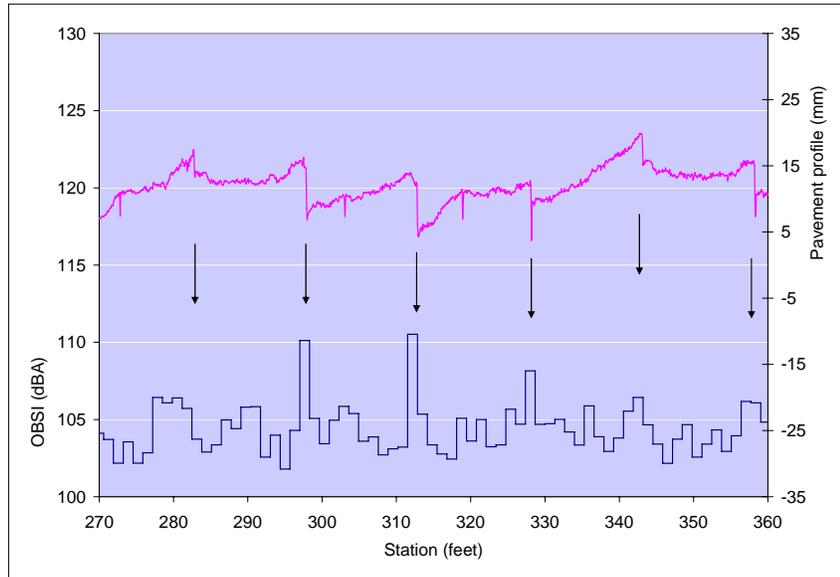


**Figure 5. Different texture types included in the study.**

### Short Interval Noise Data

The 15-millisecond noise data collected at 60 mi/h (96 km/h) means that each data point represents the noise level over a distance of approximately 16 in. (406 mm) (not even a full tire rotation). Although this type of information may not correlate well with the 5-sec typical interval used for OBSI, it at least allows identification of changes in texture types, and combined with the profile data at 1-in. (25-mm) interval, allows for an estimation of the effect of joint slapping on the overall noise. Figure 6 shows an example of 15-ms noise data (blue series) and the cor-

response to faulted joints on the pavement elevation profile (pink series). Data from other sections seem to indicate a strong effect from not only faulted joints but also from joints that are wide open. These preliminary data seem to confirm laboratory findings that joint slap creates a transient noise event which can be 4-6 dB(A) louder than tire-pavement noise produced by the pavement texture alone (15).

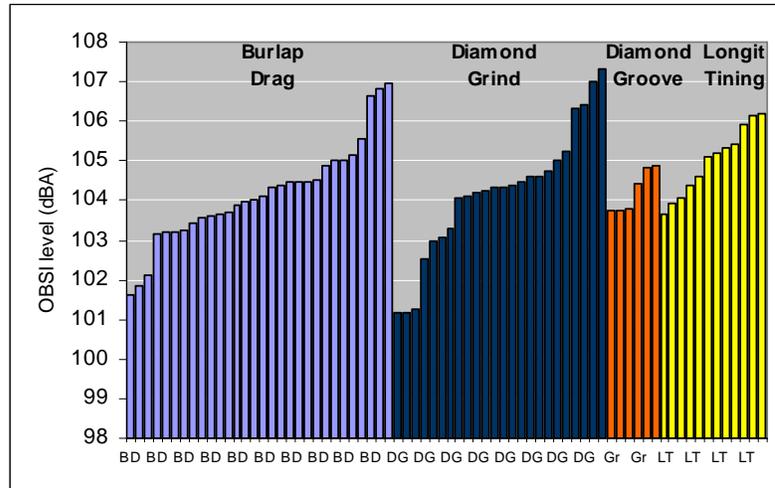


**Figure 6. Noise level spikes and correspondence to joint faulting.**

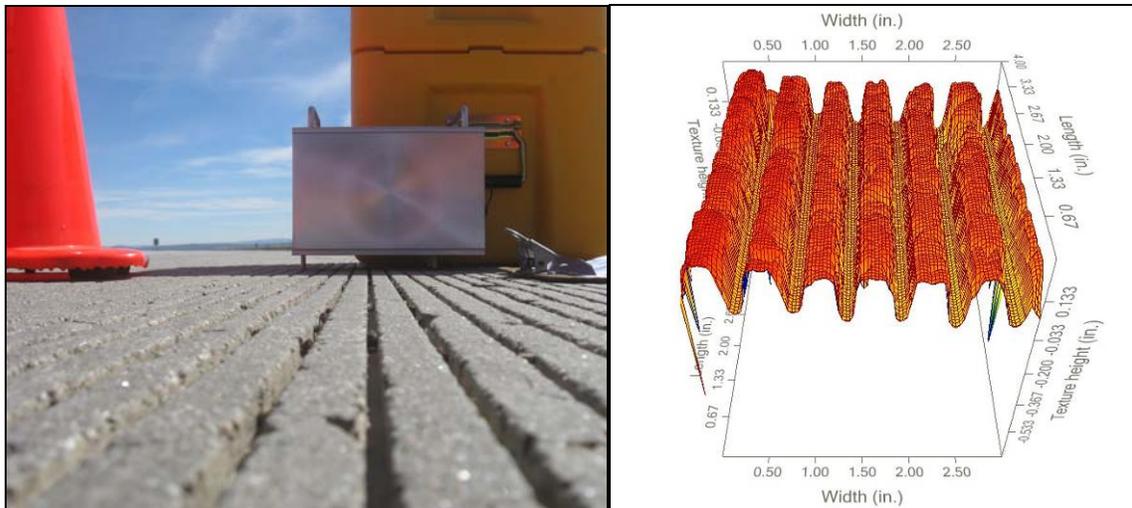
### Surface Texture Effect

So far, with only a fraction of the data analyzed, some effect from surface texture starts to appear. Figure 7 shows that diamond-ground surfaces can be the quietest, followed by the diamond grooved. No clear conclusions can be made yet regarding longitudinal tining and burlap drag. It must be noted that transverse tining is not part of the set of textures on pavements, but it is being included in the bridge decks portion of the study. The average OBSI level of the 73 pavement sections included in Figure 8 is 104.3 dB(A); the lowest is 101.2 dB(A) and the highest is 107.0 dB(A). The lowest OBSI level was measured on a diamond-ground pavement, while the highest values were measured on a burlap drag surface whose surface is worn out leaving exposed large aggregate (see Figure 5c), and on a diamond-ground surface with a base texture (original texture) of transverse tining (see Figure 5f). Although the lowest values are not as low as those reported in Figure 1, the range of OBSI levels measured in this study are comparable to the values reported in the literature.

A small study on texture will be conducted using detailed texture profiles on a limited number of sections, to complement the results obtained with the high-speed texture laser. The detailed texture data has been obtained using a device with a single laser that scans an area of 3 by 4 in. (76 by 102 mm). The device and an example of the results for a diamond-grooved surface on California State Route 58 in the Mojave desert are shown in Figure 8.



**Figure 7. Tire pavement noise for some of the concrete pavements with different textures from California QPR study.**



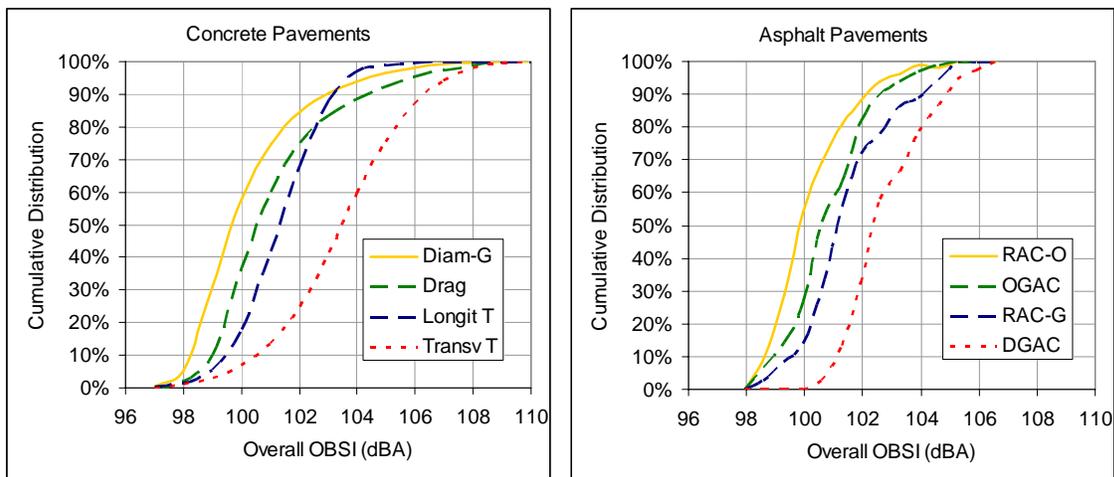
**Figure 8. Laser scanner and results of texture elevation on a diamond-grooved pavement surface.**

## COMPARISON BETWEEN ASPHALT AND EXISTING PCC NOISE DATA

The results from early OBSI measurements in Arizona and California (12) indicated that longitudinally tined concrete pavements may not be all that different in terms of tire–pavement noise levels compared to asphalt pavements over the entire service life of the pavement structure. The issue of acoustic durability of different types of pavements, surface textures, and treatments is an important objective of the California QPR Program. It basically aims at comparing the rate of tire–pavement noise change with age for concrete pavements versus those of asphalt pavements. A study in the early 1990s by the Washington Department of Transportation (13) indi-

cated that asphalt pavements start out quieter than Portland cement concrete pavements, but the noise level increases with age to the extent that after about 6 to 8 years the noise levels from asphalt pavements become greater than those of concrete pavements.

The concrete pavement noise data collected by the CP Tech Center (Figure 1) and the asphalt pavement noise data collected by the UCPRC are shown in Figure 9 in the form of cumulative distributions for the four major pavement types on each study. The quietest concrete pavements are the diamond-ground surfaces, while the quietest asphalt pavements are the rubberized open graded mixes. OBSI levels of about 100 dB(A) can be expected for the 50th percentile of the sections with each one of these two types of quiet pavements. The lower 10th percentiles are between about 98 and 101 dB(A) for all four types of both concrete and asphalt pavement, while the 90th percentiles are between about 103 and 106 dB(A) for concrete and 102 and 105 dB(A) for asphalt. The upper 90th percentiles for the quietest asphalt (RAC-O) and concrete (diamond-grind) surfaces are 103 and 102 dB(A), respectively.



**Figure 9. Cumulative distributions of OBSI levels for concrete pavements (CP Tech Center data) and asphalt pavements (UCPRC data).**

Some caution should be exercised with the information shown in Figure 9 for several reasons. First, the asphalt pavement data comes only from California sections, whereas the concrete pavement data comes from a nationwide database. Second, the OBSI levels have been collected using the draft AASHTO Standard for OBSI, which has been evolving during the time of collection. As of January 2009, there is not yet an official provisional protocol. This may cause slight differences in the way the data are collected, which has to do with acoustic data-processing, microphone arrangements, and other parameters affecting OBSI results that are still topics of research. It should also be noted that the reference test tire has also been changed and this impacts OBSI noise levels, although transformation functions have been developed so long-term comparisons can still be made.

A study conducted in May 2008 (10) compared the results of the OBSI equipment operated by four organizations. Measurements were taken at the same time and on the same pavement sections in Mesa, Arizona. It was found that all the OBSI devices produced results within 1.0 dB(A). However, the OBSI equipment used by the UCPRC to collect data from the asphalt

sections produced slightly higher OBSI measurements, while the OBSI device used by the CP Tech Center for the concrete sections produced slightly lower values compared to the average of the four devices; but all results were within 1.0 dB(A).

## CONCLUSIONS

The following conclusions can be drawn at this point from this ongoing research study:

- The OBSI levels measured so far on California concrete pavements are within the ranges reported in the literature, and the results confirm that diamond-ground surfaces are the quietest, although not as quiet as reported by other researchers.
- As indicated in reference (8), it is important that the highway community establish rational goals for tire–pavement noise. Based on the work conducted to date at the CP Tech Center and at the UCPRC, a noise level of 100.0 dB(A) measured at 60 mi/h (96 km/h) using the OBSI method appears to be a target that both asphalt and concrete pavements can achieve for some time after initial construction.
- The UCPRC study on noise monitoring of asphalt pavement shows that after 2 years, the average noise from about 70 sections has increased from 100.8 dB(A) to 102.1 dB(A). This is an increase of 1.3 dB(A) over a 2-year period. It must be noted that many RAC-O sections that are less than 8 years old are still between 98 and 100 dB(A).
- In general, none of the asphalt sections in the UCPRC study after 8 years in service offers OBSI levels lower than 100 dB(A).
- Preliminary analyses on the joint slap noise on concrete pavement seem to confirm previous finding that reported effects on the overall noise. This opens the possibility to reduce concrete pavement noise not only through surface texture enhancements, but also by improving joint construction techniques such as narrower joints or avoidance of protruding or highly depressed joint sealant.
- Further analysis on the data being collected will answer questions about durability of acoustic properties of different types of concrete pavements. For that purpose accurate information on year of construction for each pavement sections is being compiled, but it is safe to say that most of the sections are more than 10 years old.

## ACKNOWLEDGMENTS AND DISCLAIMER

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The contents of this paper reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views

or policies of the State of California or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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## **Part 5**

# **Emerging Pavement Repair and Rehabilitation Technologies**



# **I-15 Ontario Project: Technology Implementation for Accelerated Concrete Pavement Rehabilitation**

Eul-Bum Lee,<sup>1</sup> Seungwook Lim,<sup>2</sup> Jonathan C. Hartog,<sup>3</sup> and David K. Thomas<sup>4</sup>

## **ABSTRACT**

As highway agencies across the country attempt to balance rebuild existing highways while they reduce congestion and user delays and improving safety, the use of accelerated highway rehabilitation methods has become a necessity. This has been the case for the California Department of Transportation (Caltrans), which recently undertook a major concrete pavement rehabilitation project on I-15 near the city of Ontario, California. The I-15 Ontario Corridor carries about 200,000 ADT with 4-6 lanes each direction, about 6 percent of which is heavy trucks during peak hours. The size of the project is approximately \$86 million in the engineer's estimate cost. It is scheduled to start construction on February 2009 and to be completed by April 2010. The major scope of the project is the replacement of concrete pavement on two outside lanes in both directions along the 7.5-km (4.7-mi) stretch. Due to a complexity of construction access and rehabilitation process, the project was designed to implement various types of concrete pavement rehabilitation methods. Basically, the old concrete pavement will be replaced with one of: (1) normal portland cement concrete (28-day curing-time mix); (2) rapid strength concrete (12-hour curing-time mix); (3) fast-setting hydraulic cement concrete (4-hour curing-time mix); or (4) precast concrete panel. Construction scheduling and analysis program CA4PRS (Construction Analysis for Pavement Rehabilitation Strategies) was used to demonstrate that the combination of the rehabilitation methods was the most cost-effective strategy on project to shorten construction duration, to minimize lane closure impact, and to achieve longer-life pavement design. To take the advantage of unique experimental technologies being adopted on the I-15 Ontario Project, Caltrans plans to conduct field monitoring studies with the FHWA and UC Berkeley researchers to compare rehabilitation process and progress, and work-zone traffic impact between the design and material types.

## **HIGHWAY REHABILITATION**

Most of the major highways in the United States were built during the 1960s and 1970s and have exceeded their design lives, which has motivated the transportation agencies responsible for them to shift focus from construction of new highways to the rehabilitation of existing facilities. Because highway rehabilitation projects often cause congestion, safety concerns, and limited access for road users, agencies face a challenge in finding economical ways to renew

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deteriorating roadways in metropolitan areas. It is estimated that the annual costs to drivers, businesses, and transportation agencies incurred by highway construction traffic delays total \$43 billion, \$21 billion of which is in extra fuel consumption (Edwards 1998). The California Trucking Association estimates that the impact of early opening of freeways saves their “commercial operators more than \$250 per truck trip or \$500,000 per day” in trucking costs (Carr 1994).

Severe traffic disruption at construction work zones on urban highway networks is the primary source of conflict between transportation agencies and abutting communities. The frequent conflicts among these parties have brought to the fore the need to expedite construction to ameliorate the impact on motorists and affected businesses. To expedite construction, the Federal Highway Administration (FHWA) and the Transportation Research Board have recommended experimenting with new approaches that are believed to have the potential to reduce construction time (Herbsman and Glagola 1998). However, an innovative accelerated construction approach to urban highway pavement projects, typically requiring extended closures, might not be accepted by the public without a substantial public outreach effort.

In 1998, the California Department of Transportation (Caltrans) initiated the Long-Life Pavement Rehabilitation Strategies (LLPRS) program to rebuild aging and deteriorating pavements in the urban network. The LLPRS program was initiated because most State highways have exceeded their 20-year design lives and will require rehabilitation to maintain safety and function. The obstacles of highway rehabilitation, including significant costs and work zone delays, demonstrate the need for effective planning of highway closures to ensure that the most appropriate alternative is selected. Caltrans has successfully demonstrated the advantage of accelerated pavement rehabilitation on its LLPRS projects in urban highway networks, including I-10 Pomona concrete rehabilitation project (2000), I-710 Long Beach AC rehabilitation project (2002), and I-15 Devore concrete rehabilitation project (2004).

## **I-15 ONTARIO PROJECT**

### **Project Overview**

The I-15 Ontario Project is a 7.5-km (4.7-mi) pavement rehabilitation project on the Interstate highway I-15 near the city of Ontario, about 40 mi (64 km) east of Los Angeles, in Riverside County. The project section with a total of about 40 lane-mi (64 lane-km) of rehabilitation begins at the I-15/Route-60 junction (KP 82.8, PM 51.4) and continues across the San Bernardino/Riverside County line to Seventh Street (KP 6.1, PM 3.8), just north of the I-10 / I-15 interchange. Figure 1 shows the project location. The I-15 Ontario corridor carries about 200,000 average daily traffic on its 4 to 6 lanes each direction with about 6 percent heavy trucks during peak hours.

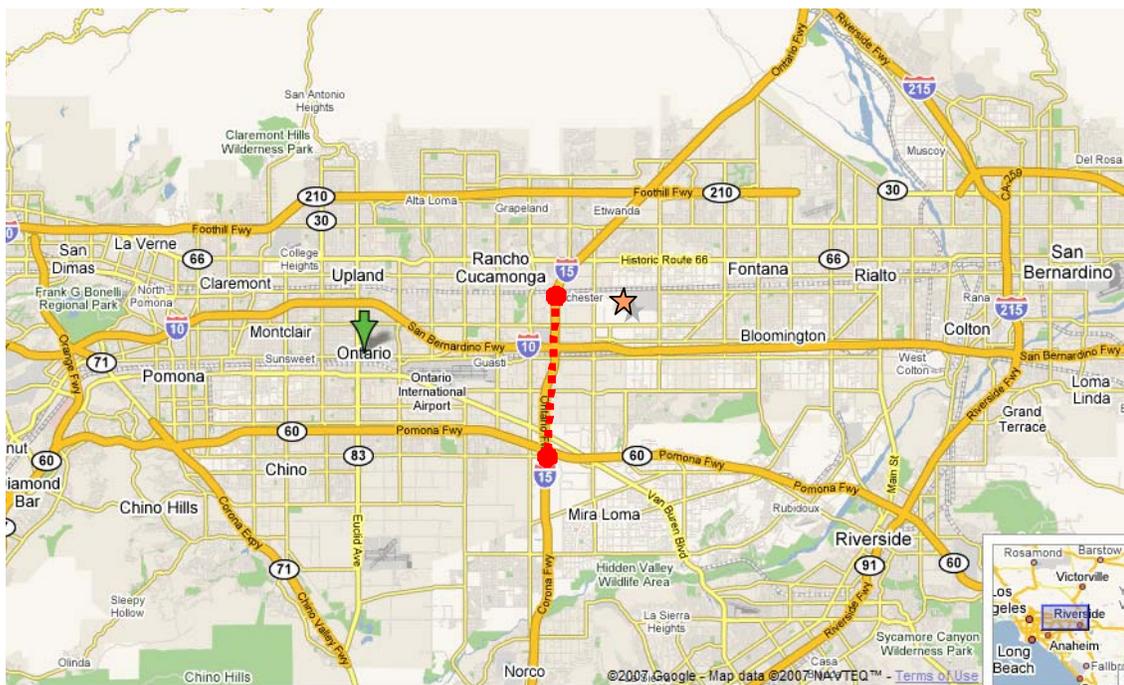
The project proposes to rehabilitate concrete pavement sections of the two outside lanes in both directions along with interchange ramps, freeway-to-freeway connectors, and asphalt concrete (AC) shoulders. Other major features of the project include widening the inside shoulder, widening the median roadway and structure crossings to accommodate traffic detours during construction, and pavement grinding of all lanes. To replace the two outside lanes during rehabilitation, the mainline traffic needs to be shifted. A temporary two-lane detour will be created in the median by paving the median and widening several bridge structures (Figure 2). This widened median temporary detour with lane shifts during construction will be used for the majority

of the project area except in some specific locations (more details are described in the following sections).

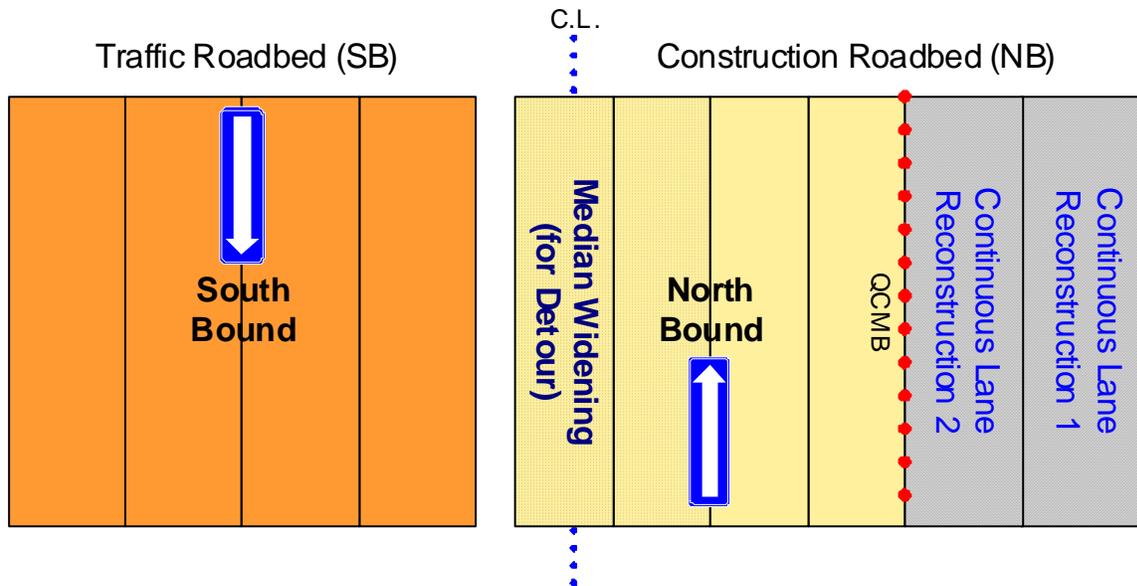
The following four pavement rehabilitation types using different concrete materials are designed to replace the existing concrete pavement (9-in. [229-mm] slab on top of a 4-in. [102-mm] cement-treated base and aggregate base) on the I-15 Ontario project. Details of the respective rehabilitation methods are discussed in the following sections.

- Portland cement concrete (PCC) pavement: about 25 lane-km (16 lane-mi)
- Rapid-strength concrete (RSC) pavement: about 12 lane-km (7 lane-mi)
- Precast pavement: about 3 lane-km (2 lane-mi)
- Fast-setting hydraulic cement concrete (FSHCC) pavement: about 5 lane-km (3 lane-mi)

The selection of the pavement type and material mainly depends on the rehabilitation location, taking into account lane closures for traffic control and construction access. Caltrans engineers believe that traditional rehabilitation practice in California would repair the pavement in small sections at a time using standard FSHCC with partial-lane closures during nighttime. This nighttime rehabilitation method is characterized by low production per closure, more expensive material cost, and a less durable pavement (15–20 years in pavement design life).



**Figure 1. I-15 Ontario rehabilitation project location and boundary.**



**Figure 2. Plan view of the construction roadbed (NB) with two lanes shifted to the widened median for rehabilitation of the outer two lanes.**

### Rehabilitation Construction Scheme

It is estimated that the traveling public will be impacted by construction for 260 days total (not including the duration of the median detour widening behind K-rail without lane closures). This number only includes days where temporary reductions in the number of travel lanes are required and weekend full-connector closures are in place. The total length of the construction contract is estimated to be about 410 working days (about 2 years of construction). The overall construction contract will impact all hours of the day and week. Normal construction operations will occur from 7:00 a.m. to 3:30 p.m. protected by temporary railing. Traffic impact during this time will be minimal with traffic shift to the widened median, and will occur in the form of reduced lane widths and/or no shoulders. Rehabilitation work during nighttime lane closures from 9 p.m. to 4:00 a.m. Monday through Thursday includes pavement grinding, random slab replacement, shoulder rehabilitation, precast panel replacement, and bridge approach slab replacements. Weekend closures from 10:00 p.m. Friday to 4:00 a.m. Monday will be used for rehabilitation of freeway-to-freeway connectors, precast panel replacement, and mainline weaving areas.

Extensive public outreach will be arranged to keep the public informed of the latest closures and construction progress in order to minimize inconvenience to the public. Some of these outreach efforts include brochures and mailers, a project Web page linked from the district's home page, media releases, public meetings, and changeable message signs deployed within and around the construction limits. Caltrans will also be working closely with the affected cities of Rancho Cucamonga and Ontario and with the counties of San Bernardino and Riverside to minimize the impacts to local arterials during construction.

## **VARIOUS REHABILITATION METHODS**

### **PCC Rehabilitation**

Typically, pavement on the two outside truck lanes will be replaced with a new 12-in. (305-mm) PCC slab and new 6-in. (152-mm) AC base, as long as the roadway has enough space to shift main lanes traffic to the widened median. The rehabilitation will take place behind K-rails during daytime with two lanes closed. However, the roadway during PCC rehabilitation will still have a minimum four lanes open as the median will be widened with concrete before main-lane rehabilitation for detours. As there is no lane closure impact with shifted traffic using the median detour, the contractor is allowed to use normal PCC (type II), which requires days or weeks for curing before opening to traffic. However, the total estimated cost of the median widening is about \$8 million (\$1 million for pavement widening and \$7 million for bridge widening), which is still cheaper than other rehabilitation types (like FSHCC rehabilitation during nighttime) considering savings in agency cost and traffic delay cost (as discussed later). The widened median pavement will be reserved for a HOV lane in the future (probably about 10 years after the rehabilitation project is done).

Demolition of the existing concrete pavement will be done first, and the 6-in. (152-mm) AC base will be placed before paving the new 12-in. (305-mm) PCC slab. The PCC rehabilitation will be applied to the areas between the interchanges without major in and out traffic. According to the Caltrans historical bid database, the average unit price of PCC rehabilitation including demolition in California is about \$200/m<sup>3</sup> (Caltrans 2007).

### **Precast Rehabilitation**

To reduce the length of construction time impacting the traffic and traffic delays as a result of these restrictive areas, and to provide a high-quality, long-lasting pavement, it is proposed to apply precast slab replacement method in these areas. The precast method will allow the contractor to perform the continuous lane replacement quickly during nighttime or weekend closures while maintaining construction quality and minimizing traffic delays. A stretch of concrete pavement about 3 lane-km (2 lane-mi) long was selected to receive precast rehabilitation. Typically, the old 9-in. (229-mm) PCC slab will be removed and replaced with the same thickness of precast on top of a newly placed thin bedding with fine sands for better contact with the precast slab.

There are two types of precast rehabilitation on the project: continuous lane replacement and random slab replacement, which will be selected depending on the conditions of old PCC pavement. Majority of precast rehabilitation areas will be replaced with precast panels for the whole lane, as so-called continuous precast lane replacement. This will happen during 55-hour weekends with full closures of the area on auxiliary lanes (but through traffic on main lanes are open, as the same as the rapid-strength RSC rehabilitation). Estimated unit price (measured by area) of precast replacement in California is about \$300/m<sup>2</sup>, which is equivalent to a volume price of \$1,400/m<sup>3</sup>.

The precast panels are designed and manufactured under ideal conditions for long-term durability. Coupled with the dowel bar reinforcing at each transverse joint, the slabs are fully supported with a precisely graded subgrade and bedding grout that could minimize joint differentials and provide a longer lasting, smoother profile. As such, the precast rehabilitation is

expected to substantially reduce the need for roadway maintenance and to generate a reliable stretch of roadway.

In a separate previous study, FHWA/Caltrans and the University of California Pavement Research center conducted accelerated loading tests with a heavy vehicle simulator (HVS) on a test strip (see Figure 3 and Figure 4) near the I-15/ Route-210 interchange to evaluate the performance and durability of precast panels (Kohler et al. 2008). Test results appeared to support an anticipated life span of over 30 years for the precast panel. Recommended performance goals in smoothness regarding the implementation of the precast panel replacement technology is less than 48 in/mi (768 mm/km) as measured by International Roughness Index (IRI). More specifically, the precast concrete tested on the study, the Super-Slab system, seems able to withstand 24 hours of highway traffic (at least 88,000 equivalent single-axle loads [ESALs]) in the ungrouted condition. From the HVS tests, there is no evidence to believe the Super-Slab system would fail before 140 million ESALs (Techtransfer 2008).

Although the initial cost for precast replacement is higher than other traditional rehabilitation methods, cost savings are realized due to the extended life of precast panels and lower life-cycle costs than those for traditional rehabilitation strategies. Furthermore, the reduced construction time translates to a reduction in the amount of time in which the traveling public is impacted by construction activities. This results in a lower road-user cost from construction delays.



**Figure 3. The construction the HVS testing-site for the precast pavement rehabilitation for the I-15 Ontario project.**



**Figure 4. Accelerated pavement testing using HVS on the precast panels for the I-15 Ontario project.**

### **Rapid-Strength-Concrete Rehabilitation**

Based on Caltrans' experience and practice on previous accelerated urban concrete pavement rehabilitation projects such as the I-15 Devore rehabilitation project, a special concrete mix using Type III cement, called rapid-strength concrete or RSC in California, will be used for some specific areas on the I-15 Ontario project. An RSC mix makes it possible to open the project to traffic 12 hours after its placement while still allowing for slipform paving (Long-life 2007). Specific areas such as auxiliary (accelerating or decelerating) lanes to/from ramps and freeway-to-freeway connectors will receive pavement rehabilitation using RSC, which earns 400 lbf/in<sup>2</sup> (2.76 MPa) flexural strength within 12 hours after mixing and placement. These RSC rehabilitation areas are closed completely during the weekend, while through traffic on the main lanes does not have lane closures. Pavements on the connectors are candidates for RSC rehabilitation. Similar to the PCC rehabilitation cross-section change, the 12-in. (305-mm) RSC slab will be cast in place on top of the new 6-in. (152-mm) AC base. Typical unit cost of RSC rehabilitation in California is about \$600/m<sup>3</sup>.

### **Fast-Setting Hydraulic Cement Concrete Rehabilitation**

Damaged PCC pavements on inside lanes will be removed and replaced (cast in place) with new concrete slab using FSHCC, which has enough flexural strength to open to traffic within 4 hours after mixing. This random slab replacement will take place during nighttime for mostly about 7 hours with partial-lane closure for the areas where other rehabilitation types (i.e., PCC rehabilitation during weekdays, precast rehabilitation, or RSC rehabilitation during weekends) are not applied. The southern part of the project area does not have enough space on the median to shift traffic for detour.

The area's lane configuration, from a geometric point of view, is not good enough to apply the weekend closure strategy to use RSC rehabilitation. PCC pavement on this southern area will be rebuilt with FSHCC pavement even for the outside truck lanes as continuous lane recon-

struction in small piece-by-piece with nighttime closures. The unit price of FSHCC rehabilitation in California is currently about \$900/m<sup>3</sup>, due to high material cost to use special calcium aluminates cement and unique admixtures.

As Caltrans' experience shows on its long-life pavement rehabilitation demonstration projects, such as on the I-10 Pomona project, FSHCC pavement rehabilitation during nighttime has a production downside compared to PCC or RSC pavement rehabilitation. Lower productivity resulted mainly from (1) the longer discharging cycle of ready-mixer trucks, which slowed concrete delivery and (2) using a roller-screed paving machine rather than a slipform paver due to high concrete slump (Lee et al. 2002).

## **INNOVATIVE FEATURES**

As previously mentioned, the I-15 Ontario project showcases a variety of pavement rehabilitation designs, materials, and construction approaches. Among the latter are continuous lane reconstruction and random slab replacement of the respective material types. The project also has a variety of lane closure strategies, including traditional nighttime and daytime closures, and more innovative means such as extended weekend closures. Construction scheduling and analysis software CA4PRS (Construction Analysis for Pavement Rehabilitation Strategies) was utilized to select the most effective lane-closure tactics under given traffic and construction conditions. It incorporates many variables, such as pavement section, construction access, and lane closures, and production rates for estimating the construction schedule, and work zone traffic delay and agency cost for a particular rehabilitation strategy. Two steps described in this section were taken in the design of this project to reduce the congestion impact to the traveling public.

### **Work-Zone Simulation for TMP**

The work-zone safety and mobility rule requires State and local governments to comply with the rule on Federal-aid projects no later than October 12, 2007. One of the three main components of the rule is to develop procedures to assess and manage work-zone safety and the mobility impacts of individual projects. Analysis may necessitate the use of analytical tools, depending on the degree of analysis required. Some tools, such as QuickZone and CA4PRS, were designed for work-zone-related analysis. Other traffic analysis tools that were not designed specifically for work zones may also be useful for analyzing work zone situations (Work-zone 2008).

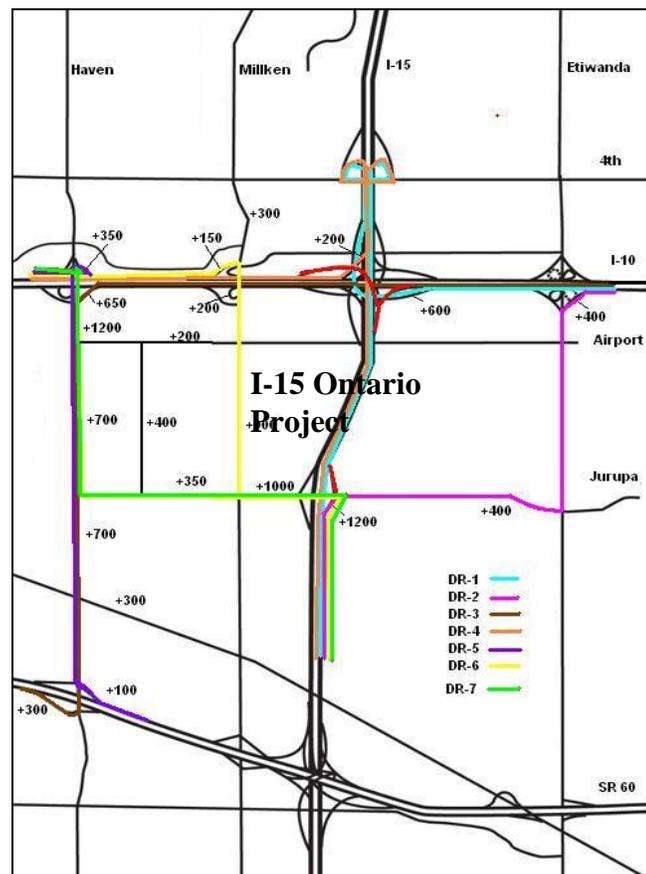
To anticipate traffic delays and impacts, a traffic model of the local arterial network was created for the I-15 Ontario project by the project team using mesoscopic traffic network modeling, so-called Dynameq software. Mesoscopic traffic simulation software enables planners to evaluate congested network scenarios with dynamic equilibrium benchmarks, a time-varying version of the same well-understood equilibrium assignments that have provided consistency for comparison in static analysis for years.

Dynameq's equilibrium traffic assignment results represent user optimal network conditions that are immediately useful as an upper-bound on network performance. In summary, Dynameq provides a more simplified yet realistic traffic model that can be calibrated with fewer parameters. It performs simulations more quickly than microscopic models, allowing more time for

analyzing multiple scenarios. It means that a Dynameq model is more cost effective to develop and run for a model of this scale.

And by using Dynameq, more alternatives can be analyzed, increasing the likelihood of choosing a staging alternative that has the least impact on the traveling public. For this project, Dynameq was used to analyze the impacts of the most significant freeway-to-freeway connector closures and adjust the project staging to minimize the impact to the traveling public. According to the network simulation for the work zone analysis, specifically for the I-15 NB to I-10 EB/WB connector closures, the major diversion routes for the trips that used to take the connectors without the construction closure are through local arterials such as Milliken, Haven, and Etiwanda. This detoured traffic combined with the detoured traffic due to the connector closures results in the traffic increase on the network shown in Figure 5.

The knowledge gained from the traffic network simulations is used to address concerns that our partners, including the residents in surrounding cities, counties, and traveling public, may have about the impacts of the construction, and to demonstrate to them the level of effort used to minimize those impacts.



**Figure 5. The mesoscopic traffic network simulation using Dynameq indicates detour volume to local arterials during the connectors (I-15 NB to I-10EB/WB) closure on weekend.**

## CA4PRS Software

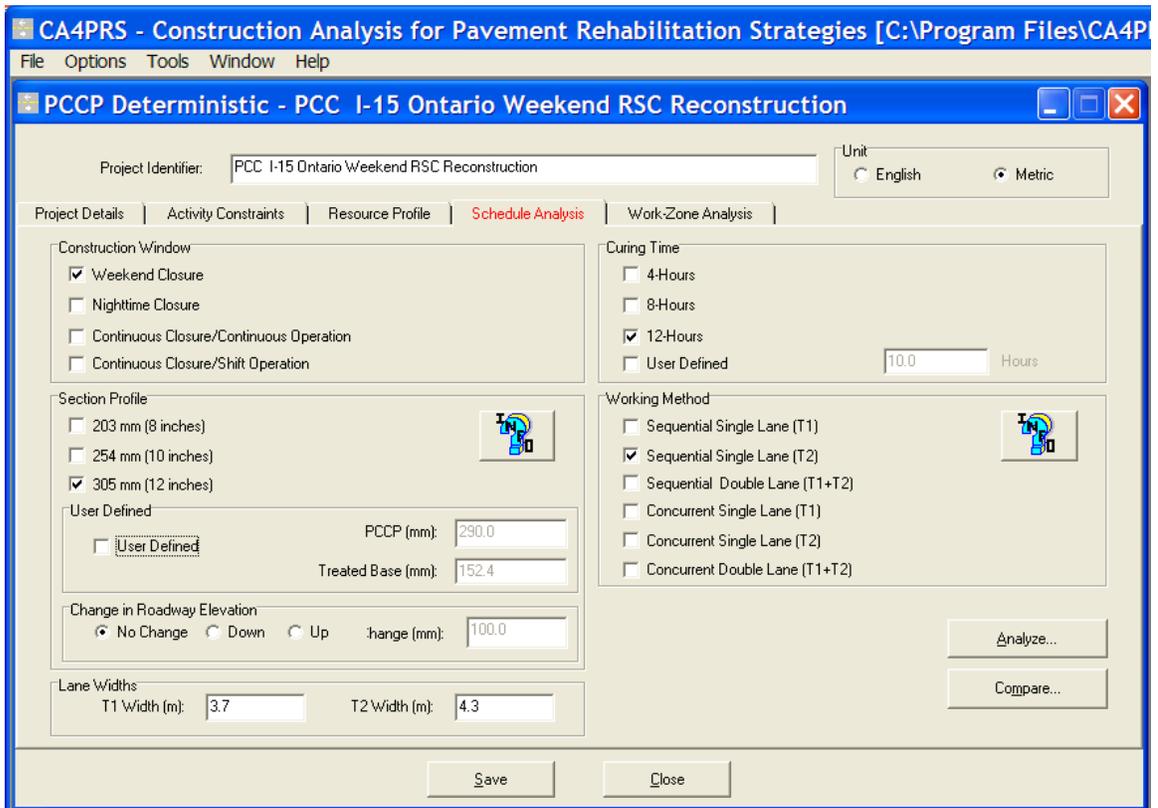
One innovation in the effort to reduce highway construction time and its impact on traffic is the CA4PRS software. CA4PRS is a decision-support tool for transportation agencies that helps in selection of the most effective and economical strategies for highway maintenance and rehabilitation projects (Lee and Ibbs 2005). Funded through an FHWA pooled-fund study supported by a multistate consortium including California, Minnesota, Texas, and Washington, CA4PRS was developed by the Institute of Transportation Studies of the University of California, Berkeley, with technical collaboration from the FHWA Turner-Fairbank Highway Research Center.

The program incorporates three interactive analytical modules: a Schedule module that calculates project duration, a Traffic module that quantifies the impact of work zone lane closures and a Cost module that compares agency cost and traffic handling cost between alternatives.

Demonstrations have shown that CA4PRS is user-friendly, easy to learn, and valuable in any project phase. Users can evaluate various “what if” scenarios for combinations of alternative rehabilitation strategies, including pavement cross sections and material types, construction windows and lane closure tactics, and contractor logistics and constraints. CA4PRS has helped agencies, contractors, and consultants save engineering time, improve estimate accuracy, and streamline team work in preparing project design and traffic management plans. CA4PRS results can also be integrated with traffic simulation models to quantify the impact of work zone lane closures on the entire highway network, including local arterials and neighboring freeways.

Since 1999, the capabilities of CA4PRS have been confirmed on several major highway rehabilitation projects in several States including California, Washington, and Minnesota. The software was validated on the I-10 Pomona project in California, in which 2.8 lane-km (1.7 lane-mi) of concrete pavement were replaced in one 55-hour weekend closure. The software was also used to develop a construction staging plan for the I-710 Long Beach Project, in which 26 lane-km (16.1 lane-mi) of AC were reconstructed in eight 55-hour weekend closures.

Recently, CA4PRS was used with traffic simulation models to select the most economical rehabilitation scenario for the I-15 Devore project. The 4.5-km (0.6-mi) concrete pavement reconstruction on the I-15 Devore project, which would have taken 10 months using traditional nighttime closures, was completed over two 9-day periods using one-roadbed, continuous closures with counter-flow traffic and around-the-clock construction (Lee and Thomas 2007). Implementing continuous closures rather than repeated nighttime closures in this project resulted in significant savings of \$6 million in agency costs and \$2 million in road user costs. Other sponsoring State transportation departments have also used CA4PRS for analyses of corridor rehabilitations, including the reconstruction of I-5 through Seattle, Washington, and the rehabilitation of I-394 and I-494 in St. Paul, Minnesota.



**Figure 6. Input window of CA4PRS software used in the preconstruction analysis of the I-15 Ontario project to compare schedule, traffic, and cost between the rehabilitation alternatives.**

### CA4PRS Analysis Summary

The CA4PRS schedule analysis estimated the approximate duration of construction operations in terms of the number of weeks or weekends required for the following four different closure types on the I-15 Ontario project (see Figure 6 for the input window):

- About 25 weeks of construction operations during weekdays are required for median detour paving (PCC and AC) without major traffic lane closures.
- About 35 weeks of construction operations are needed during weekdays for main-lane pavement rehabilitation using normal PCC without major traffic lane closures, as the traffic is shifted to the median detours.
- About 32 weekends of operations during 55-hour weekend closures are need for pavement rehabilitation on main lanes, ramps, and connectors with ramps and connectors closed.
- About 8 weeks of operations are needed during weekday nighttime (7-hour) closures for main-lane pavement (FSHCC) rehabilitation for the segment south of Mission Boulevard.

A CPM (critical path method) schedule was developed as a milestone for the 55-hour weekend closures based on a typical rehabilitation process and production estimates for the major opera-

tions (demolition, AC-base paving, and PCC paving). The CPM schedule analysis indicates that the optimal distance of pavement rehabilitation during a typical 55-hour weekend closure is about 1 kilometer (1 lane-km) (0.6 lane-mi) with sequential operations due to one-lane construction access limitation. A total of 38 hours might be assigned for the main operations with 17 hours of non-operation during a 55-hour weekend closure.

A summary of the components evaluated in the integrated construction/traffic/cost analysis using the CA4PRS software is provided in Table 1 by construction scenario. The analysis components listed include schedule (closure duration), traffic (road user costs), maximum traffic delay per closure, and agency costs. Road user costs were discounted (divided by 3 as “soft money”) before being added to agency costs (as “hard money”) to derive the total cost for the construction scenario. Also provided is the cost ratio for each scenario as compared to the Original scheme.

The comparison shows that the Original (widening detour) scheme (Scenario 1) would have the lowest total cost of \$79 million, followed closely by the CSOL (crack and seat [PCC] and overlay [AC]) 55-hour Weekend (Scenario 5) with a total cost of \$83 million (5 percent higher). The analysis findings indicate that the PCC (“Rapid Rehab”) 55-hour Weekend (Scenario 2) would have the highest total cost of \$123 million, 56 percent higher than the Original scheme. The analysis concludes that the Traditional nighttime closure approach (Scenario 4) is the least economical (68 percent higher) alternative with the estimated construction schedule of about 5 to 6 years. This relative comparison in total cost among the scenarios justified the preliminary decision of D8 management in selecting the Original scheme as the most economical scenario.

**Table 1**  
**Preconstruction Analysis for Schedule–Traffic–Cost Comparison Using CA4PRS**  
**for the I-15 Ontario Project**

Scenario	Closure Duration	Traffic		Cost (\$ Millions)		Cost Ratio, %
		Road User Cost, \$M*	Delay, min.	Agency Cost*	Total	
1–Original (widening detour)	35 weekends	3	16	78	79	100
2–PCC 55-hour Weekend	35 weekends	119	363	83	123	156
3–Progressive Continuous	8 weeks	123	363	77	118	149
4–Traditional Nighttime	1,220 nights	133	22	88	133	168
5–CSOL, 55-hour Weekend	20 weekends	69	363	60	83	105

\* Road user costs are discounted (divided by 3 as “soft money”) before being added to agency costs (as “hard money”).

\*\* Agency cost estimate using CA4PRS was adjusted in the final PS&E package due to the escalation of pavement material items.

CSOL = Crack and seat (PCC) and overlay (AC)

## **CONSTRUCTABILITY AND FOLLOWUP STUDY**

### **Constructability Issues**

A number of constructability issues were addressed to compare some design and construction alternatives from the production and scheduling perspective. It was concluded that (1) RSC is more productive than FSHCC in the concrete mix design, (2) AC is more productive than lean concrete in the base type, and (3) the nonimpact demolition (slab lift) method is preferable to impact demolition (slab rubblization) due to noise concerns inherent in the slab demolition method.

Guidelines for the contractor contingency plan were outlined to deal with potential risk management on this Rapid Rehab II project with emphasis on applicability and practicality of the plan. The contingency plan addresses contractor's logistics in case of emergency, including (1) unstable subgrade replacement with aggregate base or AC, (2) use of two concrete mixes to offset a schedule delay, and (3) standby paving materials such as AC cold mix and preparation of secondary batch plants.

Three levels of time value provisions are recommended to be specified in the contract to ensure the completion of a project on time: (1) incentives/disincentives requirements for the number of 55-hour weekend closures, (2) late-opening penalty at the end of 55-hour weekend closure, and (3) cost-plus-time (A+B) contracting for the entire project. Two types of incentives / disincentives provisions are recommended to encourage and ensure that the contractor completes the 55-hour weekend closure on time or ahead of schedule: the primary incentives provision to minimize the total number of 55-hour weekend closures required; and the secondary late opening penalty to avoid considerable traffic congestion from a delay in opening closed ramps and connectors on the following Monday morning.

Some state-of-practice innovations and technologies were recommended to be adopted, such as automated work-zone information systems and a project Web site for more proactive public outreach and participation. These have been substantially beneficial to road users on previous Caltrans LLPRS projects. The implementation of these innovations and technology on the I-15 Ontario Rapid Rehab II project will help achieve the goals of (1) expediting the construction schedule, (2) minimizing adverse traffic impacts on the highway network, (3) and encouraging the public to support and participate in the public outreach program.

### **Followup Monitoring Study**

In a sense, the I-15 Ontario project is a showcase of concrete pavement rehabilitation with various types of pavement design and construction practice, based on Caltrans successful experience from the LLPRS projects. Caltrans tries to take advantage of unique experimental situations with variety of pavement rehabilitation on the I-15 Ontario project. It is planned to collect schedule and production data during construction, which is scheduled to start construction in spring 2009, and to compare the cost of construction in conjunction with life-cycle cost analysis between the different rehabilitation types.

Field evaluations will be conducted for the pavement conditions before and after construction using inertial profiler for roughness and FWD for stiffness, which would quantify the improvement of rehabilitation for each strategy. Pavement condition monitoring will continue for

a certain period of time (e.g., 5–10 years) after the construction to identify the performance difference between the material types (PCC, precast, RSC, and FSHCC) as well as the rehabilitation strategies (continuous lane reconstruction versus random slab replacement).

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# Precast Concrete Pavement for Intermittent Concrete Pavement Repair Applications

Shiraz Tayabji,<sup>1</sup> Neeraj Buch,<sup>2</sup> and Erwin Kohler<sup>3</sup>

## ABSTRACT

Precast pavement technology is a recently improved construction method that can be used to meet the need for rapid pavement repair and construction. Precast pavement systems are fabricated or assembled off-site, transported to the project site, and installed on a prepared foundation (existing pavement or re-graded foundation). The system components require minimal field curing time to achieve strength before opening to traffic. These systems are primarily used for rapid repair, rehabilitation, and reconstruction of asphalt and portland cement concrete (PCC) pavements in high-volume-traffic roadways. The precast technology can be used for intermittent repairs or full-scale, continuous rehabilitation. In *intermittent repair* of PCC pavement, isolated full-depth repairs at joints and cracks or full-panel replacements are conducted using precast concrete panels. The repairs are typically full-lane width. The process is similar for full-depth repairs and full-panel replacement. In *continuous applications*, full-scale, project-level rehabilitation (resurfacing) or reconstruction of asphalt and PCC pavements is performed using precast concrete panels.

Recognizing the need for effective, rapid rehabilitation methods, the Federal Highway Administration, through its Concrete Pavement Technology Program, and several U.S. and Canadian highway agencies have initiated programs to investigate the feasibility of using precast concrete for pavement repair and rehabilitation. Parallel to agencies' efforts, several organizations in the United States also initiated independent development activities to refine precast concrete pavement technologies. These technologies have certain proprietary features and require licensing for product use. The Strategic Highway Research Program 2, as part of its rapid highway renewal focus area, has sponsored a study (begun in early 2008) to advance modular/precast pavement technologies to enable cost-effective rapid repair and rehabilitation of pavements in high-volume traffic areas.

This paper provides a summary of current initiatives related to precast pavement technology for intermittent repair of concrete pavements and provides a framework for advancing the technology in future years.

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## INTRODUCTION

Pavement rehabilitation and reconstruction are major activities for all U.S. highway agencies, and have significant impact on agency resources and traffic disruptions because of extensive and extended lane closures. The traffic volumes on the primary highway system, especially in urban areas, have seen tremendous increases over the last 20 years, leading in many instances to an earlier-than-expected need to rehabilitate and reconstruct highway pavements. Pavement rehabilitation in urban areas is resulting in serious challenges for highway agencies because of construction-related traffic congestion and safety issues. Many agencies also continue to wrestle with the age-old problem: longer delays now and longer service life versus shorter delays now and shorter service life. In recent years, many agencies have started investigating alternative strategies for pavement rehabilitation and reconstruction that allow for faster and durable rehabilitation and reconstruction of pavements.

A promising alternative strategy is the effective use of precast concrete pavement technologies that provide for accelerated repair and rehabilitation of pavements and also result in durable, longer lasting pavements. Accelerated construction techniques can significantly minimize the impact on the driving public, as lane closures and traffic congestion are kept to a minimum. The safety of both highway users and construction workers is improved by reducing the frequency and duration of work zones.

Precast concrete pavement technologies have been looked into sporadically over the last 20 plus years. In the early years, the technology was utilized as a matter of technical curiosity, that is, to investigate whether precast concrete pavement technology was technically feasible. No serious attempts were made then to fully develop the technology as a cost-effective strategy and to implement the technology on a production basis. Now, as more mileage on the primary highway system and urban roadways are reaching maturity and the need for timely pavement repair and rehabilitation becomes acute and urgent, highway agencies are looking at innovative technologies, including precast concrete pavement technologies, that will result in shorter lane closures and long-life pavements that are economical over the life cycle and do not require major interventions for repair or rehabilitation during their service life. Over the last 10 years, significant developments have resulted in precast concrete pavement technologies, and the use of these technologies is becoming technically feasible and economically justifiable.

## BACKGROUND

Developments related to prefabricated pavement technology have focused primarily on the use of precast concrete panels. This section presents a brief overview of the historical development of precast concrete pavement technologies for intermittent repair of jointed concrete pavements. In *intermittent repair* of PCC pavement, isolated full-depth repairs at joints and cracks or full-panel replacements are accomplished using precast concrete panels. The repairs are typically full-lane width. Key features of this application are precast panel seating and load transfer at joints. Precast concrete repairs are an alternative to conventional cast-in-place, full-depth concrete repairs, especially in situations where high traffic volumes and consideration of the delay costs to users due to lane closures favor rehabilitation solutions that may expedite opening to traffic. Precast panels also offer the advantage of being “factory made” in a more controlled environment than cast-in-place repairs, and thus may potentially be more durable and less susceptible to construction and material variability.

## **Pre-1995 State of Practice**

One of the earliest reported uses of precast concrete pavement technology in the United States was during 1960 in South Dakota where a precast pavement was constructed over a granular bedding layer. Since then, several minor efforts were made to investigate the use of precast pavements, primarily as rapid repair alternatives. During the 1960s in Michigan (Simonsen 1972) and Virginia (Tyson 1976), jointed reinforced concrete pavements were constructed with panel lengths of up to 30.48 m (100 ft) and 18.75 m (61.5 ft), respectively. A principal mode of failure for these pavements was transverse joint “blowups” due to large seasonal and daily joint movements, loss of joint sealing material, and intruded incompressible fines. In these cases, the pavement was immediately closed to traffic and required rapid emergency repair to restore it to service. Alternative full-depth repair methods included asphalt patches, very-high-early-strength concrete, and precast reinforced concrete panels. Some distinguishing characteristics of these earlier precast reinforced concrete panel applications are that they were prefabricated and stockpiled for use by State department of transportation (DOT) forces, designed without load transfer, and intended for temporary service of up to 5 years, which they provided. The successful use of precast reinforced concrete panels for “temporary” pavement repairs during the 1970s in Michigan and Virginia is a testament to an innovative practice that has found renewed interest during the last decade, primarily due to limited work windows resulting from increased traffic volumes.

No further efforts were made in the United States before about 1995 to seriously investigate use of precast concrete panels for repair of concrete pavements or for rehabilitation of concrete as well as asphalt pavements. Since about 1995, there has been significant interest in the United States to investigate the effective application of precast concrete pavements as a strategy for accelerated repair, rehabilitation, and reconstruction of pavements.

## **FHWA/CPTP Initiative**

Recognizing the need to develop effective solutions for rapid rehabilitation of the Nation’s highway system, the Federal Highway Administration (FHWA) and the Michigan Department of Transportation (MDOT), as part of the FHWA’s Concrete Pavement Technology Program (CPTP), sponsored a study during the late 1990s that investigated the feasibility of using precast concrete panels for full-depth repair of concrete pavements. This work was conducted at the Michigan State University and has resulted in several field trials of this technology (Buch et al. 2006).

## **Industry Initiatives**

Parallel to the FHWA-sponsored efforts, several organizations in the United States also initiated development activities to refine precast concrete pavement technologies, primarily for repair applications. These technologies have certain proprietary features to the products and as such require licensing for use of the technologies. One of them, the Fort Miller Super-Slab® system, has been used on several production projects (continuous and intermittent) for repair and rehabilitation applications. In continuous application, this system simulates conventional jointed plain concrete pavement sections.

## **Highway Agency Initiatives**

In the last few years, several agencies have developed specifications that allow use of precast concrete pavement systems for repair applications. Also, several agencies have installed test sections to demonstrate the feasibility of using the precast concrete pavement systems. In addition, the Illinois Tollway has established a task force to develop a generic precast concrete pavement system for intermittent repair applications.

## **AASHTO Technology Implementation Group Activities**

Recognizing the increasing interest in precast concrete pavement technologies by US highway agencies, and to provide an effective platform for technology transfer activities, the American Association of State Highway and Transportation Officials (AASHTO) established a Technology Implementation Group (TIG) during 2006, to support technology transfer activities related to precast concrete pavements. The mission of this AASHTO TIG is to promote the use of precast concrete panels for paving, pavement rehabilitation, and pavement repairs to transportation agencies and owners nationwide and to present an unbiased representation to the transportation community on the technical and economic aspects of the current precast paving systems utilized in the marketplace. In June 2008, the AASHTO TIG completed work on the following documents:

1. Generic Specification for Precast Concrete Pavement System Approval.
2. Guidance and Considerations for the Design of Precast Concrete Pavement Systems.
3. Generic Specification for Fabricating and Constructing Precast Concrete Pavement Systems.

## **Strategic Highway Research Program 2 Project R05: Modular Pavement Technology**

The objective of Strategic Highway Research Program (SHRP) 2 activities is to achieve highway renewal that is performed rapidly, causes minimum traffic disruption, and produces long-lived facilities. A related objective is to achieve such renewal not just on isolated, high-profile projects, but consistently throughout the Nation's highway system.

The focus of Project R05 is to develop tools that public agencies can use for the design, construction, installation, maintenance, and evaluation of modular pavement systems. By necessity, the primary focus of this study will be precast concrete pavements. Project funding was established at \$1 million. Phase I of the study includes a review of modular pavement systems, review of highway agency and industry experience, and identification of successful strategies, promising technologies, and future needs related to modular pavement systems.

## **WHY PRECAST CONCRETE PAVEMENT TECHNOLOGIES?**

The primary use of precast concrete pavement technologies is to achieve construction time savings in high-traffic-volume highway applications and for rapid repair/rehabilitation applications at airfield pavements. Without the benefit of time saving, use of precast concrete pavement technologies cannot be justified economically, at least at current pricing for these systems. Use of precast concrete pavement technologies *must* result in reduced lane closures or better managed lane closures that in turn result in less traffic disruptions and improved safety at

construction zones. In addition, precast concrete pavement systems must be capable of providing low-maintenance service life of the desired duration.

The following factors need to be considered when assessing the use of precast concrete pavement as a viable candidate for rapid repair of concrete pavements:

1. Fabricating the precast concrete panels at a nearby plant. Plant location is critical for economical production repairs, to reduce cost and to reduce traffic disruptions.
2. Transporting precast concrete panels to the site (traffic issues, especially for night-time operations).
3. Site access for heavy cranes.
4. Rapid removal of old pavement.
5. Rapid preparation of the base/subbase.
6. Installing precast concrete panel on finished base/foundation
7. Matching adjacent pavement surface grade as closely as possible.
8. Interconnecting precast concrete panels and existing pavement using a mechanical load-transfer system, typically a version of the dowel bar retrofit technique.
9. Grouting the dowel/tie-bar slots, as applicable.
10. Injecting bedding grout to firmly seat panels, as applicable.

Intermittent repairs, such as full-depth repairs and slab panel replacement, are typically performed at night with a work window from about 8:00 p.m. until about 5:00 a.m. the next morning. Typically, 10 to 15 panel placements are targeted during each work window. The tight work windows and the need to open the facility to traffic by about 6:00 a.m. in the morning make it necessary that the contractor have sufficient equipment and manpower to complete the planned work each night.

## **PRECAST SYSTEMS FOR INTERMITTENT REPAIR OF CONCRETE PAVEMENTS**

Two types of intermittent repairs are possible using precast pavement systems:

1. Full-depth repairs—to repair deteriorated joints, corner cracking or cracking adjacent to the joint.
2. Full-panel replacement—to replace cracked or shattered slab panels.

The repairs are typically full-lane width. The process is similar for full-depth repairs and full-panel replacement, except for the length of the repair area. Key features of this application are:

1. Slab panel seating.
2. Load transfer at joints.

## Fort Miller Super-Slab System

The Super-Slab system is a proprietary precast concrete pavement technology suitable for both intermittent and continuous paving operations. This paving system is an assemblage of precast slabs placed on a precision-graded fine bedding material (maximum aggregate size of 12 mm [0.5 in.]). The transverse joints in the assembly of precast panels are fitted with standard dowel bars to provide load transfer. The basic features of the Super-Slab system are as follows:

1. Produce base within 1.2 mm (0.06 in.), using laser controlled grading equipment.
2. Place slab panels. Subseal with grout to eliminate voids.
3. Provide slab-to-slab interlock at joints through dowel/slot system.
4. Provide surface with 6 mm (0.25 in.) diamond grinding if better tolerance desired.
5. Thickness as specified (similar to jointed concrete pavement).
6. High-performance concrete, 27.6 MPa (4,000 lbf/in<sup>2</sup>) (or as required).

Figure 1 illustrates the typical slab panels used and the joint load-transfer system. This particular precast concrete pavement technology lends itself to the construction and rehabilitation of freeway entry and exit ramps because the manufacturer can produce panels with varying cross-slopes (warped slabs). This system has the most production paving experience to date. The system has been field-tested by New York State DOT, the Minnesota DOT, the Ministry of Transportation in Ontario, Canada, and others. Caltrans conducted accelerated load testing of the system in 2006.



**Figure 1. The Super-Slab System.**

## Uretek-Based System

In 1997, Uretek USA, Inc., introduced a new process for fixing faulted joints and restoring load transfer to concrete pavements. Uretek has developed two patented technologies. The first is the Uretek® Method, which is the process that employs high-density polyurethane foam to lift, realign, underseal, and void-fill concrete slabs that are resting directly on base soils. The

second is the Stitch-In-Time® Process, which is a repair system for restoring load transfer to jointed concrete pavements that are cracked, spalled, or otherwise damaged. Pavements undergoing repair are first undersealed using the Uretek Method, and then the Stitch-In-Time Process is applied to restore load transfer.

The Uretek-based system has been applied to installation of precast panels. The basic features of the Uretek-based precast concrete pavement system are as follows:

1. The panels are brought to the site on a flatbed truck and lowered into the excavated repair site.
2. The panels are elevated to the proper grade by injecting polyurethane foam under the panels.
3. The panels are stitched to the existing slab or to another panel using fiberglass boards, as illustrated in Figure 2.



**Figure 2. Uretek fiberglass stitches.**

For the longer length application of the Uretek system, expansion joints need to be placed at intervals of 14 to 18 m (45 to 60 ft); otherwise, premature slab cracking can develop.

### **FHWA CPTP System**

The FHWA CPTP technology is a doweled, full-depth system suitable for isolated or intermittent repair of highway pavements (Buch 2007). This is a nonproprietary precast concrete pavement technology. The repair panels are typically 1.8 m (6 ft) long and 3.7 m (12 ft) wide and fitted with three or four dowel bars in each wheelpath. The dowels are placed at 305 mm (12 in.) on center, and their diameter depends on the slab thickness. Typical design and installation details are illustrated in Figure 3. The dowel slots are cut in the adjoining existing pavement. This precast concrete pavement technology was developed in a partnership between Michigan State University and the Michigan DOT. This technology can be utilized to repair both jointed and continuously reinforced concrete pavements.

In the several demonstration/test projects constructed in Michigan (I-94 BL, I-196, M-25 and I-675), Virginia, and the Province of Ontario, the typical distresses exhibited by the candidate panels scheduled for repair included medium- to high-severity transverse cracking with associated spalling and faulted joints and cracks. The advantages of this technology include the

following: (i) the transportation to and storage of multiple panels at a jobsite or at the agency's maintenance yard does not present a problem; (ii) the panels can be mass produced at the precast plant or at the ready mix supplier's yard (the latter was demonstrated in Michigan); (iii) based on the geometry and proximity of repair sites, 8 to 10 panels can be installed in a day and be ready for traffic shortly thereafter; (iv) a typical agency paving mixture design can be used for the construction of the panels if appropriate moist curing is applied; and (v) the presence of dowel bars across the transverse joints ensures adequate load-transfer efficiency.



**Figure 3. The full-depth precast repair system (Michigan installation).**

## **FIELD APPLICATIONS**

Since about 2000, many highway agencies in North America have expressed interest in the use of precast concrete for intermittent repair or continuous applications in heavily trafficked urban areas where extended lanes closures are difficult. The U.S. and Canadian highway agencies that have accepted the use of precast pavement for production work include the following:

1. Caltrans
2. Illinois Tollway Authority
3. Iowa DOT
4. Ministry of Transportation, Ontario
5. Ministry of Transportation, Quebec
6. New Jersey DOT
7. New Jersey Turnpike
8. New York State DOT
9. New York State Thruway Authority

U.S. agencies that have investigated or are investigating the use of precast pavement include the following:

1. Colorado DOT
2. Delaware DOT
3. Florida DOT
4. Hawaiian Agencies
5. Indiana DOT
6. Michigan DOT
7. Minnesota DOT

8. Missouri DOT
9. Texas DOT
10. Virginia DOT
11. Airport Authorities
  - a. Port Authority of New York and New Jersey
  - b. Metropolitan Washington Airport Authority
12. U.S. Air Force

In addition to the North American initiatives, The Netherlands, France, Russia (previously Soviet Union), and Japan are actively investigating or using modular pavement technologies.

### **Illinois Tollway Experience**

The Illinois State Tollway Authority has used the Super-Slab system of precast panels for repair and rehabilitation of jointed concrete pavements.

#### *Emergency Repairs on I-294*

During December 2007, 17 Super-Slab precast panels were installed in an interior lane of the southbound direction of a section of I-294. All panels but one were 3.8 m (12 ft 5.5 in.) long by 3.6 m (11 ft 10.5 in.) wide. The panel thickness was 305 mm (12 in.). The work window was 8-hour, weekday nights. A practical problem encountered was the need for a low-rise crane due to the adjacent overpass. A heavier crane was needed to place the slabs, which required a weight permit. The emergency repairs were caused by problems with rapidly deteriorating pavement under winter conditions between lanes that had been rehabilitated.

#### *I-88 Ramps*

Approximately 37 Super-Slab panels were installed during June 2008 along an exit ramp and an acceleration ramp on I-88 in the Chicago area. The exit ramp, shown in Figure 4, made use of warped plane slabs that were placed during a weekend traffic closure. The panel thickness was 330 mm (13 in.). The acceleration ramp used nonwarped slabs (single plane), 305 mm (12 in.) thick. These panels, shown in Figure 5, were placed during several 8-hour, weekday, night closures.



**Figure 4. View of the exit ramp with the precast panels.**



**Figure 5. View of the ramp with the precast panels.**

### *Future Projects*

The Illinois Tollway Authority plans to use precast panels for repair and rehabilitation of deteriorated pavement areas. During 2008, the Tollway Authority began working with the local paving and precast industries to develop competitive options for use of precast panels. As of March 2009, a technical task force had developed a generic precast panel repair system that can be used on the Illinois Tollway for concrete pavement repair and rehabilitation.

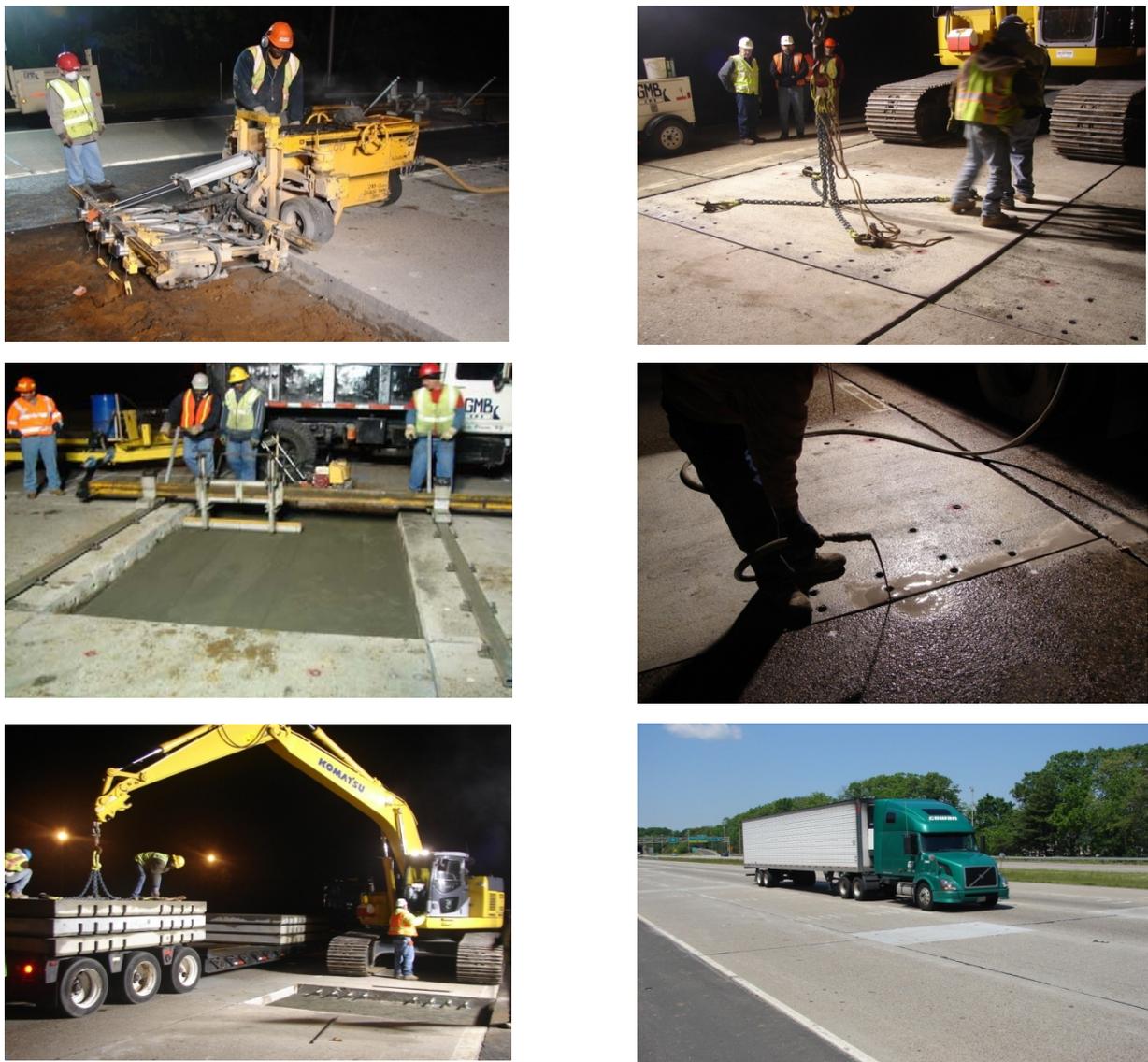
### **New Jersey Experience**

During 2007–2008, New Jersey DOT (NJDOT) used precast pavement for repairs along a section of I-295 in Burlington County. This first project was originally bid as a cast-in-place, full-depth, patching project. It was converted to a precast panel replacement project because of concerns with construction traffic management. A total of 5,760 m<sup>2</sup> (62,000 ft<sup>2</sup>) of precast panels were installed using the Fort Miller Super-Slab system. The project details are summarized below:

1. Three lanes in each direction.
2. Annual average daily traffic (AADT)—100,000.
3. Existing pavement:
  - a. 50+ years long jointed reinforced concrete pavement with panels 258 m (78 ft 2 in.) long and 19 mm (0.75 in.) expansion joints (by design) and multiple cracks per slab panel.
  - b. Slab thickness—229 mm (9 in.) over a 305-mm (12-in.) granular base.
  - c. Many expansion joints and cracks severely deteriorated, requiring repair or replacement of a large number of panels.
  - d. Repair areas located in all three lanes in each direction.
4. Repair panels—2.4, 3.1, 3.6, or 4.2 m (8, 10, 12, or 14 ft) long, full-lane width, 229 mm (9 in.) thick.
5. Load transfer at joints—Four dowels per wheelpath.

6. Nighttime placement—8:00 p.m. to 6:00 a.m.; work window about 8 hours.
7. Placement rate—8 to 16 panels replaced per night on average.
8. Process:
  - a. In advance—sawcut repair boundaries.
  - b. Night of repair—remove damaged panel; prepare base; drill dowel bar holes in existing adjacent panels; insert dowel bars; install precast panel.
  - c. Next night—patch dowel slots; underseal panel.
9. Testing—deflection testing using a falling-weight deflectometer to verify load-transfer effectiveness at transverse joints.

The precast panel installation steps are shown in Figure 6. During June 2008, a blowup occurred at one of the installed panel locations. The blowup was attributed to the elimination of the expansion joints in the repaired areas.



**Figure 6. I-295 precast panel installation.**

NJDOT has awarded two additional rehabilitation projects requiring use of precast panels:

1. Route 21, Essex and Passaic counties (under construction during 2008):
  - a. AADT—70,000.
  - b. Precast panel quantity—4,088 m<sup>2</sup> (44,000 ft<sup>2</sup>).
2. Route I-280, Essex County:
  - a. Awarded June 2008.
  - b. AADT—120,000.
  - c. Precast panel quantity—3,252 m<sup>2</sup> (35,000 ft<sup>2</sup>).

### **New York State Experience**

The New York State DOT (NYSDOT) and the New York State Thruway Authority (NYSTA) were two of the first agencies to implement the precast pavement technology for repair and rehabilitation of jointed concrete pavements. The first major project was installed by NYSTA during 2001 at the Tappan Zee Toll Plaza along I-95 using the Super-Slab system. Since then, several projects have been constructed using the Super-Slab System for repair and rehabilitation of jointed concrete pavements throughout the State of New York:

1. Belt Parkway, Queens
2. Rt 112, Long Island
3. Fordham Road, Bronx
4. X-Town, Schenectady
5. I-90, Albany
6. Korean Veteran Parkway, Staten Island
7. Route 9A, Manhattan
8. I-95, New Rochelle

All of the New York State projects have had the following in common:

1. High traffic counts:
  - a. Tappan Zee Toll plaza—135,000 vehicles per day (vpd).
  - b. I-90, Albany—105,000 vpd.
2. Short work windows (less than 8 hours).
3. Small work zones.
4. Night work (typically from 10:00 p.m. to 6:00 a.m.).
5. Use of the Fort Miller Super-Slab system.

The Tappan Zee Tollway Plaza project details are summarized below:

1. Produce base within 1.6 mm (0.06 in.) using laser-controlled grading equipment.
2. Place slab panels directly on grade and subseal with grout.
3. Provide slab-to-slab interlock at joints using a dowel/slot system.
4. Top surface within 6 mm (0.25 in.) tolerance; to be diamond-ground if not in compliance.

5. Load-transfer devices—31.75-mm (1.25-in.) transverse dowels; 19-mm (0.75-in.) longitudinal tie bars.
6. Astro-turf drag finish.
7. High-performance concrete—27.6 MPa (4,000 lbf/in<sup>2</sup>).
8. Production rate—average 20 panels installed per 8-hour shift (279 m<sup>2</sup> [3,000 ft<sup>2</sup>] with panels 2.1 m and 3 m [7 ft and 10 ft] wide and 5.5 m [18 ft] long).
9. Work area used two to three lanes at a time, with traffic using all other plaza lanes.
10. Open by 6:00 a.m. every day.

The precast panel installation under traffic is shown in Figure 7.



**Figure 7. Precast panel installation at the Tappan Zee Toll Plaza.**

The NYSDOT expects the precast pavement to perform equivalent to a cast-in-place concrete pavement. After some early failures along the Route 9A project in Manhattan, NYSDOT, during 2005, developed the following documents to ensure a quality product and minimize field installation problems:

1. Precast Pavement Design Guidelines:
  - a. Candidate project selection criteria.
2. Precast Pavement Material specifications:
  - a. Fabrication Standard Drawings.
  - b. Fabrication Working Drawings for projects.
  - c. Manufacturer's Installation Instructions:
    - i. Subbase preparation.
    - ii. Slab panel installation.
    - iii. Leveling of slab panels.
    - iv. Backfilling grout—use and strength gain.
  - d. Trial installation (test section).

### 3. Precast Pavement Construction Specifications:

- a. Joint layout plan by contractor.
- b. Slab panel installation process.
- c. Surface tolerances.
- d. Opening to traffic requirements.

NYSDOT and NYSTA continue to consider use of precast pavement on production projects and have ongoing projects (during 2008 and 2009). In addition, the Port Authority of New York and New Jersey (PANY/NJ) has also investigated the use of precast concrete pavement for roadway applications. During July 2003, the Super-Slab system was used by PANY/NJ to replace the approach slabs to the Lincoln Tunnel.

### **SUMMARY OF GAPS IN TECHNOLOGY AND PRACTICE**

While much progress has been made in the last few decades to improve precast concrete pavement technologies, many challenges remain. Some of the technical and institutional challenges are listed in Table 1.

The U.S. Congress established the second Strategic Highway Research Program (SHRP 2) in 2006 to investigate the underlying causes of highway crashes and congestion in a short-term program of focused research. As part of this program, a highway renewal research plan has been developed. The Renewal focus area emphasizes the need to complete highway pavement projects quickly, with minimal disruption to the users and local communities, and to produce pavements that are long-lasting. The goals of this focus area include applying new methods and materials for preserving, rehabilitating, and reconstructing roadways. The effective use of precast concrete pavement technologies for rapid repair, rehabilitation, and reconstruction of pavements addresses this goal.

The objective of the ongoing SHRP2 R05 project is to develop tools for use by highway agencies to design, construct, install, maintain, and evaluate precast concrete and modular pavement systems. These tools are to include the following:

1. Guidance on the potential uses of precast concrete pavement systems for specific rapid renewal applications.
2. Generic design criteria for precast concrete pavement systems.
3. Project selection criteria for precast concrete pavement systems.
4. Guidelines and model specifications for construction, installation, acceptance, and maintenance.
5. A long-term evaluation plan to assess the performance of precast concrete pavement systems and to refine these systems.

**Table 1**  
**Technical and Institutional Challenges for Precast Pavements**

1. Higher costs for constructing/installing precast concrete pavement systems in view of constrained agency budgets.
2. Field installation (production) rate.
3. Lane closure requirements for rapid installation: <ol style="list-style-type: none"> <li>a. Daytime between rush hours.</li> <li>b. Nighttime.</li> <li>c. Weekend.</li> <li>d. Extended.</li> </ol>
4. Sound understanding of factors that affect precast concrete pavement behavior and long-term performance. For precast concrete pavement systems, some of the critical factors are: <ol style="list-style-type: none"> <li>a. Load carrying capacity of each system component</li> <li>b. Seating and support condition (bedding requirements)</li> <li>c. Load transfer at joints between precast concrete units and between precast concrete units and existing pavement</li> <li>d. Connectivity at joints</li> <li>e. Expansion joint performance for precast prestressed concrete pavement (PPCP) systems</li> <li>f. For PPCP, impact of thin slab on deflection response</li> </ol>
5. Treatment for curved sections—use of 3D control for slab panel fabrication and base grading.
6. Optimizing various precast concrete pavement system design features.
7. Ensuring durability of the precast concrete pavement systems.
8. Lack of adequate long-term performance history.
9. Lack of adequate testing of precast concrete system components (e.g., joint connectivity).
10. Ready availability of nearby precast concrete pavement assembly/fabrication plants.
11. Well-developed process control (QC) procedures for different systems.
12. Well-developed acceptance testing (QA) procedures for different systems: <ol style="list-style-type: none"> <li>a. Slab panel dimensional tolerances.</li> <li>b. Ride/profile.</li> <li>c. Load-transfer effectiveness at joints.</li> <li>d. Initial faulting at joints.</li> </ol>
13. Treatment procedures for early failures.
14. Opening to traffic requirements.
15. Maintaining precast concrete system riding characteristics (safety related).
16. Maintaining vertical alignment at joints.
17. Lack of best practices for design, construction and maintenance and rehabilitation (M&R) of precast concrete pavement systems.
18. Lack of well developed, experienced based generic specifications for use of precast concrete systems.
19. General lack of support by the precast concrete industry to support refinement of the precast concrete pavement system technologies.
20. Lack of understanding of the technical capabilities by highway agencies of the potential of precast concrete pavement technologies. Need for technology transfer activities related to: <ol style="list-style-type: none"> <li>a. Selection criteria for use of precast concrete technologies.</li> <li>b. Generic specifications for use of precast concrete technologies.</li> <li>c. Design, construction, and M&amp;R issues related to precast concrete technologies.</li> </ol>

The primary warrants for use or promotion of precast concrete pavement technologies are rapid application and long-term durability. If the application is not rapid, it will not be relevant if it is not cost-competitive. If the application is rapid, but not durable, it will not have any future. As such, it is important that structural design features and precast concrete pavement material properties are optimized and integrated to assure rapid applications as well as long-life. For repair applications, the service life may be a few years to 20-plus years, depending on the projected rehabilitations needs of the pavement undergoing repair; but the other requirements are still applicable.

Although several types of precast concrete pavement systems have been successfully demonstrated and are considered proven systems, further research is needed in several key areas to ensure good long-term pavement performance, to improve constructability, and to make the systems cost-competitive.

## **IMPLEMENTATION GUIDELINES**

Based on the results of the Michigan and Ontario field trials (Buch et al. 2006; Lane and Kazmierowski 2005) and other recent installations of precast pavement repairs, the following practices are recommended for concrete pavement repair with precast panels:

1. Panel geometry: The panel size is dictated by the area to be repaired.
  - a. For full-depth repair, it is recommended that the panels be standardized as full width by 1.8 m (6 ft) long. The panels should be 13 mm (0.5 in.) shorter in both dimensions to allow the panel to be positioned in the repair area. When repair areas are longer, two panels may be used side by side.
  - b. For full-slab replacement, the precast panel should match the slab area to be replaced but be shorter by 13 mm (0.5 in.) in both dimensions. When repair areas include longer lengths, more panels may be used side by side.
2. Panel thickness: Verify the actual thickness of the concrete in the area to be repaired. The precast panels should be 6 to 13 mm (0.25 to 0.5 in.) thinner than the existing concrete pavement to account for variable thickness of the existing pavement. For economical precasting and management of panel inventory, it is advantageous if all panels are of uniform thickness and dimensions.
3. Load transfer at transverse joints: A load-transfer mechanism must be provided at transverse joints. Use of four dowel bars per wheelpath is recommended for heavy-truck traffic. Otherwise, three dowels per wheelpath may suffice. The following techniques may be considered:
  - a. Dowels embedded in the precast panel and slots provided in the existing pavement.
  - b. Slots provided at the surface in the precast panel and dowels inserted and epoxy-grouted in holes drilled in the existing pavement.
  - c. Load-transfer devices retrofitted after the precast panel is installed. This can be done by coring holes 102 mm (4 in.) in diameter and inserting a load-transfer device, such as the Double-Vee, in the holes.

- d. Slots provided at the surface in the precast panel and matching slots cut in the existing pavement after the panel is installed. Dowel bars are then installed using the dowel bar retrofit technique.
4. Bedding material:
    - a. A fast-setting cementitious flowable material placed before installation of the precast panel.
    - b. A fast-setting polymer-based material injected under the panel after installation of the panel.
    - c. Grading and compacting the base to the desired grade and placing the precast panel directly over the prepared base, especially for longer-length areas. A subsealing technique may be used to ensure proper seating of the panels.
  5. Provide an expansion cap at one end of the dowel bar to accommodate slab movement due to environmental loading and to prevent closing of the joint, especially for multilane roadways.
  6. Provide expansion material along one of the transverse joints to accommodate joint movement due to thermal expansion and contraction.
  7. Keep the dowel slot width as narrow as possible to reduce construction time and reduce the potential for dowel skewing in the horizontal plane.
  8. Take care to saw the repair areas perpendicular to the centerline to avoid skewing of the precast repair when it is placed.
  9. Multitask during the installation process to reduce construction time, particularly when the areas to be repaired are close together.
  10. Post-installation activities:
    - a. Grinding at the transverse joints to ensure desired ride level and to remove high spots at joints that might chip off later as a result of snowplowing operations.
    - b. Joint sealing, performed as soon as possible after installation, along transverse joints as well as longitudinal joints.
    - c. Deflection testing at joints to verify the effectiveness of the load-transfer mechanism.

## **SUMMARY**

Precast concrete pavement technology has seen significant improvements in the last decade. Several precast concrete pavement systems have been developed and are being implemented on production projects. While current precast pavement projects have been in service for only a few years, the field performance of the installed panels, though short in terms of time, indicates that precast pavement systems have the potential for providing rapid repairs that will be durable. In addition, limited accelerated load testing to date indicates that precast systems are viable alternatives for rapid repair and rehabilitation of existing pavements.

The installation of precast pavements may have a higher first cost compared to traditional cast-in place full-depth/full slab repair methods. However, the rapid application that minimizes lane closures and the long-term durability may easily offset the higher initial costs. The next few

years should see further improvements in the precast pavement technologies as the products from the SHRP 2 research program become available.

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# Evaluation of Concrete Pavement Repair Using Precast Technology in Virginia

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## ABSTRACT

The Virginia Transportation Research Council has recently evaluated the use of precast concrete patches for repairing jointed concrete pavement in Virginia. Six patches were placed: three had dowels cast into them during fabrication, and three had dowels inserted in place (dowel bar retrofit). Fabrication and placement were documented. The load transfer efficiency at the joints and the ride quality were determined approximately 2 weeks after construction.

After 1.5 years, the general condition of the patches was determined by a visual survey for cracks and spalls. In general, there were no distresses on the replaced slabs except for a few hairline cracks; however, there were failures in the joint area, mainly because of dowels, that were attributed to poor construction practices.

The Virginia Department of Transportation has planned another demonstration project in cooperation with the Federal Highway Administration's Highways for LIFE program for precast prestressed concrete pavement rehabilitation. This new project will include precast, precast prestressed, and cast-in-place slabs.

This paper summarizes the past work, the difficulties experienced, and the improvements that will be incorporated in the new project.

## INTRODUCTION

Concrete is a durable paving material that resists heavy and repeated loads, effectively providing long-lasting performance. However, deterioration occurs toward the end of the service life or prematurely because of base failure or variability in material and construction quality. When an area of concrete pavement is in need of repair because of extensive cracking, faulting, or spalling, the deteriorated concrete section is replaced with a concrete patch. If cast-in-place patches are used, the distressed concrete must be removed and the patch placed and cured before the repaired section can be opened to traffic.

In order to construct patches during the limited lane closures allowed, high early strength concretes are used. The durability of the patches can be compromised to meet high early strength requirements (*1*). The high cement content in high early strength concrete patches increases the chance of cracking because of thermal effects and shrinkage. The use of precast slabs as patches is an alternative to the use of cast-in-place patches. They can provide a higher quality

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product when strict time constraints are required. In some applications, they may also be more economical through the use of less cementitious products and possibly be placed faster than cast-in-place patches.

Because precast slabs are cast off-site, lane closure times could be reduced and a quality product achieved because of the controlled production environment. The reduced lane closure time was demonstrated in Michigan (1). The study showed that one slab could be placed in approximately 3 hours, from the time the deteriorated concrete is removed to the time the joints are sealed and the lane is opened to traffic. However, cast-in-place patches require additional time for setting and strength development before the lane can be opened to traffic. Thus, precast concrete patches may save time and money through a more durable patch material and accelerated construction. However, in full-depth precast patching, the selection of the bedding material is important because the material should enable proper leveling of the precast patch and provide sufficient support and drainage. In addition, the transfer of wheel loads between the slabs and the existing concrete must be done through properly used dowel bars or shear keys. The use of a durable grout for dowel bar and proper construction techniques are essential to ensure adequate performance of these joints.

Concrete slabs crack under tensile stresses. If the concrete is subjected to compressive loads through prestressing, higher tensile stresses would be needed to cause cracking. This would provide improved performance of the slabs under vehicle loads or environmental stresses. Therefore, prestressing should be considered when longevity is desired. Prestressed concrete slabs have been successfully used in several demonstration projects in the United State. The slabs can be pretensioned in the transverse direction and post-tensioned in the longitudinal direction to extend the service life. Prestressing the slabs enables increased durability, reduced slab thickness, and efficient load transfer. It also reduces the chances of cracking in the transverse and longitudinal directions.

Construction congestion and work zone safety have become national concerns. Work zones create unsafe conditions and are inconvenient to the traveling public. Therefore, reduction in needed maintenance through long-lasting pavements and rapid rehabilitation techniques are highly desired by highway agencies. The use of precast slabs in paving applications provides a rapid solution to rehabilitation with a quality product, and prestressing has the potential to extend the service life further. Both prestressed and regular precast slabs can also be used in large-scale pavement rehabilitation or in new construction.

## **BACKGROUND**

Cast-in-place patches are widely used by the Virginia Department of Transportation (VDOT) to improve rideability and protect the integrity of the distressed concrete pavement section. Precast slabs can provide a similar solution. In 2004, precast slabs were tried in an experimental project to repair distressed sections of a pavement (2). The precast slabs (patches) were 12 ft (3.7 m) wide (lane width) and 6 ft (1.8 m) long. A flowable fill material was use as the bedding material to ensure proper support. Six patches were placed; three had dowels cast into them during fabrication, and the other three had dowels inserted in place (as dowel bar retrofit) after placement of the patch. Several proprietary precast pavement systems are used by the industry with reportedly good success.

Three methods of installing precast concrete slabs were tried to repair damaged concrete pavements in Canada: (i) the Michigan, (ii) the Fort Miller Intermittent, and (iii) the Fort Miller Continuous methods (3). Fort Miller methods are proprietary system where as the Michigan method has originated from Michigan State University and Michigan Department of Transportation. In the Michigan method, three dowels are cast into the precast slab at each wheelpath. Dowel slots are cut into the existing pavement to accommodate the dowels. A cementitious flowable fill material is placed on the existing base prior to setting the precast slab. The slab is set on flowable fill material, and the exposed dowels are grouted in their slots to connect the precast slab to the adjacent pavement. In the Fort Miller Intermittent method, blockouts are cast into the precast slab to accommodate the dowels. Crusher screenings are placed on the existing base, precision-graded, and compacted. The dowel bars are grouted through ports in the precast slab to connect the precast slabs to the adjacent pavement. Bedding grout is then pumped, also through ports in the precast slab, to ensure that there are no voids beneath the slab. In the Fort Miller Continuous method, dowels and blockouts are cast alternately into a set of precast slabs that fit together and provide continuity. Crusher screenings are placed on the existing base, precision-graded, and compacted prior to setting the precast slabs. The first and last slabs are dowelled into the existing pavement at each end of the excavation, and intermediate slabs are connected to each other. All slabs in the continuous repair are tied into the adjacent lane with drilled and epoxied tie bars. Once the slabs have been set, the dowels and tie bars are grouted through ports in the precast slabs. Bedding grout is also pumped to ensure there are no voids beneath the slabs. The Ministry of Transportation Ontario (MTO) assessed all three methods as reasonable and met the load transfer efficiency (LTE) requirements of 70 percent as measured by falling-weight deflectometer (FWD). MTO recommended the use of dowel all the way across the joint rather than just in the wheelpaths.

Since about 2001, the Fort Miller system (Super-Slab®) has been used on several production projects (continuous and intermittent) for repair and rehabilitation applications. In continuous applications, the system simulates conventional jointed plain concrete pavement sections. Other proprietary systems are also used in such repairs (4). The Urettek® system has also been widely used, according to the developer, for intermittent repairs. This system requires the use of expansion joints if a series of adjoining panels is used. The Kwik Slab® system has been used on a limited basis in Hawaii. This system simulates long jointed reinforced concrete pavement sections.

VDOT has recently planned another demonstration project in cooperation with the Federal Highway Administration's (FHWA's) Highways for LIFE program for precast prestressed concrete pavement rehabilitation. This new project will include precast, precast prestressed, and cast-in-place slabs for a relative comparison. The investigation is consistent with the national interest in rapid construction with minimal disruption and longevity as echoed in the phrase "Get in, get out, and stay out!"

A study for determining the feasibility of using precast prestressed concrete pavement to provide improved durability and rapid construction was completed in 2000 by the Center for Transportation Research (CTR) at the University of Texas at Austin (5). This study was followed by an FHWA-funded implementation study conducted by CTR, which resulted in the construction of a 2,300-ft (701.0 m) precast prestressed concrete pavement pilot project near Georgetown, Texas, in spring 2002 (6). A total of 339 panels were used. Each panel was 10 ft (3.0 m) long, but some were full width (36 ft [11.0 m]) and others were partial width. Panels

were post-tensioned in 250-ft (76.2-m) sections. Each 250-ft (76.2-m) section took about 6 hours to place on a 2-in. (51-mm) hot-mix asphalt (HMA) leveling course covered with polyethylene sheeting for friction reduction. These slabs achieved acceptable ride quality, and diamond grinding was not needed. The second FHWA-funded demonstration project was conducted in California (7). A total of 31 panels were placed for a roadway 248 ft- (75.6 m) long. The length of the slabs was 8 ft (2.4 m) to facilitate transportation. Slabs were set on a lean concrete base and then covered with polyethylene sheeting to reduce friction. Placement of the 124-ft (37.8-m) posttensioned section took about 3 hours. The surface was then diamond ground for smoothness.

## **PURPOSE AND SCOPE**

This paper summarizes the repair of distressed concrete pavement using precast slabs without prestressing on Route 60 in Virginia. It presents the difficulties encountered and lessons learned by VDOT. It also describes a new demonstration project where precast patches will be placed with the focus on improving the weaknesses noticed in the previous study. It includes precast patches with and without prestressing and cast-in-place patches. This project will enable a comparison of different rehabilitation systems and will provide options for the contractor and owner.

## **PRECAST PATCH PROJECT ON ROUTE 60**

### **Description of Activities**

The project was located on US-60 eastbound about 0.5 mi (0.8 km) east of the New Kent and James City county line in Virginia. A total of six precast slabs were installed to evaluate the feasibility of such technology in concrete pavement rehabilitation. Fabrication and placement of precast patching slabs were documented. The patches were initially evaluated approximately after 2 weeks of construction for ride quality and load transfer efficiency (LTE) using nondestructive testing (NDT) methods: high speed profiler and FWD. After 1.5 years, the general condition of the patches was determined by a visual survey for cracks, spalls and joint condition.

The study included precast concrete patch installations with two types of jointing, three slabs each. In one type of jointing, patches were fabricated with dowel bars in-place at transverse joints, the existing pavement was sawcut (slotted) to receive the dowels, and the dowels were grouted into the existing pavement after installation of the patch. In the other type, dowels were retrofit after patch installation by cutting slots in the patch and existing pavement together. Three dowels were placed in each of the right and left wheelpaths on both transverse joints of the patch.

The existing pavement consisted of jointed reinforced concrete pavement, 9 in. (229 mm) thick, with a joint spacing of 30 ft. The concrete pavement reportedly was supported by 6 in. (152 mm) of soil cement. The pavement was initially constructed in 1948 using a six-bag concrete mix (564 lb of cement per yd<sup>3</sup> [334 kg per m<sup>3</sup>] of concrete).

### **Fabrication and Installation**

The patches were fabricated off-site at the contractor's shop. Dowels were cast in three of the patch slabs. During removal of the distressed concrete, the existing concrete was cut (slotted) to

receive these pre-placed dowels. In the other three slabs, dowels were retrofitted after placement of the slabs.

The thickness of the precast slabs was 8.5 in. (216 mm), which is less than the thickness of the old pavement to accommodate the subbase preparations. The mixture proportions for the concrete used in the slabs are given in Table 1. Concrete for the slabs was Class A3, which has a minimum 28-day compressive strength of 3,000 lbf/in<sup>2</sup> (20.68 MPa), and was air entrained. The grout used in both methods of dowel installation was commercially available general purpose high-strength grout (2). The 7-day compressive strength was expected to exceed 4,000 lbf/in<sup>2</sup> (27.58 MPa), and the 7-day bond strength was expected to be greater than 1,000 lbf/in<sup>2</sup> (6.90 MPa) when tested in accordance with Virginia Test Method 41 (8).

After the removal of the old patch, the existing subbase was leveled with gravel. Then, approximately 2 in. (51 mm) of flowable fill was used to level the base. The precast slabs were lifted at four preselected points and placed in the location using an excavator.

The difficulty in placing the precast patches involved connecting the new patch to the existing concrete. The joints were sealed with silicone over the backer rod and grout was poured in the places with dowels. For the slabs with preinstalled dowels, the receiving end in the existing slab was slotted before the placement of the precast slab. After placement, some areas were not level.

**Table 1**  
**Mixture Proportions**

<b>Ingredients</b>	<b>lb/yd<sup>3</sup></b>
Cement	496
Class F fly ash	124
Fine aggregate	1,072
Coarse aggregate	1,851
Water–cementitious material ratio	0.46
Air (fl oz/yd <sup>3</sup> )	6.4
Retarder (fl oz/yd <sup>3</sup> )	12.5

### **Load Transfer Efficiency and Ride Quality**

Approximately 2 weeks after construction, the LTE and ride quality were measured. The LTE tests were performed using an FWD on the right wheelpath. The testing protocol described in VDOT Materials Division's *Manual of Instructions* (9) was followed for the FWD tests. A series of four load levels (approximately 6,000, 9,000, 12,000, and 16,000 lbf/in<sup>2</sup> [41.37, 62.05, 82.74, and 110.32 MPa]) with three consecutive drops of each was used for each of the six patch locations. Ride quality was measured with a high-speed profiler at the same time FWD testing was performed. One of the measured properties from the profiler is the International Roughness Index (IRI), which indicates the smoothness of the pavement.

### **Condition Survey**

The condition of the pavement was determined through a visual survey of the cracks in the patches and grouted areas and spalls in the grouted areas about 1.5 years after placement.

## Results

### *Fabrication and Installation*

Concrete slabs with satisfactory strengths were cast. The compressive strengths for two cylinders at 27 days were 4,720 lbf/in<sup>2</sup> (32.54 MPa) and 4,706 lbf/in<sup>2</sup> (32.45 MPa). Although flowable fill was used to level the slabs, the patches had a differential height difference up to 0.25 in. (6.4 mm) in limited areas, necessitating greater attention to leveling. This difference in height greatly affected the measured ride quality (IRI data) when the patches were completed.

In the survey at 1.5 years, the contractor indicated that during installation of the slabs with pre-fabricated dowels, difficulties were encountered in aligning and centering the dowels. There is also some evidence of misalignment as indicated by cracks initiating at the corners of the dowel slots. The joint locations and dowel retrofit areas were exhibiting the problems and required more attention during construction. Specifically, in the first two patches, the grouted area was continuous between the new slab and the old pavement. The joint was not cut at that location, as shown in Figure 1. Thus, the joints at the slots were filled with grout instead of silicone. Three of the dowels in Slab 1 and all 12 in Slab 2 were reset or reinstalled within 2 months of initial construction. It is speculated that the slots were not cleaned well before grouting during the initial construction.

### *Load Transfer Efficiency and Ride Quality*

For each patch, LTE was measured using four load levels as mentioned previously. The results varied from 12 to 70 percent, with five of six patches scoring below 50 percent. According to the 1993 *AASHTO Guide for Design of Pavement Structures (10)*, LTE is divided into three categories: below 50 percent, 50 to 70 percent and above 70 percent. An LTE above 70 percent is assumed to provide satisfactory performance. The LTE values for the patches were obtained within 2 weeks of construction and deemed inadequate for such construction. Therefore, the dowel bars were probably not secured properly from the beginning and were not providing adequate load transfer between old and new slabs. Improper construction techniques may have contributed to these poor LTEs and may have resulted in the early deterioration of the grout used in securing the dowel bars. The grout may also have been a problem by not attaining the specified early strength.

Although the profiler was run for the entire 0.85-mi (1.4-km) section of the road, the IRI values presented in Table 2 are only for the patch locations as identified by the operator at the time of the test. Although patches are only 6 ft (1.8 m) long, the IRI values are an average for 15 to 20 ft (4.6 to 6.1 m) around the patch area. All IRI values were higher than 110 in/mi (1,760 mm/km), which was the allowable limit for the noninterstate roadways in accordance with VDOT specifications at that time; for IRIs above this value, a \$2 per yd<sup>2</sup> pay adjustment or a corrective action was needed (2). In most cases, the patch locations showed higher IRI (rougher pavement) relative to the overall average for the entire section. The recent field observations also revealed rougher joints at the patches.



**Figure 1. Discontinuous joint at dowel retrofit.**

**Table 2**  
**IRI Values for Precast Patch Locations (Test Date 03/24/04, Average Speed 40.6 mph)**

Patch Type	Patch No.	Average Distance (mi)	IRI Values on Wheelpath (in/mi)		
			Left	Right	Average
Retrofitted dowel bar	1	0.004	137	386	261
	2	0.003	103	157	130
	3	0.003	223	216	220
Preinstalled dowels	4	0.003	116	179	147
	5	0.004	289	251	270
	6	0.002	155	175	165
Project average			134	156	145

### *Condition Survey*

The field survey at 1.5 years, summarized in Table 3, provides more information on the lack of proper jointing between old and new slabs. According to the field survey, Patches 2 and 3 exhibited the worst grout conditions with more cracking. Patches 4, 5, and 6 (preinstalled dowels) were in relatively better condition than Patches 1, 2, and 3 (retrofitted dowels). Table 3 provides the number of cracked or spalled grout areas. In all three slabs cast with dowels, cracks were observed propagating into the patch because of the presence of dowel, as shown in Figure 2. In all patches, there were grouted areas with cracks. All except one of the slabs had spalled joint areas; in one area, the dowel was visible, as shown in Figure 3. In two of the patches cast with dowels, cracks were noticed propagating between the wheelpaths in a region

without dowels, as shown in Figure 4. In limited areas, the silicone joint material was missing or its surface was depressed up to 1 in. (25 mm) below the top of the slab as shown in Figure 5. This also indicates the need for better construction practices.

**Table 3**  
**Condition Survey Results**

Patch Type	Patch No.	Condition of Slab	Condition of Grouted Area (No. of Dowels of Total 12)	
			Crack	Spall
Retrofitted dowel bar	1	No distress	9	6
	2	Minor edge break	12	0
	3	No distress	12	9
Preinstalled dowels	4	Cracks from dowels	3	2
	5	Cracks from dowels and at mid-width	2	1
	6	Cracks from dowels and at mid-width	7	1



**Figure 2. Cracks propagating into patch cast with dowel.**



**Figure 3. Exposed dowel.**



**Figure 4. Crack propagating between wheelpaths in patch with cast-in dowel.**



**Figure 5. Missing silicone joint sealer.**

## **NEW DEMONSTRATION PROJECT**

VDOT has planned another demonstration project in a location with a very high volume of traffic where repair with precast slabs would be an appropriate application because of shorter construction windows and the high cost of construction-related traffic congestion and user delay. This project is near the Nation's capital, Washington, D.C., on I-66. It will facilitate a comparative evaluation of three repair options for concrete pavement:

1. Precast prestressed concrete slabs
2. Precast concrete slabs without prestressing
3. Conventional cast-in-place repair.

The precast prestressed concrete slabs will be used in a 1,000-ft (304.8 m) section on four lanes of I-66 west of Jermantown Road. Precast slabs without prestressing will be included for 2,000 ft (609.6 m) on the right lane of the ramp from I-66 westbound to Route 50 westbound. The conventional repair with cast-in-place slab will be used on the portion of the same lane and left lane (total is more than 2,000 ft [609.6 m]) on the ramp for comparison. The VDOT specification requires a 1-year warranty on a conventional cast-in-place patch. There is a requirement of 2,000 lbf/in<sup>2</sup> (13.79 MPa) compressive strength before the lane is opened to traffic; most contractors use high early strength concrete for such repairs. Several features of precast slab placement that will be carefully documented including the removal of the deteriorated concrete, the placement and leveling of the new slabs, the connection details with the old pavement, and the connection detail between the slabs and between the lanes. The slab thicknesses would vary between 9 in. (229 mm) and 11 in. (279 mm) for all three options.

This project was awarded incentive funding from FHWA's Highways for LIFE program as the technology and location satisfy the program's goal of advancing "Longer-lasting highway infrastructure using Innovations to accomplish the Fast construction of Efficient and safe highways and bridges."

The field demonstration will involve close collaboration with FHWA. The FHWA-designated consultants will provide the technology and expertise to instrument, cast, place, and post-tension the precast prestressed slabs. The slabs will be prestressed at the plant and post-tensioned at the jobsite. Although traffic slowdown and congestion cannot be avoided, the disruption in continuous traffic flow is not expected since only half the width of the I-66 mainline (two lanes) is planned for rehabilitation at one time. The same advantage is also expected on the ramp, where only the right lane will be repaired. On the right lane of the ramp (I-66 west to route 50 west), some slabs will be precast without prestressing, and some will be cast-in-place. All slabs will be placed on a base material that can provide uniform support (leveling course) with minimal friction.

The concrete mixtures used in the slabs will be evaluated. The testing will be conducted in the fresh and hardened states. In the fresh state, concrete will be tested for slump (ASTM C 143), air content (ASTM C 231), temperature (ASTM C 1064), and unit weight (ASTM C 138). Compressive strength will be determined at 28 days, 56 days, and 1 year. The elastic modulus, splitting tensile strength, and permeability (ASTM C 1202) will be measured at 28 days. Drying shrinkage will be measured for 6 months.

Several features of precast slab technology will be addressed in this demonstration project. These are based on the lessons learned in US-60 field trial, the experience of successful trials in other State departments of transportation, and the recommendations of AASHTO TIG Lead State Team (12). They primarily address the leveling and jointing of the slabs. Here are some of the important features:

- *Proper connection between the old pavement and the new slab and between the slabs (both longitudinal and transverse) is the most important issue using a precast slab. An LTE of more than 80 percent is required where LTE is measured using an FWD when differential deflection ( $d_{\text{loaded}} - d_{\text{unloaded}}$ ) exceeds 0.005 in. (0.125 mm) for a drop load of 9,000 lb (40 kN) on the wheelpath. In addition to the LTE requirement, the following are specified:*
  - The encasement material (grout fill) for pavement hardware (dowel bars) should be used in accordance with manufacturer's written instruction.
  - Completeness of placement at the encasement area must be demonstrated through drilling, retrieving, and inspecting at least two cores (6-in. [152-mm] diameter) from randomly selected hardware encasement locations (e.g., through dowel bars).
- *Precast slabs should be reinforced with a maximum center-to-center bar spacing of 18 in. (457 mm) in each direction. The minimum required steel to concrete ratio is 0.0014 in. (0.0356 mm) with a minimum cover of 2 in. (51 mm).*
- *An allowable tolerance for dimension is specified between 0.125 to 0.250 in. (3.2 to 6.4 mm) except keyway dimension tolerance of 0.0625 and 3 in. (1.6 and 76.2 mm) for*

*the position of lifting anchors.* The tolerances are provided for length, width, thickness, squareness, horizontal alignment, vertical alignment, deviation of ends (horizontal and vertical batter), position of strands (prestressed systems), position of posttensioning ducts at mating edges (post-tensioned systems), vertical and horizontal dowel alignment, dowel location, dowel embedment, location of reinforcing steel, straightness of expansion joints, initial width of expansion joints, and dimensions of blockouts. Diamond grinding is recommended for an elevation difference of more than 0.25 in. (6.4 mm) between old and new pavements.

- *Slabs could be placed on a precisely graded bedding layer and stabilized in place (underslab grouting) using cementitious grout to fill any small isolated voids.* The following features of the grout are specified:
  - Underslab grouting should be performed within 7 days of the placement of precast slabs.
  - Preapproved prepackaged nonshrink grout could be used if the manufacturer's recommendations are followed.
  - A nonshrink grout consist of a mixture of portland cement, a fluidifier, fly ash, and water could be used if initial set time of less than 4 hours and efflux time of 11 to 20 seconds are satisfied.
  - Stabilizing grout must develop a minimum compressive strength of 200 lbf/in<sup>2</sup> (1.38 MPa) within 24 hours.
- *The slabs could also be directly placed on cementitious support grout or urethane polymer foam.* Cementitious support grouts must develop a minimum compressive strength of 200 lbf/in<sup>2</sup> (1.38 MPa) before opening to construction or service traffic. On the other hand, urethane polymer materials must be fully cured in accordance with the manufacturer's recommendation. A complete support after slab placement should be demonstrated during trial installation by retrieving and inspecting at least three cores (6-in. [152-mm] diameter) from random locations.
- *The posttensioning tendon grout should be a prepackaged nonshrink grout conforming to the requirements for Class C grout specified by the Post-Tensioning Institute's Specification for Grouting of Post-Tensioned Structures (11).* The minimum compressive strength should be 3,000 lbf/in<sup>2</sup> (20.68 MPa) at 7 days and 5,000 lbf/in<sup>2</sup> (34.47 MPa) at 28 days.

The performance of the slabs will be monitored for at least 1 year primarily through a visual distress survey. The joint LTE, using the FWD, and smoothness (ride quality), using profiler, will be determined at least annually. Temperature and moisture gages will be embedded in the slabs to monitor the temperature and moisture of the slab during fabrication and over the life of the pavement. Temperature and moisture sensors will be located 1 in. (25 mm) from the top surface of the panel, at mid-depth, and at 1 in. (25 mm) from the bottom of the panel. Wire gages located 1 in. (25 mm) from the bottom surface of the slab will be considered to measure the in-service stresses. These instrumentations depend solely on coordination with the contractor and manufacturer of the precast slab.

## CONCLUSIONS

- *Experimental work on precast patches on Route 60 showed that the precast patches with quality concrete can be placed in a short period of time.* Particular construction issues related to jointing, leveling the slabs, and sealing the joints require special attention. In this limited study, the problems with aligning the dowels, consolidating grout around the dowels, and achieving good jointing were evident. The LTE tests revealed poor results, supporting the poor condition of the jointing that was evident in the condition surveys.
- *The grout material at the dowel locations was insufficient and needs improvement.* It must be durable, strong, and nonshrink to provide longevity.
- *The ride quality was poor mainly because of joint areas.*
- *Precast patches may provide contractors another option for limited lane closures if construction problems are resolved.*
- *The new demonstration project will document and present information on placement and performance and enable comparison of cast-in-place, precast, and precast prestressed patches.*

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## Use of Vitreous-Ceramic Coatings on Reinforcing Steel for Pavements

Charles A. Weiss, Jr.,<sup>1</sup> Sean W. Morefield,<sup>2</sup> Philip G. Malone,<sup>3</sup> and Michael L. Koenigstein<sup>4</sup>

### ABSTRACT

An innovative vitreous-ceramic coating for reinforcing steel that incorporates reactive calcium silicates from portland cement in an alkali-resistant glass has been shown both to increase the bond between the concrete to the reinforcing steel and to protect the steel from corrosion. The new enamel coating eliminates the weak layer that is associated with the interface between the steel and surrounding concrete. The vitreous coating is applied to the steel using the same process involved in porcelain enameling. In applying the enamel, the rod is coated with a porcelain slip containing portland cement and heated to approximately 1,562 °F (850 °C) for 5 to 10 minutes to allow the molten glass to fuse to the surface of the iron and the portland-cement component to become bonded to and embedded in the glass. The result is a tough, abrasion-resistant, hermetically-tight coating that develops the adhering properties of a portland-cement paste when contacted by fresh concrete.

Bleed water from the fresh concrete that normally produces a weak interfacial transition zone is taken up by the hydration of the surface layer of reactive calcium silicate. After only 7 days of curing, the chemical bond that forms is typically three to four times greater than that observed at the surface of undeformed, bare steel. The bond from the coated steel is as strong as the bonds between cement grains in the curing concrete. The lack of a weak interface results in the bond strength at the surface of the reinforcement increasing and not decreasing as the surrounding concrete cures and shrinks.

If microcracks develop in the coating, unreacted cement grains embedded in the glass coating will hydrate, forming calcium silicate hydrate gel, and raise the alkalinity. The self-healing effect in the glassy layer helps to protect the underlying steel.

In the construction of concrete pavement, the reactive, vitreous ceramic coating may permit shorter splices. The coating can also help insure that the shrinkage fractures that develop in pavement during curing remain within the desired tolerance limits. Since porcelain enamel does not delaminate, capillary transport under the coating does not occur. Porcelain enamels are considered the most durable and chemically-resistant coatings that can be put on steel. They can provide protection even in aggressive, high-chloride environments such as salt-treated pavement.

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## INTRODUCTION

Steel reinforcement and connector elements have had a long history of development in concrete pavement. As concrete pavement designs changed from jointed plain concrete pavement (JPCP) to jointed reinforced concrete pavement (JRCP) and continuously reinforced concrete pavement (CRCP), the amount of steel reinforcement and the complexity of design has increased. JPCP used dowel and tie bars to transfer loads between individual slabs 12 to 20 ft (3.7 m to 6.1 m) long. JRCP used reinforcing steel within each slab to control cracking within longer slabs that ranged up to 50 ft (15 m) in length. CRCP uses reinforcing steel to replace the contraction joints, and while cracks typically appear every 3.5 to 8 ft (1.1 to 2.4 m), the openings are held closed by the embedded reinforcing steel. The closed cracks, with the aggregate surfaces engaged, can transfer the load with minimum bending stresses.<sup>1-3</sup> New developments such as the use of precast concrete panels for pavement repair and ultrathin CRCP require the use of ever more complex reinforcement system in concrete pavement<sup>4,5</sup>. The requirements for the steel and concrete are continuing to increase as demands for better roads and road maintenance increase.

The use of steel reinforcement in concrete pavement encounters two problems:

1. The concrete surrounding the reinforcement steel changes composition as the pavement ages. Reactions with carbon dioxide in the atmosphere lower the pH of the concrete and destroy the passive iron oxide coating at the surface of the steel that prevented corrosion. Infiltrating solutions carry chlorides from the surrounding soil or from deicing salt down to the steel-concrete interface and increase the speed of oxidation of the steel.
2. It is difficult to control bonding of concrete to steel. Where steel is used to limit the lateral movement and crack openings, the concrete should bond to the steel tightly. Where steel is used in expansion joints, the steel dowels should not bond to the surrounding concrete. While it is possible to reduce the bond with the concrete, improving the bonding has been only marginally successful. The development of ceramic bonding enamel is an effort to provide a new coupling layer for steel reinforcement.<sup>6</sup>

Steel reinforcement in concrete is subject to corrosion, as the natural carbonation of the surrounding concrete allows the pH to drop and the passive iron oxide layer on the steel is lost. When the reinforcement steel starts to corrode the combined volume of the steel and the iron oxides, the increasing volume of corrosion surrounding the metal will put the concrete in tension. The volume of iron oxide is six times greater than the volume of iron that is oxidized. Cracking can occur when as little as 6 percent (by mass) of the steel has been corroded.<sup>7</sup> The concrete over the reinforcement will crack and spall, eventually putting the pavement in a distressed condition. The problems with corroding steel embedded in concrete are present whether the steel is rebar, metal support chairs, steel fiber, or dowels. The presence of chlorides from the soil or from deicing salts, greatly accelerates the corrosion rates. Corrosion problems in paving are most obvious in reinforced concrete bridge decks, where the surface freezing conditions require frequent application of deicing salts, but on-grade paving is also vulnerable.

Reinforcing steel typically has a relatively weak chemical bond to the contacting concrete. While other composite materials like fiberglass have a strong bond from the matrix to the reinforcement, in reinforced concrete comparable strong chemical bonds are not normally achieved. Deformed steel rod is typically used to obtain mechanical anchoring, but the limitations of this

approach show up when it is necessary to determine the overlap requirements for splicing rebar and the embedments required for anchoring rebar. Specifications can call for up to 45-rod diameters overlap in each splice (34 in. or 0.8 m for #6 rebar).<sup>8</sup> When organic polymer-coated reinforcement is used, longer overlaps are required.

Of all of the surface coatings applied to steel, silica and alumina oxide coatings are the most versatile in their methods of application (wet frit, electrostatic powder, plasma, etc.), and they can be engineered to produce excellent chemical inertness and mechanical properties. The mixed glass and ceramic coatings have been shown to have the best adherence and the best wear resistance of all the oxides. Porcelain enamel or vitreous enamel in industrial applications is a specially formulated durable glass that is fused to metal under high temperatures typically ranging from 1,100 to 1,600 °F (595 to 870 °C). The enameling process forms a layer at the interface that merges the chemical makeup of the glass and the underlying metal. The bond has many characteristics of a chemical bond and also maintains a mechanical bond due to the lower coefficient of expansion of the glass, which keeps it in a state of compression. Bond strength of 10,000 to 12,000 lbf/in<sup>2</sup> (69 to 82 MPa) can be obtained between steel and enamel.<sup>9</sup>

Vitreous enamel typically has hardness in the range of 3.5 to 6 on the Mohs scale of mineral hardness, where organic coatings are in the range of 2 to 3. Porcelain enamels can generally be engineered to produce abrasion resistance greater than the underlying metal. Generally, porcelain enamel will not fracture due to impact unless the underlying metal is permanently deformed. Porcelain coatings do not creep, and moisture generally will not penetrate under porcelain enamel.<sup>10</sup>

The reactive silicates in portland cement are a complex mixture and typically fuse at a temperature of 2,550 °F (1,400 °C). Firing the mixture of crystalline and glass compounds in portland cement into an enamel surface would not be expected to alter the composition of the silicates or aluminates except where the lower melting enameling glass can act as a solvent and adhere to the surface of the cement grains to form a bond.<sup>11</sup>

Lab-scale investigations with steel reinforcement were undertaken to demonstrate the bond strengths that can be obtained and investigate the effectiveness of the blended reactive silicate enamel in protecting steel reinforcement.

## **EVALUATION OF BONDING PROPERTIES**

The procedure for preparing test specimens used in this investigation has been presented in detail in an earlier report and follows conventional enameling techniques<sup>7</sup>. Test specimens were mild steel (ASTM C 1018) rods, 0.25 in. (6.35 mm) in diameter, cut to be 3 in. (76.2 mm) long. One end of the rod was threaded to allow it to be attached to the test apparatus. The rods were embedded in mortar to a depth of 3.5 in. (63.5 mm). The rods were furnished with a smooth, glass bead-blasted surface. The surfaces were further prepared for enameling using a grit polishing and water-based cleaning technique.

The composition of the glass frit for use on steel can vary with the manufacturer. In this study, the manufacturer was asked to furnish an alkali-resistant frit that would be a suitable ground-coat for a two-firing application. Most alkali-resistant frits are similar to conventional ground-coat enamels, but contain 4 to 6 percent zirconium oxide by weight. The good performance of zirconium glasses in high-alkaline environments is believed to be due to the relatively low

solubility of Zr–O–Zr species. In some applications, titania may be added to further improve the durability of the glass.<sup>12</sup> In some applications, for alkali-resistant glasses used in concrete, up to 13 percent zirconium oxide has been recommended.<sup>13</sup>

Porcelain enamels can be applied by making a slurry of frit and clay with water containing the necessary surfactants and suspending agents to achieve a stable suspension and the desired wettability and viscosity. The slip preparation used was PEMCO 06R-407 B-3 (PEMCO Corporation, Leesburg, Alabama). The test rods were coated by dipping the cleaned metal into the slip and letting the coating dry.

The porcelain enamel coating was fired onto steel at temperatures from 745 to 850 °C. Firing times were typically from 2 to 10 minutes, depending on the mass of metal to be heated and the size of the furnace. No attempt was made to obtain an even or smooth coating as would normally be the case for porcelain enamels for appliances, bathtubs, etc. (Figure 1). The enamel coatings had an average thickness of 0.8 mm.



**Figure 1. Examples of test rods prepared with various samples of glass frits with an outer coating of portland cement applied to the melted glass.**

The enameled test rods were embedded in a mortar prepared using the guidelines presented in ASTM C 109, “Standard Method for Determining Compressive Strength of Hydraulic Mortars.” Test cylinders were prepared for each mortar batch and tested to verify the unconfined compressive strength at 7 days was within the limits recognized for this mixture design.

Each enameled test rod was inserted in a long cylinder mold, 2-in. (50.8-mm) in diameter, 4 in. (101.6 mm) long, filled with fresh mortar. The rod was clamped at the top so that a 2.5-in. (63.5-mm) length of the coated portion of the rod extended into the mortar. Each cylinder was tapped and vibrated to remove entrapped air and consolidate the mortar. The samples were placed in a 100 percent humidity cabinet at 77 °F (25 °C) and cured for 7 days. After 7 days, the test cylinders were demolded, mounted in the test apparatus, and the force required to pull the rod out of the mortar was measured using an MTS Model 810 testing machine.

The results of the pull-out testing for coated and uncoated rods are presented in Table 1. Each series of test rods were prepared and tested in triplicate. The results are presented as the average

value and the standard deviation. The average bond strength is calculated as the force for pull-out distributed across the area of the metal-enamel interface.

**Table 1**  
**Comparison of Average Bond Strengths**

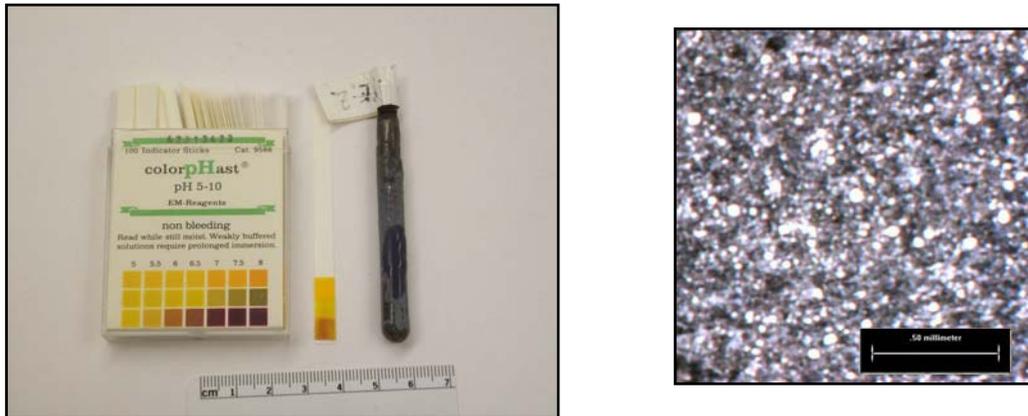
<b>Treatment</b>	<b>Average Peak Force lbf (N)</b>	<b>Std. Deviation lbf (N)</b>	<b>Average Bond Strength lbf/in<sup>2</sup> (MPa)</b>
Steel fiber embedded in mortar (various published sources)	---	---	295.4-394.5 (2.0-2.7)
Steel rods, uncoated embedded in mortar	588.7 (2,618.2)	104.8 (466.2)	298.8 (2.1)
Enameled rods without portland cement embedded in mortar	786.4 (3,497.9)	121.6 (540.8)	391.6 (2.7)
Steel rods, uncoated, surface roughened by grit blasting (reported by PPEC)	--	--	595 (4.1)
Enameled rods with portland cement (acid surface preparation reported by PPEC)	--	--	797 (5.5)
Rods with enamel containing portland cement embedded in mortar	2,500.9 (11,124.6)	52.9 (235.3)	1,274.9(8.8)

## INVESTIGATION OF CORROSION PROTECTION

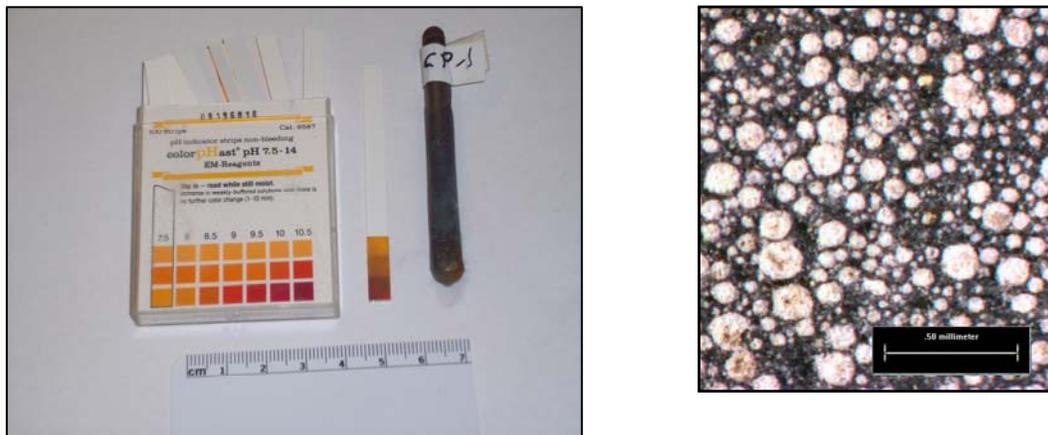
The examination of corrosion phenomena in the bare rods, enameled rods, and enameled rods with the portland cement addition was done by exposing sets of three identically prepared rods to a 3 percent sodium chloride solution in partly saturated sand (Figure 3). The goal of the testing was to provide conditions that would promote the mode of corrosion that would occur in carbonated (nonalkaline) portland-cement concrete that was contaminated with chloride. The pH of the wet, drained sand ranged from 6.0 to 6.5, and the temperature was maintained at 77 °F (25 °C). Because of the typically high electrical resistance of the enamel, corrosion will only occur if a hole is made in the enamel that exposes the metal surface. Vitreous enamel typically has a volume resistivity of  $1 \times 10^{14}$  ohm-cm; therefore, intact enamel surface is an insulator. Defects were prepared in each of the coated rods. All of the enameled rods were tested using the procedure outlined in ASTM C 876 and showed potentials more negative than -0.35 copper sulfate electrode (CSE) indicating that corrosion was occurring. The test rods were examined and photographed after 72 hours of salt water exposure and again after 40 days of exposure.

Examination of the test rods that were embedded in the salt water-sand mixture showed that, as expected, the enameled surfaces showed no detectable changes and the cleaned bare steel rods had begun to oxidize after 72 hours of exposure. Polished sections of rod surfaces after exposure are shown in figures 2 and 3. Note that the rods with enamel-only coating, in Figure 2, show the opaque particles that are in the slip and become part of the enamel during firing. The

larger particles in Figure 3 are the portland cement particles that were fired into the surface of the enamel.



**Figure 2. Enameled test rod (left) and polished section of side of rod after exposure (right). The moist pH test strip applied to the surface showed the pH of the abraded surface to be approximately 6.5.**



**Figure 3. Enameled test rod with portland cement embedded in the glass (left) and polished section of side of rod after exposure (right). The moist pH test strip applied to the surface showed the pH of the abraded surface was approximately 10.5.**

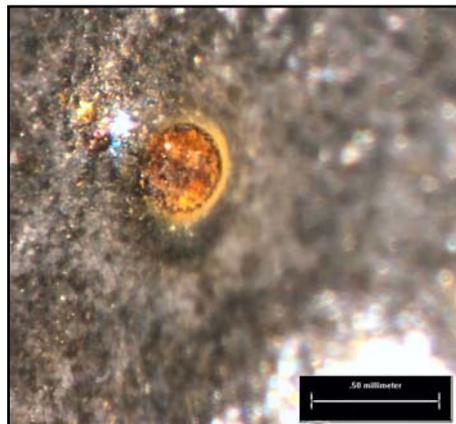
Examples of the effect on a bare steel rod and an enameled rod after 40 days exposure is shown in figures 4 and 5. Active corrosion was noted where steel was exposed to the salt-water saturated sand (left), and the build-up of iron oxide cemented the adjacent quartz sand to the surfaces of the bare steel rods (Figure 4). Enamel rods both with and without the portland cement addition showed no active corrosion where the enamel was present to protect the steel. Enamel does not debond, and studies with intentionally created defects that exposed the surface of the steel resulted in only local limited occurrence of corrosion. Figure 6 shows an optical photomicrograph of a polished section through an enameled test rod that had been drilled to remove the enamel. Note that the enamel does not delaminate or allow capillary transfer of fluids under the enamel.



**Figure 4. Corrosion on bare steel surface (left) and on enameled steel after 40 days of exposure in salt-water saturated sand. The corrosion of the bare steel cemented sand to the rod.**

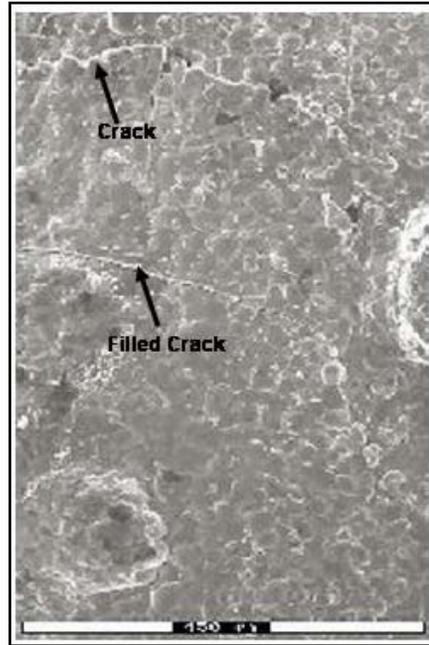


**Figure 6. Exhibition of evident corrosion of bare steel surface (left) and not evident on enameled steel containing portland cement after 40 days of exposure in salt-water saturated sand. The corrosion of the bare steel cemented sand to the rod. A powdery, alkaline deposit formed from the hydration of the exposed portland cement.**



**Figure 6. Optical photomicrograph of the edge of a hole drilled in the enamel to expose underlying metal. Rod sample was exposed to 3.5 percent NaCl solution for 40 days to produce corrosion. The enamel does not debond or allow capillary transfer of salt solutions.**

Portland cement bonded to the surface of the enamel hydrates when the coating contacts fresh cement, but unhydrated cement grains are embedded deeper in the enamel. If the enamel is cracked, the exposed grains will hydrate on contact with infiltrating moisture. The calcium silicate hydrate gel formed by the hydration reaction fills the crack and produces a self-healing effect. Figure 7 shows a cracked enamel surface that has been partly wetted to produce open and filled fractures.



**Figure 7. Surface of enameled metal wire bent to produce fractures and partly wetted to produce examples of open and filled fractures. The reacted cement on the surface produces the irregular surface texture.**

## SUMMARY

The investigation of steel reinforcement with bare steel, enameled steel, and enamel steel with embedded portland cement has shown the following:

1. The bond between the surface of reinforcing steel and concrete can be significantly increased by using a hydraulically reactive silicate fused into a layer of vitreous enamel fired onto the steel. The bond strength is increased by a factor of over three times over that developed with bare steel or with vitreous enamel only.
2. An intact coating of vitreous enamel or glass-ceramic composite coating can prevent corrosion of the underlying steel when conditions in the surrounding mortar or concrete would normally promote oxidation.
3. Enamel coating will corrode if the enamel is removed, but the corrosion is limited to the exposed metal and delamination and capillary effects were not observed.

4. The cement embedded in the enamel hydrates when exposed to water and controls the pH on the surface of the coated metal. Potentially, cracks in the ceramic-glass composite can maintain an elevated pH and inhibit the corrosion of any exposed steel.
5. Vitreous enamel application is a mature technology, and potentially glass-ceramic coatings can be used to protect and improve the performance of any steel reinforcement element used in concrete pavement including steel fiber, bar, welded mesh, and dowels.

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This material has been approved for public release; distribution is limited.



Concrete Pavements—Safer, Smoother, Longer Lasting

