Concrete Repair Best Practices: A Series of Case Studies

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16. Abstract

Concrete pavement restoration (CPR) techniques have gained greater national significance as DOT agencies attempt to further extend infrastructure service lives prior to required major rehabilitation or reconstruction. Various publications have documented design procedures and materials for CPR techniques, but less has been written about best practices for their construction, based on information from contractor and DOT agency practitioners. This report consolidates best practice case studies for six CPR techniques: cross-stitching, dowel bar retrofit, diamond grinding, full depth repair, partial depth repair and slab stabilization. Technical briefs for each CPR case study have also been written to accompany the main report. They have been separately published.

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Concrete Repair Best Practices: A Series of Case Studies

Final Report

Prepared for

Missouri Department of Transportation

Research Unit

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Prepared by

Applied Research Associates, Inc.

Champaign, Illinois

November 2017

Overview of the Report

This report is a compilation of six individual case studies on different repair methods. In addition, a technical brief has also been prepared for each case study. They have been separately published (see list below).

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#2 Dowel Bar Refit (DBR)

#3 Diamond Grinding

#4 Full Depth Repair (FDR)

#5 Partial Depth Repair (PDR)

#6 Slab Stabilization

This case study compilation as well as the individual technical briefs can be found in the MoDOT Innovation Library at http://www.modot.org/services/or/byDate.htm.

#1 Cross-Stitching Case Study for Kansas and Other Leading States

Abstract

Cross-stitching is a technique applied to an existing concrete pavement, primarily to longitudinal cracks and joints to prevent them from opening up. The causes of joints and cracks opening up are related to the daily and seasonal temperature and moisture cycles in the concrete slabs, vehicles that apply lateral shear forces during turning operations, differential settlements, and infiltration of incompressibles into the crack or joint. Tiebars are specified to control and prevent longitudinal joints from opening up. Normally, the tiebars are very effective and no significant widening occurs. However, sometimes tiebars are not specified or a construction problem occurs and bars are either left out or placed too low in the slabs to be effective. Longitudinal cracks seldom have any existing reinforcement and thus have little restraint to opening up over time (depending on base type). In fact, joints and cracks without reinforcement nearly always open up over time and, if allowed to continue, can cause tire safety problems and maintenance costs.

Cross-stitching has been used extensively for many years specifically to prevent an existing longitudinal joint or crack from widening. This case study report focuses on cross-stitching in the lead State of Kansas but also includes information from Missouri, Minnesota, and Utah. The Kansas specifications for cross-stitching are considered to be very effective. Cross-stitching is a valuable concrete pavement restoration (CPR) technique that has become standardized. In addition, inspection and acceptance of projects have seen no significant problems using this specification.

Interviews of experienced State personnel and contractors found no projects that had failed to keep the joints or cracks tight for up to 10 to 15 years of observations. Some older cross-stitched cracks (particularly in the wheel paths) were observed to exhibit spalling. Expert State and contractor personnel believe that the existing specifications are adequate and produce long-lasting cross-stitching that prevents crack and joint opening over many future years. The States and contractors interviewed report that the expected life of cross-stitching ranges from at least 10 years to well over 20 years.

1. Introduction

Cross-stitching is a well-established technique applied to an existing concrete pavement that has longitudinal cracks or joints that need to be kept tight over time. Deformed reinforcement bars are anchored into holes that are drilled at an angle and prescribed spacing along the crack or joint into the concrete slab. There are several factors that cause longitudinal joints and cracks to open

up over time, such as the daily temperature cycles and even moisture cycles in the concrete slabs, infiltration of incompressibles into an open joint or crack that prevent them from closing again, differential settlements, and vehicles that apply lateral shear forces during turning operations.

Longitudinal cracks and joints are the most often cross-stitched, but Kansas also includes the design for transverse cracks as well. However, this application should be used only after careful consideration of how a restrained transverse crack will affect the overall pavement (e.g., retrofit dowels may be a much better choice). There is also another technique called "slot-stitching" where deformed tiebars are anchored into slots cut across the joint or crack similar to dowel bar retrofitting. This procedure is not covered in this document (see SR903 - Stitching Concrete Pavement Cracks and Joints, ACPA).

Tiebars are typically specified to control and prevent cracks and joints from opening. Normally, they are very effective, and no significant widening occurs. However, experience has shown that if joints and cracks do not contain tiebars, or where a construction problem occurs and bars are either left out or placed too low in the slabs to be effective, a gradual opening process begins. Over time the joints and cracks often open up, which can cause tire safety problems and require costly maintenance.

Cross-stitching has been used often for many years to prevent an existing longitudinal joint or crack from widening. The projects below are a representative sample of cross-stitching projects to date:

- The first highway cross-stitching was performed on a section of I-70 in Utah in 1985. This project developed longitudinal cracking where 1,081 holes were drilled over a length of 1,800 feet of highway. "A review of the I-70 project in February 2000, after 15 years of service, found the project to be in generally good condition, with some faulting across undoweled transverse contraction joints. The performance of cross-stitched cracks was favorable in most areas. In some areas, spalling was noted between the holes drilled for the cross-stitch tiebars; cracks also traced from hole-to-hole in these areas. However, the cross-stitch cracks performed well overall, preventing lane separation and minimizing the settlement of the slabs." (ACPA, 1985).
- One Kansas cross-stitch project of note was performed in 2002 at the East Topeka Interchange project. During construction, several miles of the centerline longitudinal joint tiebars were installed very low in the I-70 12-inch slab (1.5 to 2 inches from the bottom of the slab) and also the K-4 portion of the project which was 9 inches thick. Given the near certainty that the joint would open up over time, it was agreed to cross-stitch using 0.625-inch deformed tiebars along the longitudinal joint with 4 cross-stitched tiebars at 3-ft spacing per 15-ft slab, alternating side to side. Observations after more than 15 years indicated excellent performance with no spalling, cracking, or joint widening occurring.

- In another project, Kansas Department of Transportation (DOT) ramps were paved full width with longitudinal shoulder joints. The pavement cracked down the middle, creating a great need for cross-stitching. These cross-stitching projects have performed very well, holding the longitudinal cracks tightly together. Kansas DOT has developed technology that has resulted in some effective specifications and standard drawings as needed for cross-stitching projects. In addition, inspection and acceptance of projects have seen no significant problems.
- Missouri has used cross-stitching on longitudinal cracking for 10 years on projects on I-70 and elsewhere with very heavy truck traffic. Rebar spacing is 24 inches on alternating sides of crack, using a 0.75-inch diameter rebar drilled at a 35 degree angle. This design results in 0.18 percent reinforcement across the crack for a 10-inch slab. Project performance indicates that longitudinal cracks are reasonably tight after 10 years under heavy truck traffic with some in wheel paths. Only a few spalling problems have occurred.

This document describes the pre-cross-stitching design considerations, then specifications, inspection and acceptance, and finally performance.

2. Pre-Cross-Stitching Considerations

Kansas and the other States typically perform cross-stitching to prevent a longitudinal crack or joint from opening up and creating a roughness, maintenance, and/or safety problem. Thus, cross-stitching is performed on concrete pavements of all ages:

- **Relatively new concrete pavements** that have developed early cracks (such as longitudinal cracks down the center of ramps in Kansas or the I-70 jointed plain concrete pavement in Utah) or at a joint where tiebars should have been but were not installed (such as the project in Kansas where tiebars were installed very low across the joint). Minnesota has used cross-stitching in last few years on some newer pavements and thinner overlays with longitudinal cracks.
- **Relatively older concrete pavements** that have longitudinal cracks that are starting to deteriorate (such as the longitudinal cracks in old Missouri jointed reinforced concrete pavement) and longitudinal joints that are beginning to open up with time (such as in urban freeways in Minneapolis where some interior longitudinal joints that were not tied are now opening up over time).
- **Crack location, width, and deterioration**. Longitudinal cracks that exist in the wheel paths can be cross-stitched if they are relatively tight and not deteriorated. The wider the existing crack, the harder it is to achieve good load transfer in wheel paths. Kansas has not had any known issues related to crack width to be cross-stitched. Minnesota requires cracks to be < 3/8 inch wide, believing that wider cracks may break down and become ineffective. Also, if a crack is spalled and working, it may not be a good candidate for

cross-stitching. Faulted longitudinal cracks or joints may be cross-stitched to help reduce future faulting if they are relatively tight.

The Kansas cross-stitching design is specified in Kansas DOT "Concrete Pavement Details, Tiebar Insertion RD723" (see Figure 1 and Figure 2). Holes are drilled on alternating sides of joint or crack at the following spacing:

- 30-inch longitudinal joints (results in 0.15 percent area steel for a 10-inch slab)
- 24-inch longitudinal cracks (0.18 percent area steel for a 10-inch slab)
- 12-inch transverse cracks (0.37 percent area steel for a 10-inch slab)

Figure 1 shows a diagram of the Kansas cross-stitch specification standard for different joints and cracks, as summarized above. Figure 2 shows a diagram of the actual cross-stitching bar with various dimensions. Note that the goal is to place the tie bars at mid-depth of the slab.

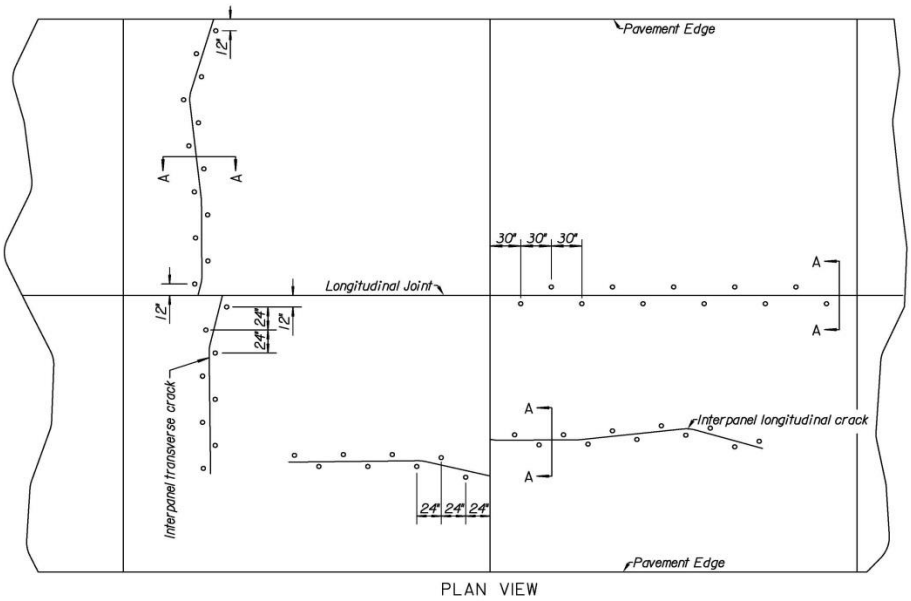

Figure 1. Kansas DOT, concrete pavement details, cross-stitching of joints and cracks.

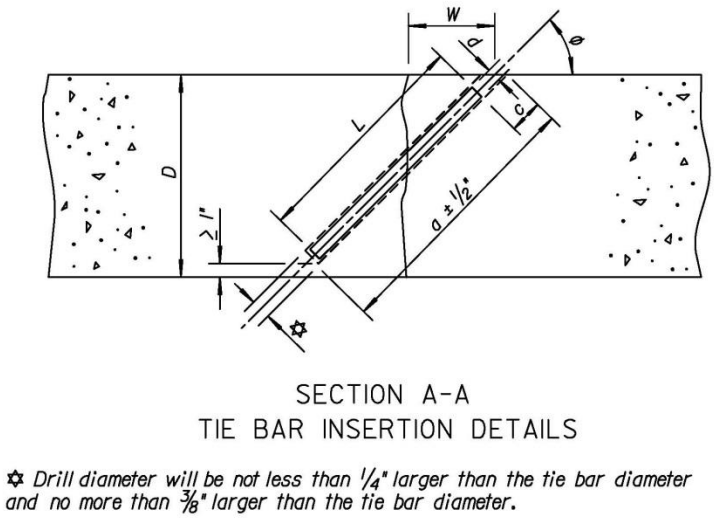

Figure 2. Kansas DOT, concrete pavement details, tiebar insertion, RD 723.

Note that all of the dimensions depend on slab thickness. Thus, it is important to determine the slab thickness, as this will directly affect the length of the rebar required in the hole, the angle with the horizontal and other dimensions. This is an interesting rebar design specification in that it provides increased reinforcement to help ensure that the most critical crack/joint is held tightly together, providing strong reinforcement to maintain a very tight joint or crack and even good load transfer under heavy truck wheels.

- **Longitudinal joint** reinforcement content of 0.15 percent is significant (for Kansas 0.625-inch bars spaced at 30 inches for a 10-inch slab). This is equivalent to that used in jointed reinforced concrete pavement (JRCP) longitudinal reinforcement of 0.10 to 0.20 percent.

- **Longitudinal crack** reinforcement content of 0.18 percent is also significant (for Kansas 0.75-inch bars at 24 inches for a 10-inch slab). This is generally larger than typical JRCP longitudinal reinforcement (0.10 to 0.20 percent).

- **Transverse cracks** will have to sustain every truck axle that drives down the lane, and if the reinforcement is not substantial, then the crack may break down. Kansas specifies a 0.75-inch bar with spacing at 12 inches which equates to reinforcement for a 10-inch slab of 0.37 percent area. This is more than double that used in typical JRCP longitudinal reinforcement (0.1 to 0.2 percent) to control transverse cracks.

This reinforcement percentage for design recommendations addresses holding the cracks and joints tightly together as well as load transfer over time quite well. Table 1 provides the Kansas

DOT slab thickness, drill hole angle with horizontal, offset from the crack or joint to drill the drill hole, rebar diameter, rebar length, depth of hole, and minimum depth of bar from surface.

Table 1. Kansas DOT cross-stitching hole drilling requirements with the goal

Slab Thickness D (in)	Angle with Horizon	Offset to Drill Hole W (in)	Rebar Diameter (in)	Rebar Length L (in)	Depth Hole a* (in)	Minimum Depth Bar C (in)
8.0	35°	5.75	0.75	9.00	12.50	3.00
8.5	35°	6.00	0.75	9.50	13.00	3.00
9.0	35°	6.50	0.75	10.50	14.00	3.00
9.5	35°	6.75	0.75	11.50	15.00	3.00
10.0	40°	6.00	0.75	11.00	14.00	2.50
10.5	40°	6.25	0.75	12.00	15.00	2.50
11.0	40°	6.50	0.75	12.50	15.50	2.50
11.5	40°	6.75	0.75	13.50	16.50	2.50
12.0	45°	6.00	0.75	13.00	15.75	2.25
12.5	45°	6.25	0.75	13.50	16.25	2.25
13.0	45°	6.50	1.00	14.00	17.00	2.50
13.5	45°	6.75	1.00	14.50	17.50	2.50
14.0	45°	7.00	1.00	15.50	18.50	2.50
14.5	45°	7.25	1.00	16.00	19.00	2.50
15.0	45°	7.50	1.00	17.00	20.00	2.50

*From the surface to the limit of drilling to prevent breaking out bottom of slab.

The following observations are based on Table 1.

- As slab thickness increases, the angle of drilling the hole increases, offset from hole to joint/crack increases, bar diameter increases, bar length increases, total depth of drilled hole increases, and minimum depth from surface to bar decreases.
- These requirements all help to refine and optimize the cross-stitching design.

Minnesota does not have a formal specification but uses an information sheet with requirements. Minnesota requires that holes are drilled at 24-inch spacing on alternating sides of crack. A similar specification table recommendation is provided in Table 2 that shows slab thickness, tiebar diameter, offset crack to hole, and bar length/drill angle from horizontal. For a 10-inch slab with 0.625-inch rebar every 24 inches, the reinforcement would be 0.13 percent for a longitudinal joint or crack.

Missouri conducts a preliminary survey 1 to 3 years ahead of construction to obtain approximate quantities of longitudinal cracking. They then increase that value about 10 percent at bid time to ensure their estimate is realistic. Missouri's specification is relatively simple, as follows:

- Rebar Spacing: 24 inches
- Angle from horizontal: 35 degrees
- Alternating side of joint
- Rebar diameter: 0.75 inches

For a 10-inch slab with 0.75-inch rebar every 24 inches, the reinforcement would be 0.18 percent for a longitudinal joint or crack. This value is typical of JRCP to control transverse cracks and is certainly sufficient for longitudinal joints and cracks to provide long-term crack tightness.

Table 2. Minnesota DOT cross-stitching hole drilling requirements.

Slab Thick (in)	Bar Diameter (in)	Offset (in)	Length (in)	Angle with Horizontal
8	5/8	5.75	8.50	35°
9	5/8	6.50	11.0	35°
10	5/8	7.25	12.5	35°
11	3/4	7.75 6.50 6.00	14.5 12.5 NA	35° 40° 45°
12	3/4	8.50 7.25 6.50	16.0 14.0 12.0	35° 40° 45°
13	3/4	7.75 7.00	16.0 14.0	40° 45°
14	3/4	8.25 7.50	18.5 16.5	40° 45°
15	3/4	NA	18.0	45°

The Utah project in 1985 followed the American Concrete Pavement Association (ACPA) design recommendations at the time. This included holes drilled on a 35- to 45-degree angle with the horizontal, alternating side of crack/joint, a rebar spacing of 24 inches, and a bar diameter of 0.75 inches. This design provides 0.18 percent reinforcement across the longitudinal cracks, which held up fairly well. For further information on cross-stitching, see the excellent references from the ACPA provided under References.

3. Cross-Stitching Specifications

Kansas, Missouri, and Minnesota all appear to have reasonable specifications (or instructions) and standard drawings for cross-stitching. Utah used the ACPA guidelines for cross-stitching. All of these States have successfully utilized these specifications, and contractors who have worked in these States affirm they are reasonable and effective. Table 3 summarizes the specifications other documents from these States.

Table 3. Summary of state specifications for cross-stitching.

State	Specification	Comments
Kansas	Kansas DOT 15-08003 Tiebar Insertion (Cross-Stitching)	Reinforcement content varies with application: longitudinal joints (0.15%), longitudinal cracks (0.18%), transverse cracks (0.37%). Typical Kansas longitudinal tiebar reinforcement = 0.34%.
Minnesota	Information sheet only: "Stitching Longitudinal Pavement Cracks."	Percent area of rebar typically is 0.13%.
Missouri	Standard Specification 613.50. Standard Drawing 613 (Sheet #3)	Percent area of rebar is 0.18%.
Utah	ACPA guidelines for cross-stitching.	Percent area of rebar is 0.18%.

Drilling of Holes

Kansas requires use of a hydraulic drill with tungsten carbide bits. The drill is mounted on a suitable piece of equipment. The drill rig frame is placed on the pavement surface such that the drilled holes are cylindrical and repeatable in terms of position and alignment on the surface being drilled. Electric handheld drills are permitted provided they can demonstrate same results as hydraulic drills. Contractors use automatic pneumatic drills, self-operated propelled drills that can lock in the proper angle. There has been no problem with spalling using these procedures.

The riskiest aspect of cross-stitching is making sure that the drilled hole does not go through the bottom of the slab. This is prevented by placing a cap on the drill. The drill hole diameter must be between ¼ and 3/8 inches larger than the diameter of the tiebar.

Anchoring Tiebar

Kansas requires that the drilled holes are cleaned in accordance with the anchoring material (epoxy) manufacturer's recommendations. As a minimum, holes are cleaned with oil-free and moisture-free compressed air. Then the approved epoxy is injected into the back of the hole, using a nozzle or wand of sufficient length. Tiebars are inserted into the hole to 1 inch below the pavement surface. The tiebar is rotated during installation. Epoxy is added or removed until it is flush with the pavement surface. The tiebar is inserted until the anchoring material is evenly distributed around the tiebar. A contractor adds that the bar should be slowly turned and pushed into the hole to make sure all air is out and bar fully covered. The epoxy amount used should be so that it slightly extrudes out the hole as the tiebar is inserted. Excess is removed, and the anchoring material is troweled smooth to the pavement surface, filling any chipped areas.

An experienced contractor who works in Missouri provided this information regarding drilling of the hole: Drill "pilot" holes about ¾ inch deep at proper locations. When the drill at a 35 degree angle hits the portland cement concrete surface, it will not "dance" around on the concrete and

create some damage. This has been a very effective technique. The contractor has successfully cross-stitched cracks/joints up to 1 inch wide, and they have performed well. The technique has not worked well on cracks/joints wider than 1 inch, and it should not be done. The contractor does not like the slot concept with a rebar for wide joints, as it does not work very well.

When can the cross-stitching be opened to traffic? Kansas specifies as soon as it is recommended by the epoxy manufacturer's specifications. Other guidelines state as soon as the bonding agent has cured.

4. Inspection/Acceptance

The inspection and acceptance process for cross-stitching focuses on the proper angle, spacing of holes, and anchoring of the tiebars into the holes. Slab thickness is a key factor that affects all of these items. Thus, observing that the specified process is followed is the key to inspection and acceptance of cross-stitching, which includes:

- Maintaining the angle of the drill.
- Making sure the drill cannot drill through the bottom of the slab.
- Checking the hole location (distance from the joint or crack) and spacing.
- Verifying the size of the tiebar.
- Checking the anchoring process, including cleaning of the hole and insertion of epoxy and bar in the specified rotational way.
- Observing that the drilling and anchoring procedures should not spall the surface of the concrete, as many projects have shown.

Missouri requires that unacceptable cross-stitching repairs must be mitigated by a method proposed by the contractor and acceptable to the engineer.

Incentive/Disincentive

There are no incentives/disincentives used by any of the States for cross-stitching.

5. Performance & Survival of Cross-Stitching

The performance of cross-stitching in Kansas and the other States is summarized below.

Kansas: Cross-stitching of longitudinal cracks and joints has maintained crack width over time, and no spalling has occurred. The 2002 East Topeka Interchange project with over 30 miles of cross-stitching of the longitudinal joint is still performing well, and the joint is very tight with no spalling. Overall, 20+ years are expected if performed according to the Kansas DOT specs.

Missouri: The oldest cross-stitching projects are 10 years old. These projects exhibit only a few locations of spalling of the longitudinal cracks. These projects on I-70 and elsewhere were under very heavy truck traffic, and some cracks were in the wheel paths.

Minnesota: Longitudinal cracks have maintained crack width over time. One project in Minnesota was a 5- to 6-inch thin portland cement concrete overlay with longitudinal cracks. The project is now nearly 10 years old, and longitudinal cracks are still in good condition. Overall, a 20+ year service life is estimated.

Utah: The I-70 project in Utah in 1985 involved reflective longitudinal cracks and was re-examined after 15 years of service. The performance of the cross-stitched cracks was favorable in most areas, but some areas had crack spalling between the holes. Overall, the key result was that the cracks were held tight, which is the critical objective of cross-stitching.

Cross-stitching has been performed on slabs typically 7 inches or thicker successfully. One contractor reported that cross-stitching performed on a 5-inch concrete slab also worked out well.

Thus, overall the performance of cross-stitching shows that this technique is capable of holding longitudinal cracks and joints together over a significant timeframe ranging from 10 to 20 years or more if properly installed.

6. Summary of Cross-Stitching

The cross-stitching technology has been around for a long time and has proven to be an effective repair technique to stabilize and hold tightly together the following distress types:

- **Longitudinal joints** where tiebars were not installed or were installed improperly and may begin to open up, creating potential maintenance and safety problems. Kansas has an effective and detailed design and specification for this application with reinforcement content of 0.15 percent.
- **Longitudinal cracking** develops in any type of concrete pavement, and it is desired to keep cracks as tight as possible for as long as possible. Kansas has an effective and detailed design and specification for this application with reinforcement content of 0.18 percent.
- **Transverse cracking** where it is desired to hold the cracks as tight as possible for as long as possible. Kansas has a design (higher steel content of 0.37 percent) and specification for cross-stitching transverse cracks that appears to be effective in holding them together and prevent spalling and faulting. This application should only be used after full consideration of the impact that locking up a transverse crack will have on the pavement.

Cross-stitching has produced a significant increase in the life of longitudinally cracked slabs and for longitudinal joints with no or ineffective tiebars through holding them tightly together over

10 to 20+ years. The Kansas inspection and acceptance procedures are observational but are fairly straightforward and effective. Slab thickness is a key input to proper cross-stitching. The State and contractor staff interviewed believe that the specifications are adequate and produce long-lasting cross-stitching and prevent crack and joint opening over many future years.

7. References

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#2 Dowel Bar Retrofit Case Study for Washington and Other Leading States

Abstract

This document presents a case study for dowel bar retrofit (DBR) for the lead State of Washington. It also discusses the specifications and experience of other State leaders and contractors from Missouri, California, Minnesota, and Utah. DBR technology has been developing for many years, primarily to increase the load transfer efficiency of existing transverse joints with no dowel bars but also for working transverse cracks. This case study is based primarily on specifications and interviews with the States and contractor staff focusing on providing long-term DBR performance.

The case study identified four key aspects to DBR. The first key is to limit usage to the appropriate slab locations and conditions. Washington, California, Minnesota, and Utah use DBR for non-doweled transverse joints where there is no deterioration in the bottom half of the slab. Missouri and Minnesota use DBR for working transverse cracks in jointed reinforced concrete pavement (JRCP). Properly installed dowel bar retrofits have had an extraordinary impact in controlling the amount of faulting over time on both non-doweled transverse joints and working transverse cracks that exhibit sound concrete.

The second key is the detail and effectiveness of the specifications, special provisions, and standard drawings (design) for DBR. Substantial information is provided from Washington and the other States regarding their designs and specifications for DBR including field layout, diameter of the bars, removal of concrete and cleaning of the slots, reforming of the joint, and proper placement of the dowel assembly (lightly lubricated).

The third key is the slot material, where proprietary rapid setting and early opening cementitious mixtures have been used with success in the small slots.

The fourth key is the inspection/acceptance procedures and their effectiveness. Warranties may be an effective component.

DBR has produced a significant increase in the service life of faulted joints in jointed plain concrete pavements and working cracks in JRCP where the specifications are effective, repair materials are durable, and inspection/acceptance procedures are followed. In these cases, a life of 10 to 20+ years has been achieved. However, everything must go right—if there are

deficiencies, the DBR will experience a shorter service life. This makes the case for increased just-in-time training of State and contractor personnel.

1. Introduction

Dowel bar retrofit (DBR) is a well-established technique applied to an existing jointed plain concrete pavement (JPCP) that has developed transverse non-doweled joint or working crack faulting. In addition, DBR has been used to prevent further deterioration of working transverse cracks in jointed reinforced concrete pavement (JRCP) that typically include limited reinforcement and open up and spall or even fault over time. DBR is an important component of overall concrete pavement restoration (CPR).

Transverse joints and cracks have an overwhelming tendency to fault under repeated heavy loads and precipitation (especially with an erodible base) if sufficient dowels or reinforcement are not provided to carry the shear forces to reduce differential deflections. DBR is defined as placing smooth and lightly coated (parting material) steel dowel bars in prepared longitudinal slots located in truck wheel paths across a traffic lane and surrounded with cementitious material that hardens similar to that of concrete, thus providing a high degree of load transfer efficiency (LTE) for the joint or crack. The high LTE reduces deflections across the joint, especially differential deflections, which reduces pumping and erosion beneath the slab, thereby limiting further faulting of the joint or crack.

The Washington State Department of Transportation (DOT) constructed its first full-scale DBR project in 1993 for the repair of a severely faulted concrete pavement. The DOT has completed many projects since then with excellent success. (Pierce, Muench & Mahoney, 2009) The success of DBR has been very good and consistent over time for Washington and the other States after solving some early construction problems. Interviews with State and contractor staff indicate there are several key aspects to providing effective DBR: (1) usage limited to the appropriate slab locations and conditions; (2) appropriate detail and effectiveness of the specifications, special provisions, and standard drawings; (3) a durable slot material, and (4) effective inspection and acceptance procedures.

Faulting in the existing pavement can be easily removed by diamond grinding. However, the joint or crack will still re-fault over the next few years, often more rapidly. Thus, there is a great need for DBR to prevent the re-faulting of the project joints and cracks and thereby extend the service life of the pavement. Faulting of transverse joints and cracks occurs at nearly every joint or crack that does not include a sufficiently sized dowel bar or deformed rebar. There are several factors that must be present to create faulting: (1) heavy repeated axle loads; (2) lack of dowels of sufficient diameter and sufficient deformed reinforcement content across a crack that provides long-term high LTE to control deflections; (3) free water at the bottom of the slab or base course; and (4) erodible material in the base, subbase, or subgrade. It seems that even if the base is

"non-erodible," some joint faulting can still occur, creating roughness. All four factors exist for a large number of older JPCP and JRCP projects.

This case study report focuses on DBR in Washington but also includes information from Missouri, Utah, Georgia, and California. The Washington specifications and acceptance procedures have evolved over many years into a reliable and cost-effective process that has produced excellent long-term performance. The other States' DBR specifications are also excellent, and each adds some valuable information. This report first describes pre-DBR design considerations, followed by specifications and construction, inspection and acceptance, and finally performance.

2. Pre-Dowel Bar Retrofit Considerations

Washington and the other States typically perform DBR to provide improved joint load transfer of non-doweled transverse joints and working cracks that result in long-term prevention of joint or crack faulting. Diamond grinding after performing DBR eliminates pre-existing faulting. DBR can provide a long-term service life extension in the order of 10 to 20+ years. Of course, there are many projects where this much life extension is not possible due to severe durability deterioration or inadequate structural capacity (thin slab). However, a JPCP with no dowels or JRCP with working transverse cracks may be a good candidate for DBR as part of pre-overlay repair to improve LTE and lessen the severity of the reflection cracking.

Typically, justification for DBR requires a design service life of 10 to 20 years or more. For example, one Washington faulted JPCP was subjected to CPR (including DBR) in 1995 that has still not developed any significant faulting. Another example is a JRCP project in Missouri where transverse cracks were working and faulting when DBR was performed as part of overall CPR. These projects are still in service 10 years later and in good condition. Thus, DBR as part of CPR is critical to a successful future of non-doweled JPCP as well as transverse working cracks of long-jointed JRCP.

Appropriate Existing Condition for DBR

In Washington, ideal candidate projects for DBR are those that are 25 to 35 years old and have mean fault measurements 0.125 to 0.25 inches. Their research indicated these projects perform better than those with larger amounts of joint faulting. On the other hand, Washington will only diamond ground, without DBR, pavements with mean faulting greater than 0.5 inch. (WSDOT Pavement Policy, June 2015) JPCP with excessive amounts of joint faulting would also typically have large amounts of loss of support and rocking panels which DBR could not eliminate.

Most usage of DBR has been with non-doweled JPCP, but States like Missouri and Minnesota have JRCP, and they have used DBR to stabilize the transverse cracks if they begin to become working cracks that fault. Missouri's DBR on transverse JRCP cracks have stabilized these cracks, extending their life more than 10 years.

Some slots in Washington have failed when longitudinal cracks in the existing pavement form at a DBR slot. When a DBR slot is intersected by a longitudinal crack, Washington does not chip out the slot and place a DBR, but will rather just clean and reseal the sawcut with an epoxy material. Sound concrete must exist at transverse joints and cracks throughout the depth of the slab. Coring and visual inspection are required to ensure there exists sound portland cement concrete (PCC) in the lower portion of the slab. Minnesota strongly contends that if there is significant lower slab deterioration (lower portion of core falls apart), then DBR may not be effective and full depth repair is more effective for that joint.

Are Dowel Bars Required to Control Joint/Crack Faulting?

The main question that must be answered to justify spending additional funds for a CPR project is, "Are dowel bars required to control future transverse joint and crack faulting?" One way to answer this question is to examine the existing pavement and measure the amount of faulting that exists since construction or a previous diamond grinding operation. The typical magnitude of what is considered "significant" faulting for a JPCP with short joint spacing is 0.125 inches. This much mean faulting affects International Roughness Index (IRI) and user ratings significantly, and many States use this level for pavement management decisions, as does Washington for diamond grinding. Thus, if an existing JPCP has mean joint faulting close to this value over a number of years, then it is highly likely that after diamond grinding the existing pavement will begin the faulting process again, and the faulting may develop at a more rapid rate (e.g., traffic is heavier and joint or crack LTE is lower).

One tool that is now available to predict future joint faulting after an existing JPCP has been diamond ground is the AASHTOWare Pavement ME Design software. The "Restoration" pavement type must be chosen and the required data entered to run the program to estimate mean joint faulting over the next 10 to 20 years. Figure 3 shows the projected joint faulting for a JPCP over a future 20 years, both with and without DBR of varying diameters. This is a valuable tool to provide another estimate of what will happen if no dowels are installed versus a full DBR installation with varying diameters. Figure 3 shows the potential for extra life until critical faulting is reached. Note the impact of dowel diameter on mean joint faulting, which is due to lower dowel/concrete bearing stress under load. The Pavement ME Design software also calculates slab fatigue transverse cracking and IRI, given the after-grind slab thickness.

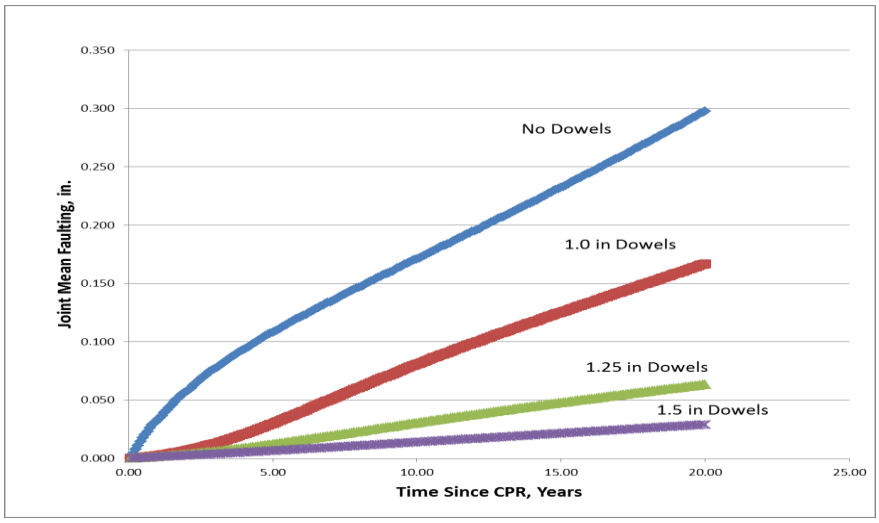

Figure 3. AASHTOWare Pavement ME Design example output for a JPCP CPR grinding project, with and without DBR.

Design and Layout of DBR

Washington has devoted substantial effort to the development of a detailed joint DBR layout plan and details as shown in Figure 4, Figure 5, Figure 6, Figure 7, and Figure 8 (Washington State Standard Plan for DBR, A-60.20-03). Another excellent reference source for DBR is, "Dowel Bar Retrofit: Do's and Don'ts," by Pierce, Weston, and Uhlmeyer (2009), which is recommended for many more details and step-by-step of DBR. Many critical details can be found on these diagrams, including spacing and diameter.

Washington requires three dowels per wheel path that are spaced at 12 inches apart. One key dimension of note is the 18-inch spacing from the outer edge of the slab to the first dowel bar. Retrofit dowel bars used to be placed at 12 inches from the longitudinal lane/shoulder joint. This often resulted in various random cracks in the area of the dowel. Washington modified the spacing from 12 to 18 inches from the outside edge, and this eliminated the random cracking and placed the dowels more in the truck wheel paths. Another notable dimension is the large 1.5-inch dowel diameter, as noted below:

- Dowel diameter: 1.50 inches. Large diameter dowel is extremely effective in reducing dowel/concrete bearing stress to greatly reduce joint faulting.
- Dowels epoxy-coated.
- Dowel expansion caps tight-fitting, non-metallic material.
- Chairs are epoxy-coated or non-metallic material.

Washington, Utah, and California (>9-inch slab) use the 1.5-inch dowel diameter in their DBR. Minnesota and California (<9-inch slab) use a 1.25-inch dowel. Missouri uses three 1.25-inch dowel in each wheel path. The different sizes appear to work for the States involved with the

projects they have constructed. For only three dowels per wheel path, the use of a 1.5-inch dowel may very well be worth the extra cost given the variations in placement of DBR.

California has DBR usage criteria that the LTE of the transverse JPCP joints be less than 70 percent (non-doweled joints are nearly all less than this value in cooler weather) and the extent of existing slab cracking is less than 5 percent of slabs. Projects in California have shown an increase in transverse joint LTE from 30 to over 80 percent after DBR (Smith and Alarcon, 2002). A large amount of cracking is indicative of loss of support and rocking slabs and that an acceleration of fatigue cracking is likely to occur as the pavement receives increased levels of truck traffic.

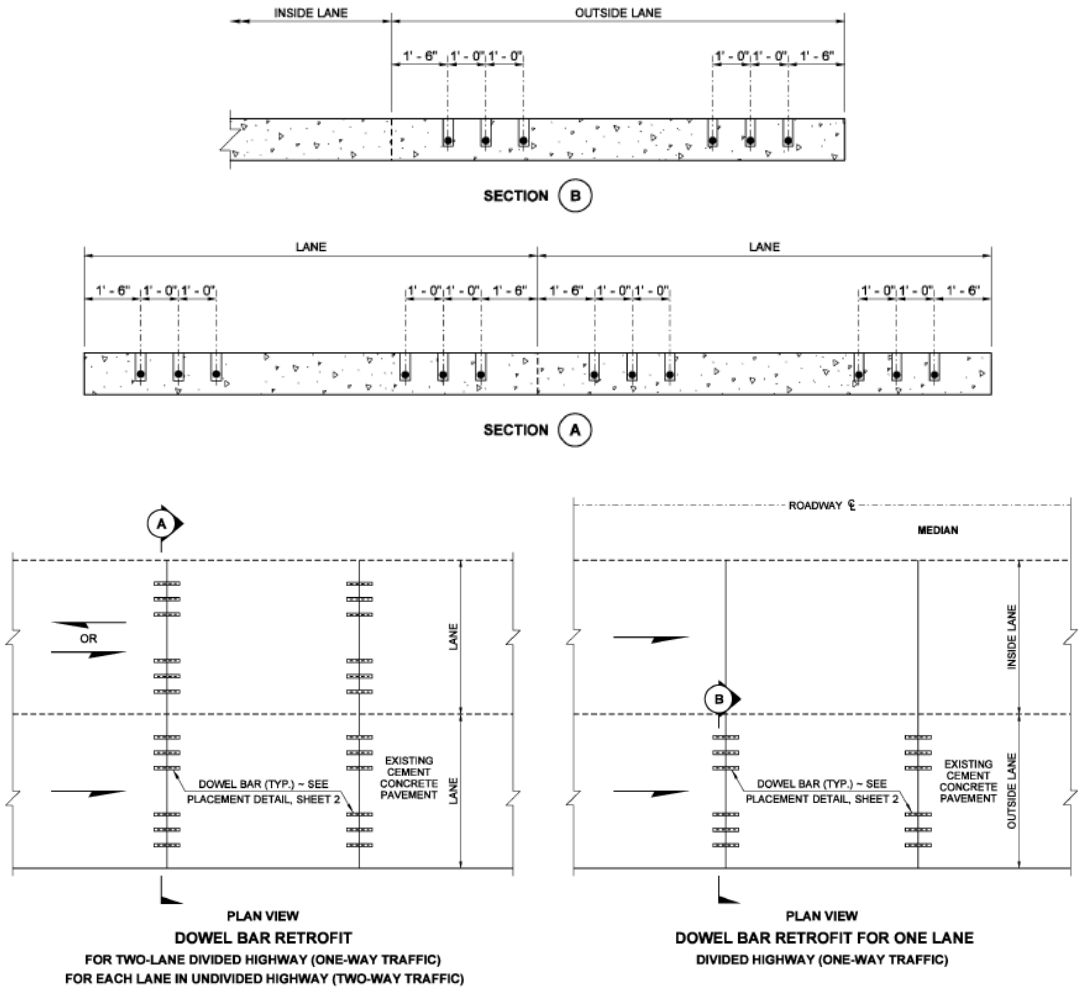

Figure 4. Washington State DOT DBR design and plan layout diagram for perpendicular joints. (Washington State DOT Standard Plan for DBR, A-60.20-03)

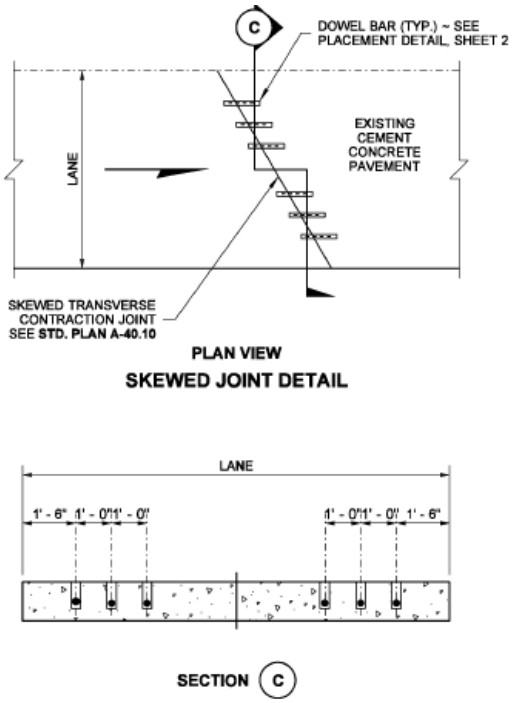

Figure 5. Washington State DOT DBR design and layout diagram for skewed joints. (Washington State DOT Standard Plan for DBR, A-60.20-03)

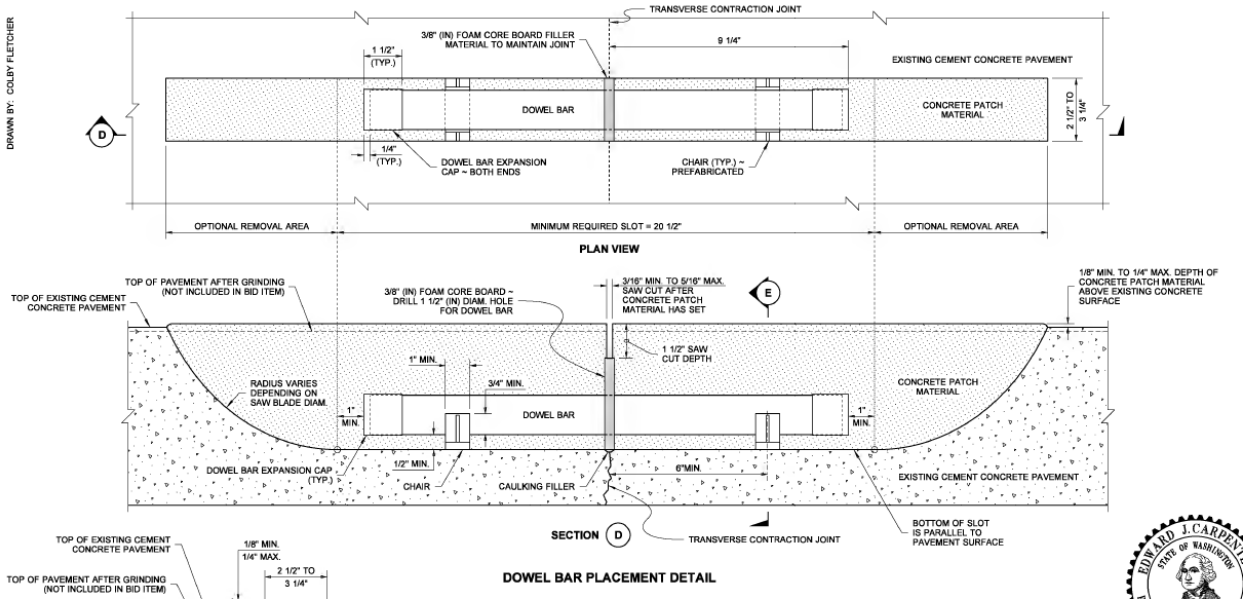

Figure 6. Washington State DOT DBR placement detail. (Washington State DOT Standard Plan for DBR, A-60.20-03)

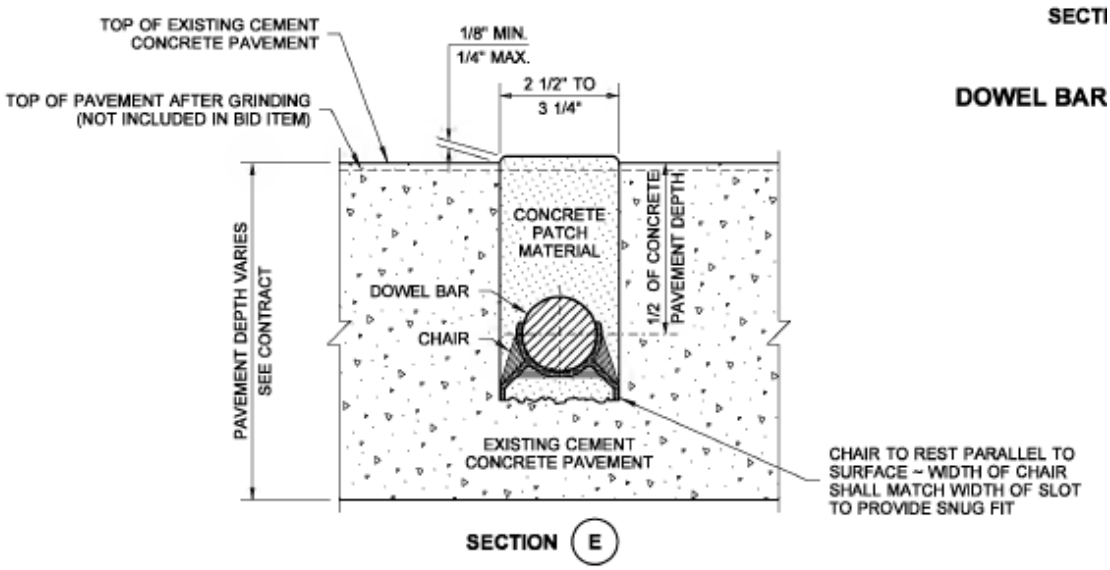

Figure 7. Washington State DOT DBR cross-section layout diagram. (Washington State DOT Standard Plan for DBR, A-60.20-03)

An experimental DBR project on an 8-inch JPCP was constructed in Iowa with a variety of dowel shapes (circular and elliptical), materials (steel and fiber reinforced polymer), and number of dowels per wheel path. Results after several years of traffic showed that all dowel cross-sectional types, dowel material types, and number of dowels per wheel path (two, three, and four) performed equally at controlling faulting. (Cable et al., 2008)

3. Dowel Bar Retrofit Specifications

Washington and the other States appear to have very effective specifications and special provisions for DBR. Contractors who have worked in these States affirm they are reasonable and effective. Table 4 summarizes the various specifications, special provisions, and other documents from these States. Each of the key topics of DBR construction is covered in this section.

Table 4. Summary of State specifications for DBR.

State	Specification/Documents
Washington	WSDOT 5-01 Cement Concrete Pavement Rehabilitation Standard Plan A-60.10-03 "Dowel Bar Retrofit: Do's and Don'ts" WSDOT (2009)
Minnesota	MnDOT 2302 SP. Concrete Pavement Rehabilitation Best Practices Manual, MN Local Road Research Board, 2006.
California	Section 40-1, SSP 41-.02, RSP P10 Dowel Bar Retrofit, Revised Standard Plan P7 SSP 41-8. Dowel Bar Retrofit SSP 41-8.03L SSP 41-8.01D(6) "Evaluating Load Transfer Restoration," Karl Smith, Raul Alarcon, July 2002, California DOT
Missouri	MoDOT Standard Specification 613.40 MoDOT Standard Drawing 613 (Sheet #4)
Utah	UDOT Standard 2754

Training of State and Contractor Staff

Training was a major concern for DBR for all of the States and contractors included in the survey because, if anything goes wrong, the DBR is likely to fail early. Washington provides "just-in-time" training. In California, training is provided by the contractor 2 weeks before DBR starts: saw cutting slots, concrete removal, cleaning & preparing slots, placing dowel bars, mixing & placing polyester concrete backfill, opening to traffic & contingencies, test section construction, and coring. Minnesota used to provide online videos (YouTube) and used to require contractors to watch. Apparently, the level of experience in Minnesota has increased enough to end this requirement a few years ago.

Training is a very important aspect for both inspectors and contractors. Inexperience and turnover have been an issue at the State level; therefore, training is strongly recommended.

Removal of PCC in DBR Area and Cleaning the Area

In Washington and other States, they use diamond saw blades to cut the dowel slots. Slots are not cut near cracks, as this leads to more cracks and spalls. Light jackhammers (<30 lb) are used to remove material to minimize breaking through the slot. Slot surfaces are cleaned by sandblasting to remove slurry caused by saw cutting, and enable bonding. Contractors emphasize that sandblasting is a key issue to get the slurry out of the slot, since it is difficult to tell if it has been really cleaned, and cleanliness is essential to achieving bond.

Caulking filler, neatly placed in the joint across the slot bottom and sides, is required to prevent patching material seeping into the joint or crack. However, caulking is often overspread, up to 2 inches, on either side of the joint or crack. Optimally, the caulking material should not extend more than ½ inch on each side; otherwise, the extra width of chalking material reduces the effective bonding area between the slot material and the slab.

A Washington contractor stated that a method has been developed to remove the slot materials without full depth saw cuts. This method reduces the amount of material used and reduces the number of workers on the job, thereby reducing cost for the same quality and performance. The current cutting of the slot itself with saw blades ends up using a lot of slot material that is not needed. Sheet plan A60.2003 is not drawn to scale, but if it were, it would show a lot of additional slot fill material is wasted. The new approach removes 20 inches of slot length using a vertical drill shaft operation that gives better bonding and requires less material.

Dowel Bar Selection

Since the life of DBR is typically 10 to 20 years, whether the bars need to be corrosion resistant varies from climatic region to region. Washington has dowel bar corrosion-resistant alternatives: stainless steel, low-carbon chromium steel, zinc-clad, and most used epoxy-coated (ASTM A934). Dowel bar spacing remains at three bars per wheel path regardless of the dowel type. Washington uses corrosion-resistant dowel bars for new construction and has used non-corrosion resistant dowel bars for DBR. However, non-corrosion resistant dowels in Washington are epoxy-coated.

The other States interviewed specify corrosion-resistant dowels for DBR. If an agency specifies corrosion-resistant coated dowels for regular concrete pavement and also for full depth repairs, then coated dowels should obviously be used for DBR. If a dowel becomes corroded it is likely to bond to the surrounding concrete, creating a locked up joint which may result in severe cracking of the DBR slot and surrounding concrete over time.

Lightly Coated Dowel and Placement of the Dowel Bar Assembly

All of the States believe it is essential that, prior to the dowel assembly being placed into the slot, the dowel be lightly coated with lubricant or parting compound. (see Pierce, Weston & Uhlmeyer, 2009) If the dowel is not coated with lubricant, it may bond to the slot cementitious material, lock up the joint, and result in cracking of the slot material as well as beyond into the slab, because the joint is going to continue to open and close.

A recent study showed that the pullout of ungreased dowels requires a significantly higher force than for greased dowels, suggesting that a lack of grease may restrain a doweled joint from opening and closing and cause joint lockup (Khazanovich, Hoegh & Snyder, 2009). A DBR experimental project, constructed in Florida on I-10 in 1988, developed significant early cracking near and between the dowel slots. The appearance of the cracks made it seem that the joint had

locked up, and this is believed to have been due to no lubricant on the dowel bars. (Hall, Darter & Armaghani, 1992) Thus, it is deemed essential that the dowels be lightly coated to prevent bonding and joint lockup.

The lubricated dowel bar assembly is placed into the slot as shown in Figure 8. The assembly is placed in the center of the slot. Other States utilize a very similar approach to that of Washington.

Figure 8. Inserting dowel bar assembly into slot. (Photo Courtesy of Jeff Uhlmeyer, WSDOT)

Joint Separation Material and Sawing of Joint

The foam insert in Washington is a ⅜-inch-thick material that is placed at the dowel center to maintain the transverse joint. The foam insert used to keep incompressibles out of transverse joints is called "foam core board." The foam insert must be capable of remaining in a vertical position and tight to all edges during the placement of the concrete patching material. Silicone caulk is used for sealing the transverse joint at the bottom and sides of the slot. The slot is overfilled a little to provide for diamond grinding of the pavement surface.

In Washington, the joint is maintained by saw cutting the surface with a hand pushed single blade saw. The cut width is 3⁄16 to 5⁄16 inch and the depth 1½ inches. The cut length is 2¼ feet long centered over the three retrofit dowel bars.

A Utah contractor stated that maintaining the proper location, especially standing it up vertically, is perhaps the most important installation item. Many times the foam falls over in the slot or

curves around and prevents the joint from forming. The “foam tabs” must be straight up and down. The other States use a similar material and requirements.

DBR Slot Material

Washington’s concrete slot material is prepackaged mortar extended with aggregate (example product: CTS non-shrink rapid set grout). See WSDOT Standard Specification Section 9-20 for strength, scaling resistance, and freeze-thaw requirements. This material has provided good performance in Washington.

The slot material is placed, consolidated, and cured until ready to open to traffic. Saw cut slots are prepared such that dowel bars can be placed at the mid depth of the concrete slab, centered over the transverse joint, and parallel to the roadway centerline and surface. Placement tolerances for dowel bars are as follows:

- ± 1 inch of the middle of the concrete slab depth.
- ± 1 inch of being centered over the transverse joint.
- ± ½ inch from parallel to centerline, ignore joint skew.
- ± ½ inch from parallel to the roadway surface.

The Washington contractor interviewed recommends that the material used in the slot needs to be as hard as the existing PCC. If not, the retrofits wear down from studded tire wear more than the existing PCC itself. Studded tire wear is excessive in Washington State. The increased studded tire wear in the eastern portion of the State is due to higher numbers of studded tires and a softer aggregate than on the western side. This factor must be considered in most CPR work. One current project shows ½ to 1¼ inch of studded tire rutting along the project. Retrofit material must have good hard aggregate (pea gravel or No. 8 stone) to match the wear down of the filler material.

California uses polyester concrete, which consists of polyester resin binder and dry aggregate. The existing slot surface is treated with high molecular weight methacrylate (HMWM) bond agent. A description of polyester concrete is provided in the partial depth repair case study prepared for Missouri DOT. Minnesota uses packaged, dry, non-shrink, rapid hardening concrete conforming to ASTM C 928 (R3) plus other tests. Missouri uses rapid set concrete patching material. Example products include Western Materials product (MO) and CTS product (CA). Utah uses prepacked, dry, non-shrink, rapid hardening concrete such as Five Star, AHT DB Retrofit Mortar.

Minimum Time to Open to Traffic

Washington’s slot material sets up well and allows opening to traffic in 2 hours. Required compressive strength in 3 hours is >3,000 psi. California opening times require at least 4 hours plus 2 additional minutes for each 1 minute that the initial set time exceeds 30 minutes; or the

contractor may, upon providing certified laboratory test results for compressive strength under California Test 551 or ASTM C109, submit a request for authorization to open to traffic in less than 4 hours depending on the time for polyester concrete to reach 1,250 psi compressive strength. Minnesota requires >3,000 psi compressive strength or manufacturer's recommendation. Missouri requires both 2 hours' time and compressive strength >1,600 psi prior to opening. Utah requires >3,000 psi compressive strength.

4. Inspection/Acceptance

Washington has a detailed inspection plan in their construction manual that includes meeting with the contractor, visual inspection of slots, sandblasted faces, dowels, foam core inserts, fill material, and equipment. First, a preconstruction meeting is held to ensure everyone understands the DBR requirements. The inspector verifies that the slots are located in accordance with the plan and cut parallel to the centerline of the roadway and to each other, and that they are centered over the transverse joint. All exposed surfaces and cracks in the slot must be sandblasted to a clean concrete surface. All grout residue and debris must be removed from the slot.

The inspector should ensure that dowel bars are as specified and are placed in accordance with the plan. Foam core inserts must be placed at the middle of the dowel, in line with the transverse joint, and must fit tightly to the sides and bottom of the slot. The foam core inserts should extend to the top of the existing pavement. It is important that the foam core inserts are placed perpendicular to the bars and line up with the perpendicular transverse joints. The top of the foam core insert will be removed when the joint is saw cut through the section top above the dowel. Concrete material is placed in the slots in a manner that does not disturb the dowel bar and to a level slightly above the level of the surrounding roadway.

Materials. The contractor must use concrete patching materials meeting the requirements of Standard Specifications Section 9-20. The inspector should inspect and document all prepackaged cementitious materials to ensure that they are properly labeled and that the contractor mixes them to the correct proportions, and follows any placement restrictions, listed on the packages.

Ensure that dowel bars and tie bars are placed in accordance with the plan and meet the requirements of Standard Specifications Section 9-07.5(1) and 9-07.6. The inspector should collect the manufacturer's Certificate of Compliance documentation (and Certificates of Materials Origin on federally funded projects) for all dowel bars and tie bars prior to use on the project. <u>As previously described, the dowel must be lightly coated with lubricant or parting compound</u>.

Equipment. The inspector should verify that all equipment used is in good working order and meets the requirements of the contract. Ensure that air compressors are of sufficient size and capacity to perform the work.

Cleaning the Slot. Contractors believe that inspection of the slot is extremely important in Washington. Sandblasting is believed to be the only way to get it clean (water blast does not appear to work as well).

Foam Core Board Inspection. If the board "rolls over" it is easy to get buried in the fill material and will ultimately result in spalling, so it is vital to keep the core board straight up to form the joint. Specs require that the board will not be higher than the retrofit slot material. However, if the board is slightly higher than the retro surface, it would provide a simple visual that the board is straight. This helps the inspector see if it is rolled over. If rolled over, then the area can be cleaned out and sealed to avoid a spalling condition. The DBR does not need to be replaced.

California uses core tests to determine alignment, placement, and polyester concrete consolidation. California requires a test section at least 1 traffic lane wide and at least 300 feet long. The contractor must drill cores for the Department's evaluation of dowel bar placement and polyester concrete consolidation (section 41-8.01D(4)). If DBR work is noncompliant, the contractor is required to:

1. Stop dowel bar retrofit activities
2. Modify equipment and procedures to demonstrate corrective action
3. Replace noncompliant dowel bars
4. Perform additional core tests to verify compliance
5. Construct another test section

Minnesota requires that, prior to major DBR operations, a DBR construction demonstration is performed that includes 24 dowels. The engineer marks three locations for 6-inch coring at center of dowel. The engineer will examine the core to see if dowel anchoring is acceptable. If dowels are located improperly or air voids exist around the dowel, replacement is required. A 30-day warranty on all repairs is specified that starts after diamond grinding is completed in the lane. An additional core is taken at a frequency of 1 per 600 bars. If there are problems, the frequency is increased.

Utah requires a similar 24-DBR test section. Cores are taken for verification of alignment, placement, and consolidation. Slot material approval is required.

Missouri inspection includes visual inspection of the slots, sandblasted faces, dowel assembly placement, foam core inserts, and equipment. Tests are specified for the slot material. Basically, the inspection is only visual. Unacceptable DBR repairs must be mitigated by a method proposed by the contractor and acceptable to the engineer. Missouri contractors agree that a preliminary test section (of a few DBR installations) is a good idea to ensure the contractor knows how to properly do DBR work.

Incentive/Disincentive

There are no incentives/disincentives used by any of the States for DBR. The Utah contractor stated that some type of warranty would be valuable. Then the subcontractor would be motivated to accomplish better quality. Minnesota has a 30-day warranty on all repairs that starts after diamond grinding is completed in the lane. Minnesota feels that if a DBR is going to deteriorate it often happens very quickly, and the 30-day warranty will very often catch this occurrence. Missouri and Utah also agreed that a short-term warranty may be a valuable idea for DBR.

5. Performance & Survival of Dowel Bar Retrofit

Washington. The Washington State OT Pavement Policy states:

> Dowel bar retrofits can be effective since WSDOT did not generally place dowels in PCC pavement up until 1993. Dowel bars placed in the wheel paths have been shown to significantly restore load transfer and hence reduce reoccurring faulting. Dowel bar retrofits can be expected to perform adequately for about 10 to 15 years. Following this, it is common WSDOT experience to observe accelerated slab deterioration. (WSDOT Pavement Policy, 2015)

Overall, DBR performance has been good with very few performance issues. If constructed as part of CPR and done earlier in the JPCP's, life the future pavement life can be extended 20 to 30 years. An example includes a Washington DBR section constructed in 1995 that has still not developed any significant faulting. DBR as part of CPR is critical to successful future non-doweled pavement life. Another DBR project on I-82, constructed in 1997, also continues to perform well.

California. DBR projects have performed well in California except for some initial projects. Backfill or slot material may crack, but that doesn't mean the dowel is not providing good LTE. As long as a bar is providing LTE and slot material has not come out then it has not failed. DBR joints can be tested using Falling Weight Deflectometer equipment for LTE, which is the ultimate DBR performance criterion (typical DBR joints have an LTE > 80 percent). Projects may require subsealing first to provide sufficient support so that DBR will provide good performance (in other words, DBR does not solve existing loss of support problems). Truck traffic has increased on all older projects, greatly making good joint support even more important to prevent erosion and loss of support. Typical performance has been 10 to 15 years.

Minnesota. DBR is applied on both non-doweled JPCP and at transverse working cracks of JRCP. DBR should last 25 years if the surrounding PCC is sound and the repair material remains durable. A few bad lots of repair material have been obtained. A Minnesota contractor estimates DBR will last 20+ years if properly constructed and applied.

Missouri. DBR has been applied mostly at working transverse cracks of JRCP. Types of deterioration includes spalling (minor), joint fault (minor), and cracking (minor). Performance has been very good to date. The oldest projects are 10 years old under heavy I-70 truck traffic and showing good performance to date; there have been no failures. A Missouri contractor says that all projects are lasting >10 years. He has observed >20 years on a few projects in other States.

Utah. Types of deterioration include spalling (minor), joint fault (minor), and cracking (minor). Performance has been excellent to date for 11 DBR projects that have been constructed since 2002. The oldest DBR project in Utah is a JPCP project on I-215 with skewed joints that is now 15 years old. The project shows no deterioration and should last 20+ years.

In summary, typical CPR projects without DBR have lasted from 8 to 15 years (depending on truck traffic, climate, base type, joint spacing, and tied shoulders) before faulting returns and diamond grinding is again required. Justification for DBR requires a design service life of at least 15 to 20 years or more. DBR service life has been consistent across all of the States included in this survey, which include a wide range of climates and pavement types and deterioration. Thus, if long-term service life is desired and the existing JPCP or JRCP has durable PCC, then DBR may be a cost-effective alternative to include in CPR.

The AASHTOWare Pavement ME Design procedure can be used to evaluate the future pavement performance (faulting, cracking, IRI) with and without DBR. DBR performance depends heavily upon proper construction. Preconstruction just-in-time training and adherence to good specifications will enable a significant service life extension for a JPCP or JRCP project.

6. Summary

The dowel bar retrofit technology has been improving for many years and has proven to be a highly successful repair technique to increase the joint load transfer efficiency of non-doweled transverse joints in JPCP and working transverse cracks in JRCP. To be successful and perform 15 to 20+ years, a DBR must meet the following requirements:

- **Sound existing slab concrete surrounding the DBR.** Otherwise, early failure will occur. Full depth panel replacement/joint repair should be placed if sound concrete is not available at the joint where the slots are located.
- **Proper sawing of the slots and removal of concrete.** Removal must not damage the lower portions of the concrete slab.
- **Cleaning (sandblasting best) of the dowel slot area** is absolutely essential, along with placement of an effective bonding agent, if specified.
- **Sealing of the underlying crack** below the DBR is required to prevent material from infiltrating the joint or crack and causing future failure.
- **Forming of the joint/crack** through the center of the dowel bar properly.

- **DBR slot material must be durable and long lasting,** not shrink significantly, and bond to the slots. Regular concrete has worked, but rapid setting and other specialty materials have also shown very good long-lasting performance in the small slots.
- **Inspection is observational** and thus must be monitored adequately. Warranties (e.g., 30 days or longer) are believed to be an effective approach to improved quality and longer lasting DBRs.
- **Service life** for DBR projects exceeds that of non-DBR projects because joint faulting does not return and require additional grinding or an overlay. Typical CPR projects with DBR have lasted from 10 to 22 years, depending on PCC condition, truck traffic, climate, base type, joint spacing, and PCC tied shoulders.

Thus, DBR has produced a significant increase in life of faulted JPCP and working transverse cracks JRCP in Washington and all of the other States interviewed when the specifications are followed, materials are durable, and inspection/acceptance procedures are followed. In these cases, a life of 10 years to 20+ years has been achieved. However, many things can and will go wrong if the State inspection staff and the contracting staff are not trained on proper procedures. This points to the importance of just-in-time training for the State and contractor personnel prior to the start of work.

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#3 Diamond Grinding Case Study for Utah and Other Leading States

Abstract

Diamond grinding is a technique applied to an existing concrete pavement to produce a smooth ride, longitudinally textured skid resistant surface, and lower pavement/tire noise level. Diamond grinding has been used extensively since the mid-1960s. Combined with other needed concrete pavement restoration (CPR) techniques, diamond grinding provides a major restorative and cost-effective preservation treatment that can significantly increase pavement service life before structural overlay or reconstruction is required (Rao et al. 1999, Darter & Biel 2016). This case study report focuses on diamond grinding in Utah but also includes information from Washington, California, Missouri, Georgia, and Minnesota.

The first diamond grinding project in Utah was performed in 1988, and more than 60 major projects have been completed since then covering over 6 million square yards or 852 lane miles of traffic lanes. Utah builds jointed plain concrete pavement (JPCP) and did not include dowels until the late 1990s. Thus, some JPCP developed joint faulting from erosion of the base courses. A recent survival analysis indicated that diamond grinding projects helped to extend the 20-year design life of JPCP in Utah by more than double (Darter & Biel 2016).

The Utah Department of Transportation has developed technology that has resulted in some effective specifications, special provisions, and standard drawings as needed for diamond grinding projects. In addition, inspection and acceptance of projects have improved over time. Utah has several positive aspects going for it to improve life of diamond grinding, including a typically hard aggregate (river gravel, granite) in the concrete slabs, durable concrete slabs, a drier climate, base courses that have bonded to the slab and not pumped excessively, and reasonable joint spacings (12 to 19 ft). Lack of dowels and a cold climate are negatives that contribute greatly to joint faulting, however, and some faulting has occurred on heavier truck trafficked highways.

Performance of diamond grinding in Utah has been good, providing a service life ranging from 10 to 20 years for undoweled JPCP. Dowel bar retrofit has been utilized on 10 projects that have not re-faulted, so this is a major factor that will improve the life of diamond grinding. The other States included in this case study report a similar life of diamond grinding, ranging from 7 to 20+ years depending on various conditions.

1. Introduction

Diamond grinding is a well-established technology that has been utilized extensively since the 1960s to provide a smooth yet textured surface for older existing concrete pavements that have been repaired adequately. Diamond grinding in combination with other repairs can reduce the existing pavement International Roughness Index (IRI) from 20 to 80 percent. Thus, an existing jointed plain concrete pavement (JPCP) or jointed reinforced concrete pavement (JRCP) with an IRI of 160 inches/mile, could be expected to have an after grind IRI of about 160*0.5 = 80 inches/mile for a 50 percent reduction. Minnesota reported that a typical IRI of diamond ground projects is 60 inches/mile. Figure 9 shows an illustrative time history of IRI before and after diamond grinding with dowel bar retrofit (DBR). Note the 52 percent reduction in IRI and that the IRI has not increased after grinding due to DBR that has prevented faulting at transverse joints.

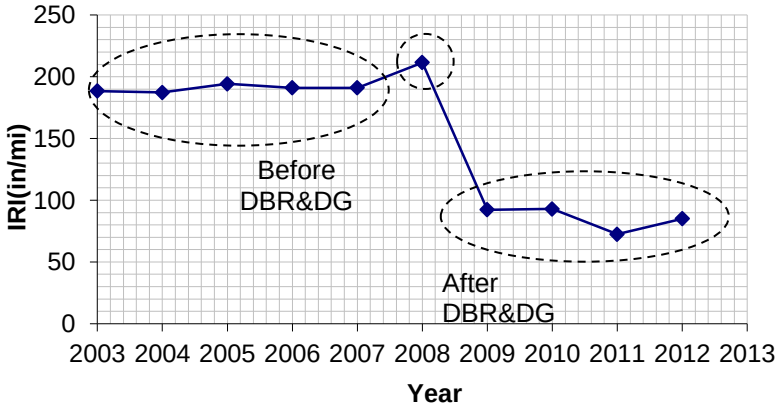

Figure 9. Texas US 69 JPCP diamond grinding (52% reduction).

Diamond grinding is considered a cost-effective preservation treatment for many existing JPCP and JRCP that can significantly increase pavement life before more costly treatments such as overlay or reconstruction are required.

This report is a case study for Utah but also includes information from Washington (JPCP), California (JPCP), Missouri (JRCP), Georgia (JPCP), and Minnesota (JPCP, JRCP). The technique was first used in 1965 on a 19 year old JPCP on I-10 in Southern California to eliminate transverse joint faulting. This pavement was again ground in 1984, 1997, 2005, and 2017 still carrying heavy traffic 70 years after it was first constructed. Diamond grinding became a widely used technique in other States in the 1970s, particularly in Georgia, Washington, and Minnesota. The first diamond grinding project in Utah was performed in 1988, and more than 60 major projects have been completed since then. Utah builds JPCP that did not include dowels until the late 1990s. Many of these JPCP developed joint faulting from erosion of the concrete treated base (CTB) or lean concrete base (LCB) courses.

The Utah Department of Transportation (DOT) and other State DOTs have developed improved technology over the past few decades that has resulted in today's highly effective specifications, special provisions, and standard drawings. In addition, inspection and acceptance of projects have improved over time. Information for this document was obtained from both State and contractor/industry interviews of those considered experts in diamond grinding and concrete pavement restoration (CPR) as well as current published literature. Current information indicates that diamond grinding has been very effective in both smoothing rough older pavements and providing good frictional surfaces and noise reduction. Pavement performance data show that diamond grinding (as part of an overall CPR program) has extended the life of JPCP until overlay or reconstruction in Utah out to 40 to 50 years, which is more than double the original 20-year design life of the JPCP. A significant life extension of JPCP and JRCP was also found in the other States surveyed.

This document presents key information from the interviews for pre-construction considerations, diamond grinding specifications, acceptance and inspection, and performance and survival results for diamond grinding. A Tech Brief was also prepared that provides concise recommendations and guidelines for diamond grinding.

2. Pre-Diamond Grinding Considerations

Utah and other States typically perform diamond grinding primarily to remove joint faulting and to restore pavement texture (and friction) and smoothness. Another reason is to address studded tire wear in States like Washington, where over 1 inch of rutting wear out can develop on concrete surfaces caused by studded tires. Reduction of pavement/tire noise level is another reason to diamond grind an existing pavement to remove harsh cross tining or other texture issues.

Utah conducts an initial scoping of the project 2 to 3 years in advance of expected construction. A final scoping occurs about 2 months before construction to get final quantities for bidding purposes. Utah and other States typically take into account the following key pre-diamond grinding considerations:

- Consideration of the pavement age, traffic, design, and past rehabilitation history.
- Assessment of the existing pavement condition in terms of distress type (joint faulting, rocking panels, slab cracking, settlements), severity, and extent, as well as the existing IRI along the project, and lane by lane.
- Depth of sound concrete at transverse joints.
- Desired service life of the restored pavement.
- Noise level (pavement/tire).
- Blade spacing considerations.

These considerations provide answers to questions like:

- Is this a suitable candidate for CPR?
- What are the major types of deterioration that requires direct consideration in the CPR construction project?
- Are there any serious problems along the project that will require special treatment for diamond grinding, or areas where diamond grinding should not be performed?
- Is this diamond grinding being done to provide a short life extension (e.g., <10 years) or to achieve a longer service life?

Answers to these questions then lead to the development of the design plans and specifications.

Pavement Age, Traffic, Design, and Past Rehabilitation

The younger the pavement (and less deterioration), the longer the CPR and diamond grinding can be expected to last. The heavier the traffic, the shorter the diamond grinding can be expected to last. And of course, the existing pavement design is critical:

- JPCP: Critical design features are dowels at transverse joints, joint spacing, slab thickness and underlying deterioration, and base type.
- JRCP: Critical design features are joint spacing, reinforcement adequacy (are the transverse cracks deteriorating?), slab thickness and underlying deterioration, and base type.

Past restoration and rehabilitation activity provides clues about the progression of deterioration. Diamond grinding projects have been repeated at least twice in all of the surveyed States and three to four times in some States. Utah increases slab thickness slightly to account for future multiple grindings. Service lives of these multiple CPRs may decrease over time because more deterioration exists and traffic is heavier, but they have been acceptable to the agencies involved. None of the States noted any durability issues caused by diamond grinding itself.

Pavement Condition

Utah assesses the existing pavement condition in terms of distress types, including the following:

- **Joint faulting** (increased faulting requires greater concrete removal and is indicative of greater amounts of faulting to come after diamond grinding). Utah uses the AASHTOWare Pavement ME Design software to assess future faulting of the diamond ground JPCP with and without dowels and the diameter of dowels.
- **Rocking panels** indicate a serious structural problem exists in the base/subgrade.
- **Slab cracking** (increased slab cracking may be indicating fatigue cracking from heavy traffic, or from some previous design or construction deficiency). Utah uses the AASHTOWare Pavement ME Design software to assess future transverse fatigue cracking of the diamond ground JPCP with reduction of a certain amount of slab

thickness. California specifies that > 15 percent slabs cracked may not produce a good performing CPR/diamond grinding project.

- **Joint spalling** indicates partial depth and full depth repairs that must be placed prior to grinding.
- **Settlements** or heaves along the project may need attention prior to diamond grinding.
- **The existing IRI** along the project, lane by lane, is measured. The higher the IRI, the greater the difficulty to grind the project, and the "acceptable" IRI for acceptance may need adjustment.
- **Friction** is specified by California as another criterion; a coefficient that is less than 0.30 is considered suitable for diamond grinding retexturing.
- **Underlying concrete deterioration** at joints is a serious problem in Minnesota. Durability of the existing portland cement concrete (PCC) is critical. Some joints do not appear to have underlying deterioration but actually do, and if not repaired this deterioration will shorten the life of the diamond ground project. Cores should be taken at joints to determine the extent of deterioration.

Service Life of Restored Pavement

Most CPR and diamond grinding is done to provide for a long service life—10 to 20+ years. There are occasions, however, when a life of 5 years may be desired. In this case, the diamond grinding may not need to achieve as low an IRI after grinding as if the pavement is required to serve for a much longer time period.

Noise Level of Restored Pavement

Conventional diamond grinding significantly reduces the noise level of the existing pavement. When the project is located where low tire/pavement noise is very important, use of the Next Generation Concrete Surface (NGCS) can provide the quietest texture available for non-porous concrete pavements. The texture can be obtained using conventional diamond grinding equipment and blades but in a somewhat different configuration than traditionally used. Of the States surveyed, Minnesota and California have utilized the new generation texture for grinding where noise is of concern since 2007. This has proven to result in lower tire/pavement noise with good performance. Another application for either conventional diamond grinding or NGCS is when there exists a harsh transverse tining finish that creates too much noise. Elimination of existing cross tining can significantly reduce tire/pavement traffic noise. It uses conventional diamond grinding equipment and blades but in a somewhat different configuration than traditionally used. (Schofield 2016)

Blade Spacing and Texture Required

In all of the States included in this study, it is the contractor's responsibility to select the number of blades per foot to be used to provide the proper surface finish for the aggregate type and

concrete present on the project. Recommended blade spacing to achieve optimum texturing was strongly related to concrete coarse aggregate hardness with softer aggregate requiring a wider blade spacing (or increased land area). Table 5 summarizes the texturing requirements of the States included in this study. Most States specifications particularly emphasize the difference required in blade spacing between soft and hard aggregates.

3. Diamond Grinding Specifications

All of the States included in this study have up-to-date and highly effective specifications for diamond grinding. All of these States have spent many years refining and improving their specifications. California and Georgia began experimenting and refining diamond grinding in the early 1970s, and others came shortly after that. Table 6 summarizes the specifications, special provisions, and other documents from these States.

A "Standard Specification for Diamond Grinding for Pavement Preservation" is also under development at the present time. This document may be approved later in 2017 or early 2018 (AASHTO 2017).

Table 5. State diamond grinding texturing requirements.

State	Groove Width	Land Area, Between Groves*	Grove Depth	Comments
Utah	0.09 to 0.15 in	0.06-0.13 in	1/16 in (0.06 in)	Grinding texture has not been an issue due to hard aggregate.
California	0.08 to 0.12 in 55 to 60 grooves per foot of width		0.06 to 0.08 in from the top of the ridge to bottom of groove.	
Georgia	Select the number of grooves per foot to produce the surface finish for each aggregate type that is in the concrete surface on the project (wide range of aggregate hardness exists.	Want the minimum blades that will allow land area to break, so as to maximize land area of the grind.	1/16 in (0.06) +/- 1/32 in (0.03) measured from peak of groove to bottom of the groove.	Select the number of grooves per foot to produce the surface finish for each aggregate type that is in the concrete surface.
Minnesota		Limestone: 0.09 to 0.11 in Granite/gravel/ quartzite: 0.08 to 0.095 in	1/8 in (0.125) +/- 1/16 (0.06 in) when measured from peak of groove to bottom of the groove	
Missouri	0.22-0.24 in. Blade spacing increased due to		1/32 in (0.03 in)	Contractor: Soft aggregate can be a problem. Adjust blade

State	Groove Width	Land Area, Between Groves*	Grove Depth	Comments
	softer nature of predominate limestone aggregate, otherwise fins break down more quickly.			spacing accordingly. 0.124 in spacers for soft aggregates and 0.105 in spacers for hard aggregates.
Washington	3⁄32 and 5⁄32 inches wide	1/16 to 1/8 in	<1/16 in (0.06 in)	Aggregate hardness is major factor. East side (Spokane) has soft aggregates and more studded tire wear occurs. West side has hard aggregates with less wear occurs. Careful selection of blade spacing required.

*Note: For all of these States, it is the contractor's responsibility to select the number of blades per foot to be used to provide the proper surface finish for the aggregate type and concrete present on the project.

Table 6. Summary of State specifications for diamond grinding.

State	Specification	Comments
California	Section 42-3: Grinding Specifications for grinding concrete pavement surfaces. 40-1.01D Smoothness requirements. (2015)	
Georgia	GA 431 Grind Concrete Pavement (2014)	
Minnesota	MnDOT SP 2302 S-142 (SP 2302) CONCRETE GRINDING MnDOT Pavement Surface Smooth-ness 2399 (Includes incentives/disincentives. (2016)	Special Provision 1717: Air, Land, and Water Pollution for Concrete Grinding
Missouri	MoDOT Sec. 622.30 Diamond Grinding of Existing PCC Pavement. (2016)	
Utah	Grinding Pavement 02981. Special Provision 02742S. Pavement Smoothness 02701 (2017)	
Washington	WSDOT 5-01 Cement Concrete Pavement Rehabilitation (2016)	

The States and other sources make various recommendations regarding diamond grinding of concrete surfaces that should be considered in specifications or guidelines. These are summarized here:

- **Roughness measurement.** Diamond grinding equipment has become much more reliable. The profile data help to ensure that the correct amount of grinding takes place and minimizes the low or high spots along the pavement. To meet smoothness specifications, the State and contractor need to perform the roughness tests during the same time period. The measured profile index or IRI changes throughout the day due to thermal gradients through the slab that result in curling, which affects profile measurements. The time between grinding and profile measurement must be short to avoid impact of additional joint faulting, especially if project includes no dowels or DBR.
- **Equipment checks** for the state and contractor equipment are performed yearly to ensure they measure the same values. State and contractors also recommend that a national certification process should be performed. Currently, each State has their own methods and testing. Reciprocity is needed among all States.
- **Widening projects.** California and Washington specify that, before new JPCP or continuously reinforced concrete pavement (CRCP) lane replacement or widening projects, the entire adjacent lane width should be ground (California recommends if IRI > 90 inches/mile) to establish a smooth profile for concrete paving equipment to work against.
- **Diamond grinding of projects patched with viscoelastic type materials.** The past decade has brought about new partial depth patching materials generically called hot applied synthetic resin compounds (HASRC). These materials are much softer than the cementitious materials used for decades to repair concrete pavements. This material is a polymer-modified resin-based material that is hot applied and is flexible. Diamond grinding of some projects where HASRC materials have been placed has resulted in severe problems during diamond grinding operations. (Frentress 2010, Darter & Rao 2014). One manufacturer, Crafco, Inc., issued specific procedures in 2011, 2012, and 2015 for TechCrete patching procedures when diamond grinding would be performed.
    - "Flexible patch materials such as TechCrete, especially when placed in larger patches, can experience roughness issues because the material can form gummy ridges during the placement process which may create uneven surfaces. In worst case scenarios, the grinder can deflect into the TechCrete patch area due to its heavy weight creating scallops and roughness issues well outside of typical specifications. Additionally, in long patches paced longitudinally on a pavement surface, polymer strings may be created during the grinding process which will wrap around the grinder's cutting head."
    - The recommended procedures would increase the stiffness of the material (20 to 30 percent aggregate added to the resin) to stiffen up the mix. (see Crafco, Inc. Installation Instructions 2011, 2012, 2015). These special procedures may help the situation, but there may still be difficulties especially when repairs are greater than 2x2 ft and there are many of these in close proximity.

- One recommendation is to complete diamond grinding prior to placement of the HASRC patches if they are not already in place.
- Utah has placed many of these partial depth repairs and has not had the diamond grinding problems mentioned. Utah requires that the patch area is cleaned and primed first. Then about ¾ of the resin is placed first in the hole and bulk aggregate is added on top and worked down into and mixed with the resin with the trowel including the rest of the resin. It is easier to mix it that way than to add the aggregate first and then the resin on top. Having a patch deeper than 1 inch without adding aggregate is not allowed. Repairs that are 1 inch depth or greater require the addition of the bulking aggregate into the repair area, up to 25 percent by volume. The bulking stone is single size ¾ inch dried granite, crushed and double washed, and free of any dust, placed up to the top surface. The bulking aggregates are heated and dried in a vented barrel mixer at 300 °F. The bulking stone is placed evenly throughout the patch to within ¼ inch of the surface. Utah also requires a specified topping stone on the surface.
- Utah concludes that if you do not have any aggregate in the patch, and it is large, it will cause issues with the diamond grinding head. If the repair is finished flush, and has the bulking aggregate and topping stone, then they have not had any issues with the grinding these repairs. (See Crafco, Inc. Instructions 2011, 2012, and 2015).

- **Significant studded tire wear.** A few States have significant studded tire damage on their concrete pavements that create "ruts" in the wheel paths. Washington has significant studded tire wear in the wheel paths that over time trigger diamond grinding to minimize hydroplaning and splash. Diamond grinding projects, particularly in the eastern side of the mountains, have developed 1 to 2 inches of wear down from studded tires. There are multiple grinds on some projects, such as in the Spokane area. It is common to grind 1.5 inch from the surface (due to surface ruts or channels), which requires multiple passes. Nevertheless, there is approximately a 15-year life for these pavements.

4. Inspection/Acceptance

The inspection and acceptance process for diamond grinding focuses on the resulting wheel path profiles (both short and longer traffic lane lengths) as well as a uniform texture with proper depth and spacing of grooves across the lane. These criteria are quite effective to control the construction quality of diamond grinding, and they are practical to measure and fairly repeatable.

Utah DOT specifications include Pavement Smoothness 02701, Grinding Pavement 02981, and Special Provision 02742S that have been updated in 2017 along with new smoothness incentives. Key elements are summarized as follows:

- The ground surface is a corrugated type texture consisting of longitudinal grooves between 0.090 and 0.150 inches wide. The distance between the grooves is between 0.060 and 0.13 inches (which depends upon hardness of the coarse aggregate type). The peaks of the ridges are approximately 1/16 inch higher than the bottom of the grooves. The blade spacing must be provided so that the concrete fins break off.
- Provide a uniform transverse slope of the pavement with no depressions or misalignment of slope greater than ¼ inch in 10 feet when tested with a 10-ft straightedge.
- Smoothness is accepted by the Mean Roughness Index (MRI). MRI is the average of two wheel path IRIs taken from each pavement section.
- Localized lane length roughness limits for IRI are also included. Profile deviations in a continuous 25-ft lane pavement section are identified using the Profile Viewer and Analysis (ProVAL) "Smoothness Assurance" analysis, calculating IRI with a continuous short interval of 25 feet, and the 250-mm filter applied.
- An Inertial Profiler (AASHTO M 328) is used to measure IRI in the wheel paths. Each wheel path is measured once. Note: Some advocate that two or even three repetitions of profile measurements should be conducted to reduce variability and ensure accuracy, especially when incentives/disincentives will be calculated from the results.
- 95 percent coverage of the traffic lane is required.
- Incentives/disincentives are applied based on the wheel path results using the MRI results.

Pavement Section

A pavement section measurable with IRI is defined as a travel lane or median, 0.1 mile long. Sections include the following in Utah: All traffic lanes, ramps, medians 8 ft and wider, turn lanes, and approach slabs with final riding surfaces placed as part of the contract. Each pavement section is laid out consecutively from the start of the project. The localized roughness criterion also applies to bike lanes and shoulders.

Utah DOT Acceptance Procedures

Utah evaluates longitudinal deviations for all roadways using acceptance profiles performed by the contractor. The existing pavement surface profile is measured before beginning construction to determine MRIo (defined below). The final acceptance pavement surface profile is measured after all corrective work has been completed, including the diamond grinding to determine MRI_f (defined below).

1. Determine IRI using the most recent version of the ProVAL software. Refer to AASHTO R 54 and ASTM E 2560. Identify areas of localized roughness using the ProVAL "Smoothness Assurance" analysis, calculating IRI with a continuous short interval of 25 feet and the 250-mm filter applied.
2. Determine MRI for each 0.1-mile pavement section with the 250-mm filter applied.

Limit transverse pavement deviations to less than 3/16 inch from the lower edge of a 10-foot straightedge.

Local Deviations

1. Include profile deviations from bridge decks, approach slabs and transitions, manholes, valves, and other facilities in the profile when the contract requires the adjustment, new construction, or reconstruction of these facilities.
2. Exclude profile deviations from bridge decks, approach slabs and transitions, manholes, valves, and other facilities in the profile when the contract does not include adjustment, new construction, or reconstruction of these facilities.
3. Limit profile deviations in shoulder or bike lane as specified in Section 02742S.

Acceptance Testing

1. Collect longitudinal profiles in each wheel path and in the center of each paved shoulder and bike lane using Department-certified profilers and operators in accordance with AASHTO R 54, R 57, and M 328. Collect profiles with no filters applied.
2. Determine the MRI for each 0.1-mile pavement section in every lane to be diamond ground.
3. Determine "localized roughness" using IRI using for each 25-ft pavement section in every lane to be diamond ground.
4. Submit a summary report within 2 working days that includes pavement section identification, profile results, and bump locations showing localized roughness corrections by section.

Incentive/Disincentive

A number of States use incentives/disincentives for grinding smoothness. Utah, Missouri, and Minnesota all include incentives, and California is investigating their use. The incentive has to be enough to make it worth it to the contractor to increase their effort to achieve a smoother surface. The States and contractors surveyed believed that the "best" contractors will know that they can hit the requirements for the incentive, which they then account for in the bid price. The International Grooving and Grinding Association (IGGA) has incentive and disincentive guidelines which can be examined.

In terms of benefits, it is believed that what was discovered under National Cooperative Highway Research Program (NCHRP) Project 1-31—that for both concrete and asphalt pavements as well as overlays, smoother pavements last longer—also holds for diamond grinding (Smith et al. 1997). The contractors interviewed all recommend incentives to get the best quality of CPR projects. They also recommend that IRI (or MRI) be used as the measuring criterion on all projects.

The "percent improvement" per 0.1-mile pavement section of existing pavement MRI_o over the final after grinding MRI_f is used to calculate the Utah incentive/disincentive. Utah DOT applies incentive/disincentive for diamond grinding according to Section 02742S and calculates the percent improvement using the following equation:

$$\text{Percent Improvement} = [\ (MRI_o - MRI_f) * 100\ ] / MRI_o$$

Where: MRI_o = MRI of original roadway surface, inches/mile

MRI_f = MRI of final corrected roadway surface, inches/mile

For example, a concrete pavement 0.1-mile section having an MRI_o = 170 inches/mile prior to restoration activities that was ground to an MRI_f = 60 inches/mile would show a 65 percent reduction. Table 7 provides the incentives/disincentives for several ranges of MRI percent improvement for each 0.1-mile measurement pavement section. This section with 65 percent reduction would receive the highest incentive.

Table 7. Percent improvement incentives/disincentives for 0.1 mile pavement section, Utah DOT.

Pavements (Diamond Grinding)	
% MRI Improvement per Pavement Section	**Incentive/Disincentive $/0.1-mile Pavement Section**
≥60	$500*
50 to 59.9	$250
40 to 49.9	$125
30 to 39.9	0
20 to 29.9	-$250
<20.0	Corrective Action

*This incentive is $0.71 per SY to achieve a very smooth ground surface.

Localized roughness is also limited as specified in Utah DOT Section 02742S. Table 8 provides the limiting criteria for "localized roughness limits" that use IRI directly in the outer wheel path of the lane (located at 2.5 feet inside the traffic lane). This is for sections where the IRI is calculated over 25 feet along a traffic lane. If a section exceeds these limits, the contractor must regrind and improve the smoothness below these limits.

Table 8. Localized roughness limits for 25-ft section.

Localized Roughness Limits	
Roadway	**Maximum Allowed IRI w/base length of 25 ft (inches/mile, outer wheel path)**
Interstate including ramps	IRI ≤ 190
Bridge decks, approach slabs & transitions, manholes and valves	IRI ≤ 250
Non-interstate	IRI ≤ 190
Urban roadways with speed limits less than 45 mph	IRI ≤ 190
Shoulders and bike lanes	IRI ≤ 250 (single profile)

Utah DOT does not apply incentives/disincentives to pavement sections shorter than 1,000 feet, shoulders, bike lanes, medians narrower than 8 feet, horizontal curves with a centerline curvature radius less than 900 feet, and areas within the super elevation transitions to these short radius curves, tapers, surfaces within 15 feet of bridge decks and approach slabs are not constructed as part of the contract.

In summary, Utah DOT proceeds through the following basic steps on a diamond grinding project:

1. Assess the suitability of a project for diamond grinding for the selected design life: age, truck traffic level, transverse joint faulting, surface texture, friction, slab cracking, rocking panels, unusual profile deviations (settlements), PCC durability (spalling), signs of pumping and erosion.
2. Collect longitudinal profiles in each wheel path and in the center of each paved shoulder and bike lane using Utah DOT-certified profilers and operators in accordance with AASHTO R 54, R 57, and M 328. Collect profiles with no filters applied.
3. Determine the MRI for each 0.1-mile pavement section and calculate incentives/disincentives.
4. Determine localized roughness using 25-ft section calculated IRI and check to see if criteria are met. Smoothness improvement is required if criteria are not met.
5. Ensure cross-slope meets requirements.
6. Ensure that at least 95 percent of surface has been ground.
7. Submit a summary report within 2 working days that includes pavement section identification, profile results, and bump locations showing localized roughness corrections by section.
8. Calculate incentives using the MRI improvement for all sections along the project.

Other State Inspection and Acceptance for Diamond Grinding

California: Ground concrete pavement must meet the following acceptance requirements:

- A coefficient of friction equal to or greater than 0.30 must be achieved.
- Pavement smoothness as indicated by a mean IRI of 60 inches/mile or less within a 0.1-mile section must be achieved or further grinding is required.
- No area of localized roughness with an IRI greater than 120 inches/mile within a 25-ft section is allowed.

No incentives are used currently; however, incentives/disincentives for smoothness specifications pay adjustment factors are currently under development.

Georgia: A Profile Index (half-car measured in both wheel paths) is used for measurement. The ground surface must meet a pavement ride index value not exceeding 900 on each 0.25-mile segment for each vehicle traffic lane. Profilograph testing of ground surfaces may be required according to GDT 78 to isolate locations with individual bumps or depressions greater than 0.20 inch outside the blanking band. Transverse slope must be uniform and depressions not greater than 1/8 inch in 12 feet.

Minnesota: MRI per 0.1-mile segment is the average of the IRI from both wheel paths. Minnesota recently increased incentives to get better results and found that smoothness works well as an incentive. Acceptance includes incentives on the MRI for 0.1-mile sections.

- < 40 inches/mile (higher incentive)
- >= 40-60 inches/mile (lower incentive)
- > 60-70 (No incentive or disincentive)
- >70 inches/mile (correct until =< 60 inches/mile)

For "areas of localized roughness" (ALR) the following incentives/disincentives are paid for 25-ft segments based on MRI (speed >45 mph):

- < 175 inches/mile (Acceptable)
- > = 175 < 250 inches/mile (Corrective or $25)
- > = 250 (Corrective Work)

In addition, a 10-ft straightedge measurement may be applied for deviations > ¼ inch.

Missouri: Smoothness, as measured by IRI, is the key acceptance factor for diamond grinding. Prior to performing any grinding work, but after completion of all pavement repairs, the contractor provides a control IRI per pavement (0.1 mile) segment from the right wheel path of each lane being diamond ground in accordance with TM-59. This control IRI will be used to identify the required smoothness for the project. Each segment of the finished ground surface is re-profiled in the right wheel path and a final IRI per segment of 65 percent of the control IRI, or

80 inches per mile, whichever is greater is determined. After the initial diamond grinding operation has been profiled, additional correction is performed, where determined necessary by the engineer, to reduce the average segment profile to the specified final profile requirements. The contract unit price for diamond grinding will be adjusted based on the final IRI for any segment before corrections, according to Table 9.

One interesting comment from Missouri is that they may consider declaring CPR failures 'unacceptable' and then let the contractor propose mitigation methods.

Washington: Fins must break off properly after grinding operation. Smoothness is a key factor in acceptance with profile index. Calculation of the measured profile index excludes dips and depressions in the existing surface. Smoothness perpendicular to the centerline is measured with a 10-foot straightedge within the lanes. There can be no vertical elevation differences of more than a ¼ inch between lanes. No incentives or warrantees for diamond grinding in Washington. Ninety-five percent of the surface area of the pavement to be ground shall have a minimum of ⅛ inch removed by grinding. Eastern Washington has significant wheel path wear down from studded tires. During grinding, it is important to ensure that wheel path rutting is removed sufficiently so that the grinding operation is simply not texturing the wheel path depressions but providing acceptable lateral drainage.

Table 9. Missouri DOT Incentives/Disincentives for Diamond Grinding.

IRI, inches/mile	Increase in Contract Unit Price
40.0 or less	$0.25 / Sq. Yard
40.1 to 54.0	$0.15 / Sq. Yard
54.1 to 80.0	None
80.1 or greater	None*

*After correction to either equal to or less than 65 percent of the control IRI or 80.0 inches per mile.

Slurry Handling and Removal

Utah DOT requires that all residue from the grinding process becomes property and responsibility of the Contractor. Vacuuming equipment is required to remove residue and excess water. Other States have similar requirements. Contractors reported that some States specs are not clear on determining what is required for the grinding effluent and the cost between alternatives can be large, so this needs to be made clear. Alternatives include: shoulder disposal (most cost effective), settlement basins (pond liner used to contain slurry), Brandt system (uses a shaker, a centrifuge, and a vertical clarifier), and the filter press (squeezes the water out to separate the fines). The IGGA has a best practices document that is recommended for

consideration. (IGGA, 2013). Of course, slurry disposal be conducted in accordance with local and national regulations.

5. Performance & Survival of Diamond Grinding

Sixty major diamond grinding projects have been completed in Utah covering over 900 lane miles. The design of these JPCP did not include dowels until the late 1990s. Many of these JPCP developed joint faulting from erosion of the CTB or LCB courses.

The first diamond grinding project in Utah was performed in 1988 on a non-doweled 9-inch JPCP over LCB south of Salt Lake City built in 1972. This grinding project lasted until 2005, when the pavement was cracked and seated and overlaid. Another early diamond grinding project in 1994 was a non-doweled 9-inch JPCP over LCB on I-15 urban freeway built in 1971. A summary of diamond grinding work follows:

- 1971 JPCP/LCB project construction (no dowel bars).
- 1994 diamond grinding performed to remove joint faulting at 23 years age.
- 2005 diamond grinding performed again after 11 years to remove joint faulting.
- 2016 reconstruction of project. The second grinding (also with no dowel bar retrofit) lasted 11 more years.

In summary, the service life of this JPCP was extended 25 years beyond its original 20-year design life with two diamond grindings. Had dowel bar retrofit been installed in this project at the first grinding, it may have lasted the full 22 years until reconstruction (there was of course other distress, including joint spalling and slab cracking, that required repairs over this time period).

Figure 10 shows the results of a survival analysis of 60 diamond ground largely non-doweled JPCP (about 10 had received DBR). At 10 to 14 years, 60 percent of these diamond ground JPCP had received no CPR, overlays, or reconstruction, but from 15 to 19 years nearly all had received CPR, overlays, or reconstruction. Thus, the 50th percentile CPR survival age is approximately 15 years for these non-doweled JPCP. The faulting of these DBR projects is significantly lower, and they are expected to have a much longer life than those without dowel bars because faulting has not returned.

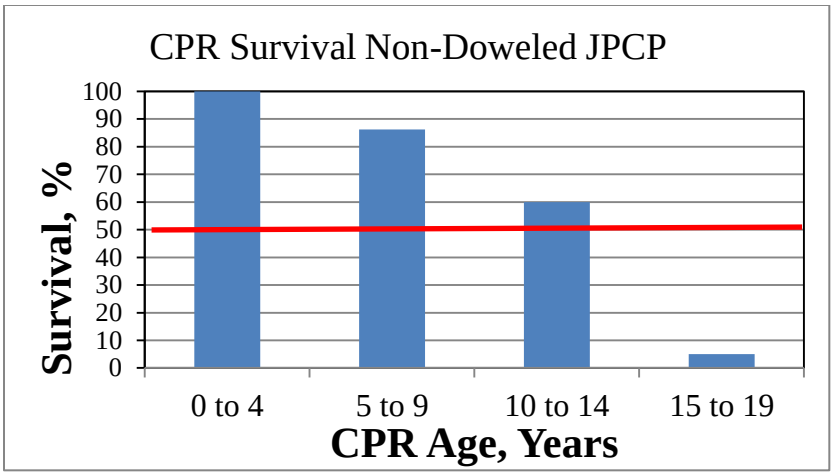

Figure 10. Survival analysis of diamond grinding of Utah non-doweled JPCP.

What about life extension of these non-doweled JPCP beyond their 20-year design life? Analysis results of all 108 JPCP projects in Utah are shown in Figure 11. Two sets survival results are illustrated:

- The blue bars on the left indicate the percent survival of the 108 JPCP projects until either an overlay or reconstruction occurred, during which time CPR was performed as part of a preservation program. About 50 percent of these projects have survived a total of 40 to 51 years of service that typically includes one or more CPR applications as needed, before requiring either a more costly overlay or reconstruction.
- The orange bars on the right indicate the percent survival of the 108 JPCP projects until either an overlay or reconstruction occurred, during which time no CPR (diamond grinding) was performed. Approximately 50 percent of these projects survived between 10 to 19 years of service until an overlay or reconstruction was performed.

These results indicate that the application of CPR with diamond grinding preservation program extended the life of JPCP significantly until a much more costly application of overlay or reconstruction was needed.

These results show that the service life of non-doweled JPCP (until an overlay or reconstruction) can be more than doubled by applying CPR including diamond grinding as needed. Diamond grinding has thus been an effective preservation treatment in Utah. Note also that, in terms of meeting the 20-year design life of these JPCP, the timely application of CPR with diamond grinding made it possible for over 95 percent of these projects to achieve the 20-year design life.

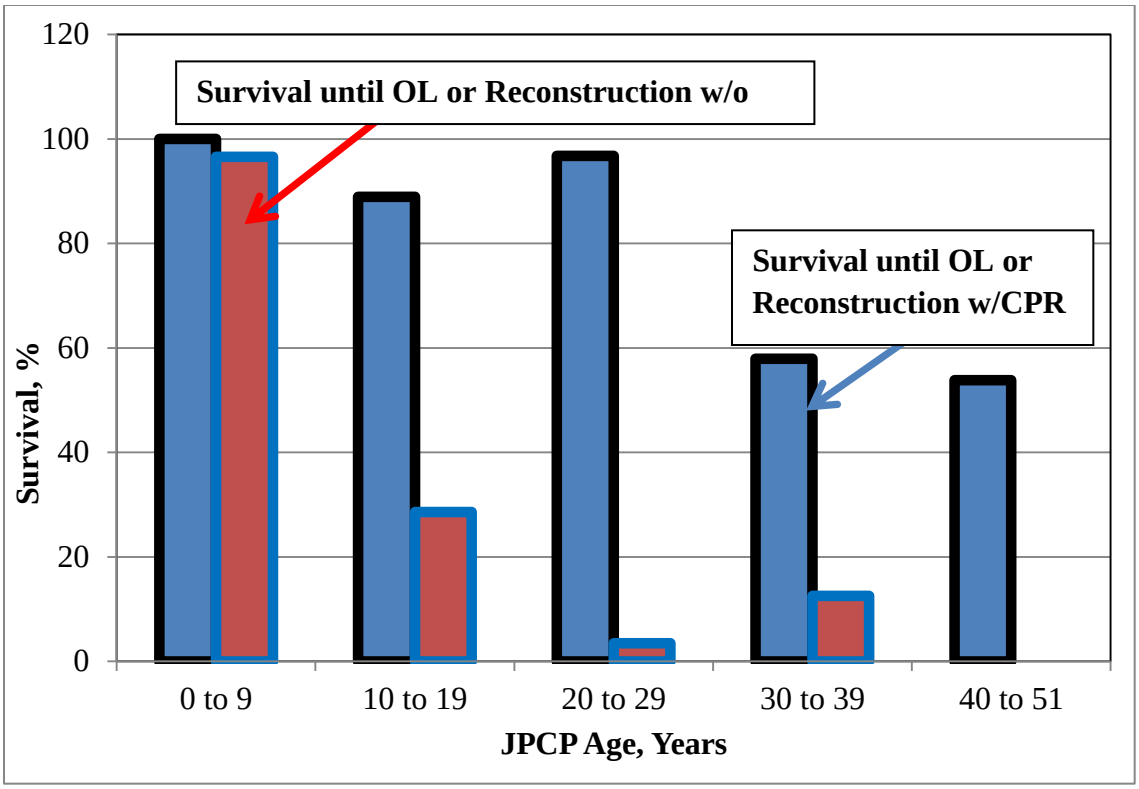

Figure 11. Comparison of service life of non-doweled JPCP with and without CPR in Utah.

Performance of diamond grinding in other States has been similar but highly dependent on factors like the hardness of the coarse aggregate and the extent of studded tires where these exist.

- **California:** Grinding can maintain smoothness for up to 15 years of service life. This estimate is based largely on no dowel bar retrofit. Typically, harder aggregates exist in California and not much polishing develops. The northern part of State has harder aggregates than the southern part. One JPCP in southern California on the San Bernardino Freeway just east of Los Angles was constructed in 1946 as part of Route 66 and later became I-10 was 19 years old when first ground in 1965 due to joint faulting (it was noted that it was structurally sound). Then this 8 inch JPCP/CTB was ground again due to faulting in 1984, 1997, 2005, and 2017, carrying traffic for 70 years. (Stubstad, 2007)
- **Georgia:** CPR projects built in the last 30 years indicate a service life of at least 20 years, depending on pavement design and materials durability. The level of truck traffic is also a factor. Coarse aggregate type (limestone versus granite) also has a major effect on texture wear.
- **Minnesota:** There has been excellent performance with DBR (prevents faulting). Grooves don't seem to wear out. The service life of the diamond ground pavement is long but depends on the adequacy of other repairs. If no DBR has been performed, then it can be expected that the pavement will last 10 years before faulting; with DBR, it is

about 20+ years. There are no signs of fatigue cracking on existing diamond ground pavements.

- **Missouri:** Blade spacing has been increased due to softer limestone aggregate; otherwise, fins break down more quickly. If no DBR is included, joint faulting occurs. Slab cracking is limited after diamond grinding. Overall, Missouri has seen very good performance, with diamond ground pavement life extended by about 7 to 9 years. With multiple grinds, PCC thickness reduction may be an issue.
- **Washington:** Grinding texture is not an issue due to the hard aggregate on the western Pacific side. However, it is a major issue on the eastern side of the State due to softness of the aggregate. Joint faulting does not reoccur if DBR is included. Slab cracking is limited after diamond grinding. Joint spalling is also limited. Texture life is in the order of 15 years in the Spokane area before the texture wears down due to studded tire damage. On the west side of the State, diamond grinding lasts 25 to 30 years. There are multiple grinds on some projects.
- **Utah:** The survival analysis indicated a mean life of CPR with diamond grinding of 15 years. About 85 percent of these sections did not have any dowel bar retrofits.

A national CPR (including diamond grinding) survival analysis was conducted in 1999 from a number of States that showed that the 50th percentile life was 13 years. (Rao et al. 1999) In addition, some projects in these States have been subjected to CPR with diamond grinding several times over a span of 40+ years prior to overlay or reconstruction. A large majority of these JPCP did not include dowel bars, thus limiting their future service life.

6. Next Generation Concrete Surface Application

This document has focused on the traditional diamond grinding texture that has been used all over the world for many years. This texture results in a reduction of roughness and a reduction in noise level plus improvement in texture and friction. Need for a greater reduction in noise level in the early 2000s resulted in development of the NGCS diamond grinding texture. (Schofield 2016)

Minnesota, for example, has placed the NGCS surface on several projects where noise was an issue. This has proven to be very quiet with good performance. The NGCS texture has resulted in a lower tire/pavement noise than conventional diamond grinding. Thus, for situations where noise abatement is of great importance, the NGCS can be used. Fourteen States have constructed NGCS on projects with positive results in lowering noise levels. In addition, when a soft aggregate in the concrete exists, then the NGCS may also be a good choice as it has a “grooving” component with a deeper groove every 5/8 inch.

The owner (e.g., State or local government) must specify that they want NGCS so that it will be bid by the contractor. The NGCS surface is not dependent upon any aggregate surface type; it has been successfully placed in both hard and soft aggregate (e.g., limestone) in the concrete.

7. Summary of Diamond Grinding

Diamond Grinding has been very effective in (1) smoothing rough older pavements, (2) providing good frictional surfaces, (3) significant noise reduction, and (4) providing multiple pavement life extensions until major OL or reconstruction is required.

The diamond grinding technology has been around for a long time and has produced a significant increase in life for JPCP, particularly JPCP without dowels. Once DBR has been performed, however, faulting no longer becomes a problem. The newer JPCP with proper dowels will not fault for many years, if ever, but other distress types may develop that will still require diamond grinding to produce a smooth and well textured pavement.

Unusual situations exist, such as in Washington and Oregon, where wear from studded tires is excessive and continues to be a major problem. These projects can be diamond ground multiple times to remove the wheel path ruts and extend their service lives.

The key factors to long life of diamond grinding include the following:

- Project does not have serious durability problems (e.g., joint and crack deterioration).
- For non-doweled projects, full consideration is given to joint re-faulting (use of the AASHTOWare Pavement ME Design software can predict re-faulting). Dowel bar retrofit can be specified that will eliminate the faulting problem in the future.
- Large aggregate hardness is an issue that needs to be considered. If the existing concrete slab has a softer type aggregate (e.g., limestone), diamond grinding blade spacing must be considered to minimize texture wear.
- Proper construction of all CPR repairs must be accomplished. Each of the States mentioned herein has excellent specifications, special provisions, and standards that provide long life CPR and diamond grinding projects.
- Multiple diamond grinding on projects over the years has not shown much evidence of an increase in structural fatigue cracking. This is due to the small amount of thickness removed and the large gain in strength over time. The AASHTOWare Pavement ME Design software can be used as a tool to check for future transverse cracking, joint faulting and IRI.

With proper project selection, design, construction, and inspection of CPR projects involving diamond grinding, these States have all demonstrated that many restored pavements can last from 15 to beyond 20 years before another restoration or rehabilitation as long as joint load transfer is restored with dowel bar retrofits. Even after one of these projects has deteriorated to

the point of needing additional restoration, an additional CPR project can be applied with acceptable service life. Some projects in these States have had CPR with diamond grinding up to four times with acceptable service lives.

The future of diamond grinding holds great promise. The most innovative technology currently under development is the ability to determine the full pavement profile in the right and left wheel paths that will be used to determine how to grind a specific project to achieve optimal results. A high-speed profiler is used to obtain the pavement profile prior to grinding. The profile data are loaded into the diamond grinding machine. The profile data help to ensure that the correct amount of grinding takes place and minimizes the low or high spots along the pavement. 3D surveying, a technology under development, will also greatly help with cost.

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#4 Full Depth Repair Case Study for California, Missouri and Other Leading States

Abstract

Full depth repair (FDR), as described in this document, includes both partial slab replacement and full slab replacement to address any variety of distress. FDR has been used extensively since the 1970s for both jointed plain concrete pavement (JPCP) and jointed reinforced concrete pavement (JRCP). Combined with other needed concrete pavement restoration techniques, FDR provides for the long-term repair of structurally and/or functionally deteriorated joints, working cracks, shattered slabs, and multiple slab distress. FDR can significantly increase pavement service life before structural overlay or reconstruction is required, and it can increase the life of a pavement that will be overlaid with hot mix asphalt by providing high joint load transfer efficiency (LTE). Properly doweled and dowel anchored FDR avoids the problem of reoccurring joint faulting with resulting roughness and cracking.

This case study report focuses on FDR in California for JPCP and in Missouri for JRCP, but also includes information from Georgia, Minnesota, Utah, and Washington. .

FDR has been performed in California and Missouri on many projects. These States and others have developed and improved FDR techniques and specifications over a long period of time that are now producing 15+ year service lives as needed for JPCP and JRCP projects. In addition, a recent performance analysis in California indicated that a very high percentage (98.6 percent) of rapid strength concrete FDRs are performing very well after 3 to 8 years on a dozen projects. An effective concrete FDR process used by these States includes the following:

- Repair boundaries include all surrounding concrete deterioration.
- Removal of concrete is performed without damaging surrounding concrete and base.
- The repair area base course and subgrade are adequately repaired.
- Dowel bars are securely anchored into the existing slab to provide high long-term LTE.
- Concrete placement procedures and finishing do not result in an FDR that is significantly curled upward due to hardening during temperature extremes on sunny days.
- Concrete mixture does not exhibit a high amount of drying shrinkage.
- Opening to traffic at a sufficient strength to avoid excessive early fatigue damage.

1. Introduction

Full depth repair (FDR) of existing jointed plain concrete pavements (JPCP) and jointed reinforced concrete pavements (JRCP) is a critical repair technique that includes removal and replacement of a portion or all of a given slab or two or more slabs in a row. FDR is used frequently by State highway agencies across the U.S. to repair cracked or otherwise deteriorated slabs to extend JPCP and JRCP pavement life. FDR is typically performed in combination with other concrete pavement restoration (CPR) treatments, such as partial depth repair (PDR), dowel bar retrofit (DBR), cross-stitching, and diamond grinding to increase existing JRCP and JPCP life. The most critical keys to a successful FDR are that it must be structurally sound (e.g., length and width, long-term transverse joint load transfer through proper anchoring of proper sized and number of dowel bars), the boundaries must encompass existing deterioration, FDR concrete must be durable, the base course must be repaired if damaged, the dowels must be securely anchored, and the FDR must not be excessively curled upward from extreme hot temperatures at placement. Whether as a stand-alone treatment or as part of CPR, FDRs are perhaps the most critical component for achieving the desired JPCP and JRCP life extension.

This case study report focuses on FDR in California for JPCP and in Missouri for JRCP, but it also includes information from Washington (JPCP), Georgia (JPCP), Minnesota (JPCP and JRCP), and Utah (JPCP). These States consider all of the key aspects required to produce an effective concrete FDR process, as summarized below:

- Repair boundaries and area must be well defined to include existing concrete deterioration.
- Sawing and removal of all existing concrete must be performance without damaging surrounding concrete.
- The repair area foundation (base course, subdrainage) must be adequately prepared.
- Adequate levels of transverse joint load transfer anchored properly into the existing slab must be provided to prevent faulting.
- Concrete placement procedures and finishing must be adequate.
- Concrete curing must be practiced to inhibit a high upward curling of the slab.
- Concrete mixture must not exhibit a high amount of drying shrinkage.
- Intermediate joints must be sawed and sealed at optimal timing.
- Traffic must not be allowed on the repair until sufficient strength is attained to avoid excessive early fatigue damage.

FDR is considered a cost-effective treatment for many existing JPCP and JRCP that can help significantly increase pavement life before more costly treatments such as overlay or reconstruction is required. This document presents pre-construction design considerations, FDR specifications and construction, acceptance and inspection, and finally performance and survival

results for FDR. Note that not every aspect of FDR is included in this report. The oral interviews only covered the most important aspects.

2. Pre-Construction FDR Design Considerations

California, Missouri, and other States typically perform FDR to restore the local structural integrity of the pavement and to extend its service life. Individual slab replacement is a frequently used FDR practice in California for JPCP. Individual slab replacement may also require underlying base repair along with slab replacement (see Figure 12). Slab and base replacement consists of removing the concrete panels and sometimes underlying base and replacing both layers separately. New layers typically consist of cast-in-place concrete or rapid strength concrete (RSC) or precast concrete panels placed on either a lean concrete base or concrete base, separated by a bond breaker. Due to the high capacity demand for most of California's urban freeways, maintenance of concrete pavements in urban areas is a challenging task. To reduce the disruption to traffic, Caltrans carries out overnight repairs, limiting lane closures to the hours between 11 pm and 5 am. Severely distressed concrete panels are removed and replaced with RSC during this limited time period. Only specialty or proprietary cement mixes are used that meet the early opening strength requirements within 2 to 4 hours after placement.

Missouri utilizes FDR for their JRCP to repair both deteriorated transverse joints and cracks. As is typical for long-jointed JRCP (e.g., 60-ft joint spacing), the joints cannot handle the excessive opening and closing required due to temperature changes and combined with incompressibles infiltrating the joints results in considerable joint spalling. Also, durability issues like "D" cracking create a need for FDR at both joints and cracks. In addition, the reinforcement content of JRCP (e.g., < 0.2 percent area) is too low to maintain transverse crack tightness, and cracks begin to open and deteriorate over time, creating a major need for FDR.

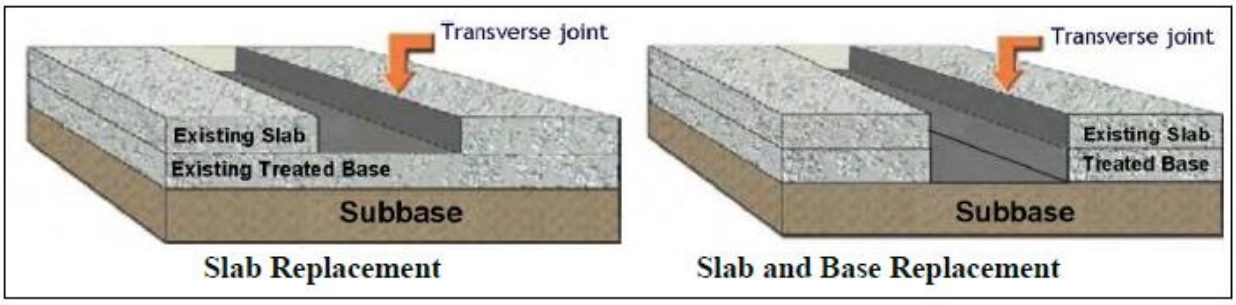

Figure 12. Slab and base replacement. (Caltrans Concrete Pavement Guide, 2015)

Appropriate Existing Condition

Slab replacement is required in some distressed slabs but not all concrete pavements displaying cracks, spalls, or other distress. Some cracked panels may provide acceptable performance for an extended period of time without any repair, depending on the traffic volume, vehicle loading, climate, underlying base conditions, and distress type, severity, and extent. Those same variables affect the anticipated service life of slab replacements. Slab and base replacement criteria based on existing pavement condition and anticipated service life are summarized below:

- <10% stage 3 cracking (3 or more pieces) – individual slab replacement.
- 10-20% stage 3 cracking – life cycle cost analysis to determine whether to replace slabs or lanes.
- >20% stage 3 cracking – lane replacement (all slabs).

Slab replacement in California is used to address severe deterioration (stage 3 cracking) of individual slabs in isolated areas when other strategies, including doing nothing, cannot extend the service life by at least 5 years or are not cost-effective.

Missouri uses FDR for both JPCP and JRCP slabs. The extent of longitudinal cracking, transverse cracking, and JRCP joint spalling are used as criteria for FDR. More than two JPCP cracks per slab or panel is the criterion that Utah uses for FDR. Washington uses FDR for corner breaks, settled slab, transverse cracks within 4 to 5 feet of transverse joints, highly distressed dowel bar retrofit slots, panels in three or more pieces, severely distressed (spalled and moving) transverse joints, and single unrepaired panel between two repaired panels. In Minnesota, concrete slabs with mostly underlying deteriorated transverse joints and cracks are replaced by FDR. The extent of deterioration is often not shown on the surface. FDR is used whenever deterioration of the joint/crack is significant (especially in wheel paths) and a longer term life is desired (e.g., > 10 years). Transverse, longitudinal, and corner cracking are criteria used for FDR in Georgia.

Replacing Distressed Slabs

In determining the need for slab replacement, Caltrans considers the extent and type of distress observed within a project. The following assessment criteria are used to determine if slabs need to be replaced. (Bhattacharya, Zola & Rawool, 2008)

- Slabs with two or more corner breaks.
- Slabs with stage 3 cracking (3 or more pieces).
- Slabs with segments that are moving relative to each other.

- Slabs damaged due to lack of support caused by settlement, base failure, or excessive curling.
- Slabs with longitudinal or transverse cracks more than ½ inch wide. Depending on traffic level, slabs with lower-severity cracks may also be included to ensure that additional repairs are not needed within the target rehabilitation design life.
- Cracks with spalling greater than 6 inches wide.

Replacing Base

Assessing the underlying base condition and determining when underlying base should be replaced is one of the more challenging aspects to slab replacements, since the base layer is not visible until removal of the slab layer. Coring and ground penetrating radar can be used to investigate its condition, but these techniques have limitations, including time, expense, and accuracy (limited sampling). An efficient method used by Caltrans for estimating base replacement during design is to use the surface distresses of the concrete slab to indicate base failure, considering that deteriorated base can extend beyond an individual slab into adjacent areas. Distress indicators include:

- Cracking with settlement > ¼ inch.
- Rocking slabs (typically with pumping fines).
- Spalling > 2 square feet total or > 2 inch wide over 75% of the crack length.

FDR Dowel Load Transfer Design

California. Standard Specifications 2015 Section 41-9.01C (3), Revised Standard Plan: RSP P10 and P8. For non-truck lane, three dowels per wheel path are used ,and for truck lane, four dowels per wheel path are used. Dowels are spaced at 12 inches. For slab thickness =<9 inch, 1.25-inch dowel diameter is used, and for slab thickness ≥ 9 inch, 1.5-inch dowel diameter is used. The minimum FDR length is 6 feet, and the lane width is 12 feet. Figure 13, Figure 14, and Figure 15 provide plan and layout of FDR with dowel bar retrofit. Note the isolation longitudinal joint where transverse joints in the adjacent lane do not match to avoid reflection cracking. Half lane widths are not allowed due to instability of the FDR under loading. Maximum length is 15 ft.

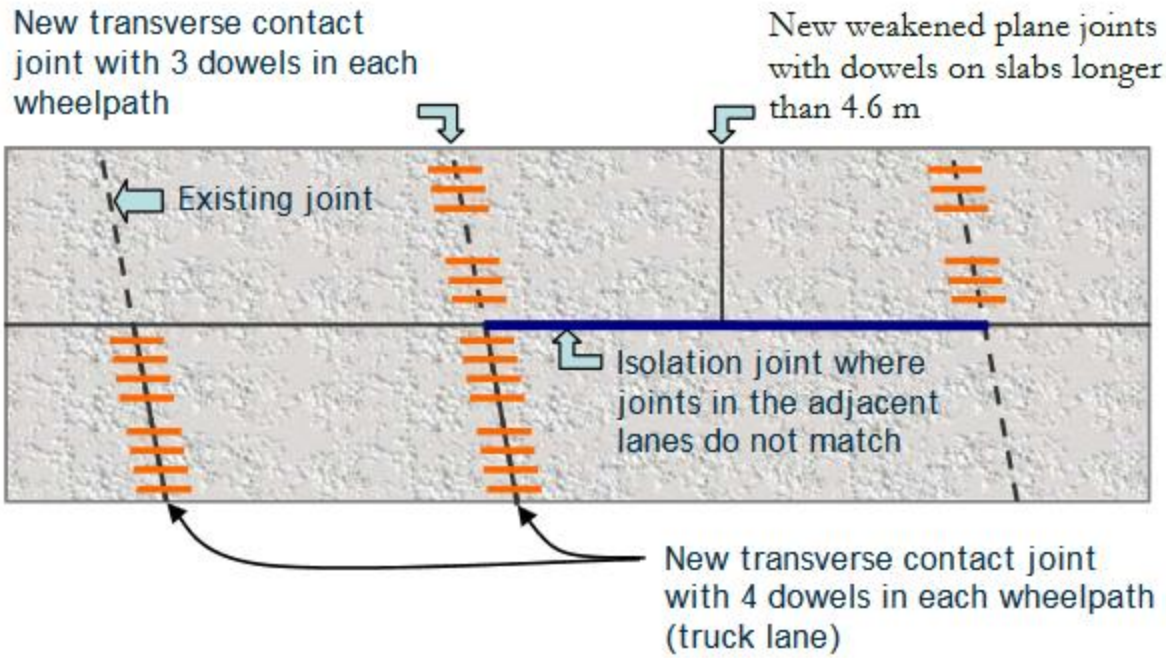

Figure 13. Plan showing number of dowel bars per joint in California. (Caltrans Slab Replacement Guidelines, 2004). Note that perpendicular joints are used with dowels.

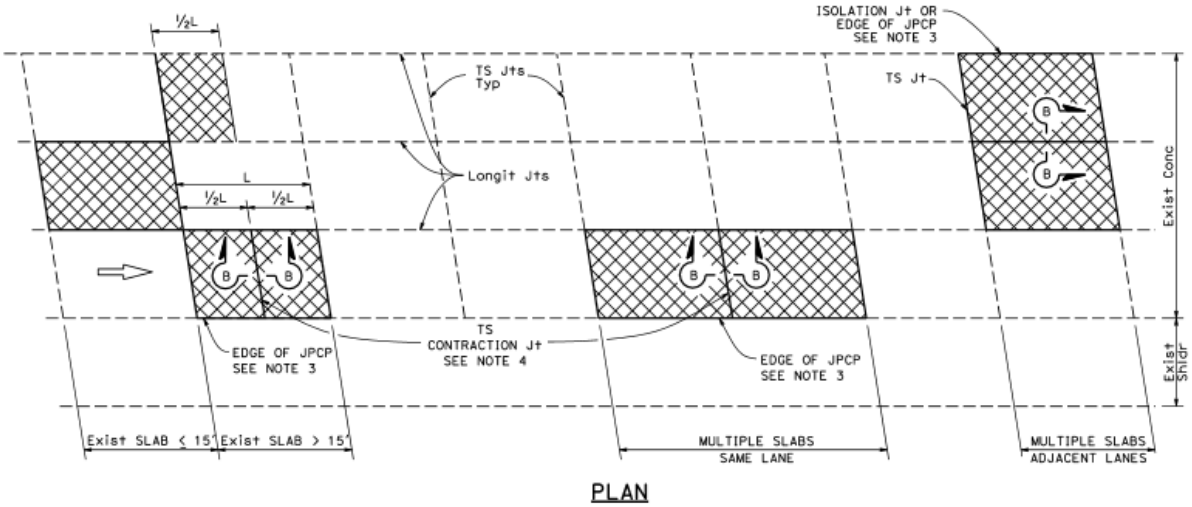

Figure 14. Plan showing individual slab replacement in California. (Caltrans Standard Plans, P8, 2015). Note that perpendicular joints are used with dowels.

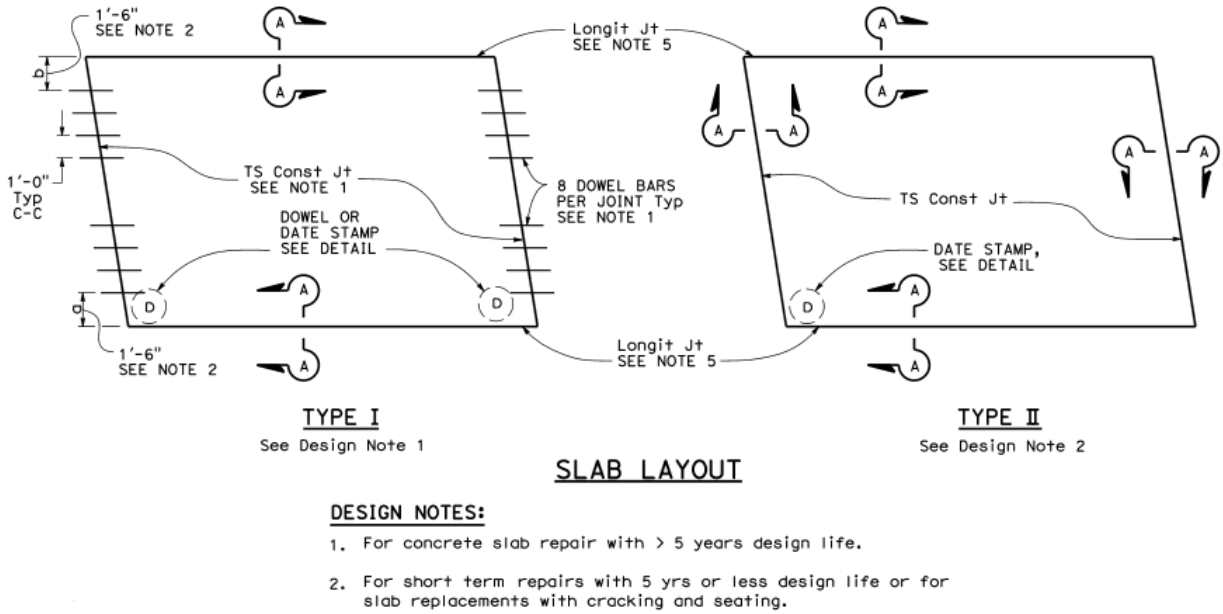

Figure 15. Slab layout showing individual slab replacement with number of dowels per joint in California. (Caltrans Standard Plans, P8, 2015). Note that perpendicular joints are used with dowels.

Missouri. Five dowels per wheel path, 12-inch spacing, and a 1.0-inch dowel diameter are specified. The minimum FDR length is 6 feet and the width is 12 feet. FDR of JRCP is typically done at transverse joints and at transverse deteriorated cracks that are the weak points in JRCP design. Both of these areas can be fully repaired with an FDR. Figure 16 shows plan and profile layout for JRCP in Missouri. Repair lengths ≤ 30 feet are not tied to the adjacent lane. Note the transverse joints spacing requirements (15 ft +/- 5 ft) if repair length is >= 30 ft.

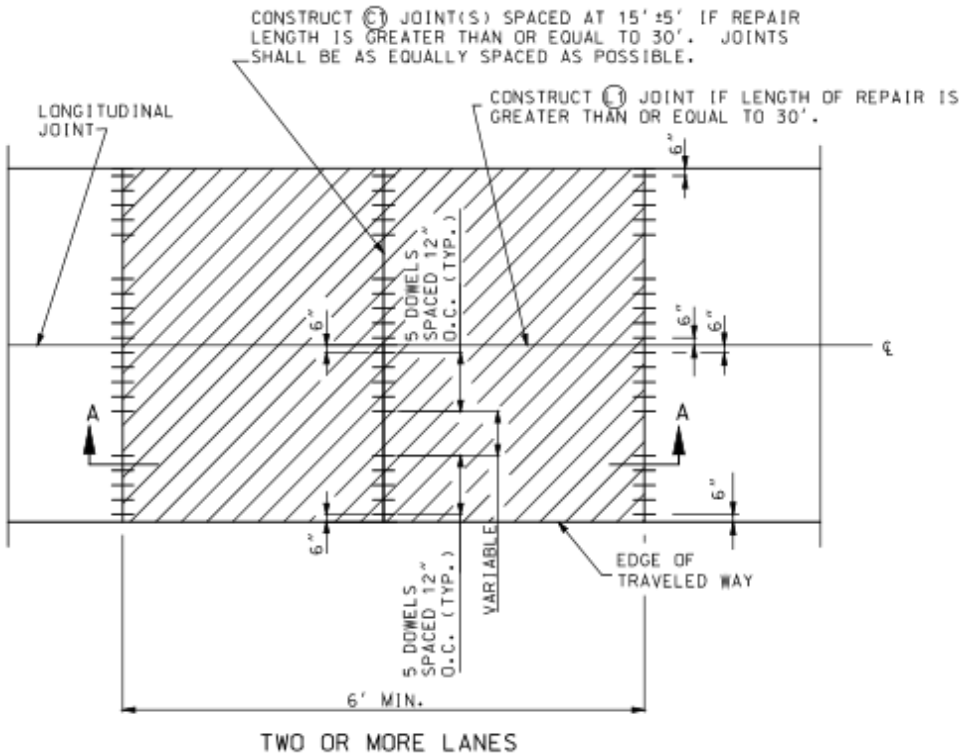

Figure 16. Plan showing number of dowel bars per joint in Missouri. (MoDOT Standard Plans, 2017)

Georgia. Eight dowels are evenly spaced across lane at 16 inches apart. Dowel diameter is 1.5 inches for slabs => 10 inches. The minimum FDR length is 6 feet and the width is 12 feet.

Minnesota. Currently, 11 dowels uniformly spaced per 12-ft-wide lane. Dowel diameter is 1.25 inches. Minnesota conducted an evaluation of dowel bar anchoring at MnRoad after some failures of dowel bars in FDR. They found a lack of bonding material around the dowels from all of the contractors. They changed their dowel design from 4 bars per wheel path to 11 dowel bars uniformly spaced due to problems with workmanship (not getting grout to surround the dowels) to increase the likelihood that the overall joint will provide the required load transfer.

The minimum FDR width is 12 feet, and the minimum length is 4 feet. To control longitudinal cracking that often occurs when FDRs are less than 6 feet long, Minnesota places two #4 tiebars transversely across the repair maintaining 3-inch clearance with the ends of the dowel bars. Minnesota has a standard for repair of longitudinal joints with similar design only rebars are used instead of dowel bars. They also use shorter 15-inch dowel bar lengths in their repairs, as their research has shown they perform similar to 18-inch bars. This provides some cost advantages.
Washington. Eleven dowels uniformly spaced per 12-ft-wide lane. Dowel diameter is 1.5 inches. The minimum FDR width is 12 feet. The minimum length is 6 feet. If JPCP includes existing dowels, they are avoided by moving over 6 inches. Some projects in other States have not had success in drilling a dowel hole between old dowels. One solution is to extend the FDR

by 12 inches to drill into sound concrete free of existing dowels. This will ensure that the new dowels are placed into sound concrete.

Utah. Four dowels per wheel path at 12-inch spacing and 1.5-inch dowel diameter. The minimum FDR length is 5 feet, and the width is 6 feet.

There are some interesting differences in these States' designs for dowel bar layout, dowel bar diameter, and length of FDR, as summarized in Table 10.

Table 10. State FDR dowel bar layout and diameter.

State	Slab Thickness, inches	Location of Dowels	Number of Dowels Across Entire Transverse Joint	Dowel Diameter, inches
California	=< 9	Wheel paths	8 @ 12-inch spacing*	1.25
	>9	Wheel paths	8 @ 12-inch spacing*	1.5
Missouri	All	Wheel paths	10 @ 12-inch spacing	1.0
Utah	All	Wheel paths	8 @ 12-inch spacing	1.5
Washington	All	Uniform	11 @ 12-inch spacing	1.5
Minnesota	All	Uniform	11 @ 12-inch spacing	1.25
Georgia	=< 10	Uniform	11 @ 16-inch spacing	1.25
	>10	Uniform	11 @ 16-inch spacing	1.5

*Use for truck lanes. Non-truck lanes use 3 dowels per wheel path.

The different States have widely differing dowel bar sizes and layout. California and Utah specify four 1.5-inch dowels per wheel path. Missouri specifies five 1.0-inch dowels per wheel path. Washington, Minnesota, and Georgia place 11 dowels (1.25 to 1.5 inches) uniformly across the traffic lanes. The dowel diameter increases with slab thickness in California and Georgia. This is logical, since increased traffic results in thicker slabs and thicker dowels are required to carry the high numbers of shear stresses without causing the concrete surrounding the dowels to wear down.

The dowel bar diameter is perhaps the most critical factor, since a slight change in diameter dramatically affects the steel/concrete bearing stress under a wheel load, which greatly affects joint load transfer efficiency (LTE), erosion, and faulting. Many experimental tests have shown that larger diameter bars show less joint faulting in new JPCP and also in FDR. (Darter & Barenberg, 1976; Snyder & Lippert, 1990) Given the fact that transverse joint faulting is always greater at the outer edge of the slab, the outer two or three outer dowel bars are most critical to provide long-term high LTE. Therefore, the diameter of these bars and spacing are most critical.

Which is the best design? The larger diameter dowel bar layout should be the most effective in reducing long-term transverse joint faulting of FDR. California, Utah, Washington, and Georgia

have shown no significant faulting of FDR with their designs that use four to five larger dowels per wheel path. However, Missouri and Minnesota have also had some success using a larger number of smaller dowels. However, placing them in the wheel paths would appear to be a more efficient layout since that is where the heavy wheel loads pass. There have been some dowel bar failures in States are due to improper anchoring at construction; water-filled dowel slots and settlement issues have occurred.

Can dowels be anchored between existing dowel bars? Contractor feedback concludes that placement of dowels between existing dowels does not work. Practical solution is to extend the FDR about 12 inches to get past the existing dowels (e.g., move the transverse joint about 12 inches).

Length and Width of FDR

Another major design detail is FDR length (both minimum and maximum) that may have an effect on joint faulting and cracking. Nearly all States require full lane width for structural stability purposes. State recommendations for minimum length vary from 4, 5, and 6 feet. While most of these States require 6-ft minimum length, none report having excessive longitudinal cracking develop over time. Research has been somewhat inconclusive for doweled FDR, with some evidence that shorter slabs (especially 3 ft) tend to develop more longitudinal cracking due to:

- A combination of high built-in upward slab curling (placement in hot summer day) that under axle loading could result in high top-of-slab tensile bending stresses, causing top-down fatigue cracking.
- Compressive stress from the adjacent pavement on hot days that could cause high compression in the short FDR, resulting in longitudinal type ladder cracking. (Darter, Barenberg & Yrjanson, 1985; Snyder et al., 1989; Yu, Mallela & Darter, 2005)

Minnesota has used 4-ft slabs on many projects with reportedly no significant amounts of longitudinal cracking. Minnesota believes it was more cost-effective, as a precautionary measure, to reinforce the 4-ft-long FDR transversely to hold tight any longitudinal ladder cracks, rather than construct a longer FDR with no reinforcement. So it appears that a range of FDR lengths from 4 to 6 feet have been effective in controlling faulting and cracking and that it's primarily a lower cost advantage in Minnesota. California, Missouri, Georgia, and Washington require minimum 6 feet length and 12 feet width (Missouri uses a 5-ft length on some projects). Utah requires 5 feet minimum length and 6 feet minimum width.

The maximum length of FDR is also important. Most States have maximum lengths to control transverse fatigue cracking. Missouri has a 15 + or – 5 ft maximum when the repair area equals or exceeds 30 feet. California has a maximum joint spacing of 15 feet.

Training

Both States and contractors agree that maintenance and construction personnel should receive training on a regular, ongoing basis so they may provide the necessary oversight to ensure proper quality assurance on FDR projects. Training helps with situations where field conditions are not considered ideal according to specification. A field guide for panel replacement and selection will help inspectors. The field guide should identify the main criteria for repair of midpanel cracks, corner cracks, and moving panels, with emphasis on location and size. In Utah, the success of the repair depends on the training of the designers, inspectors, and contractors. Caltrans developed a slab replacement video to train inspectors and contractors. Caltrans also mandates pre-construction, just-in-time training for both contractor and Caltrans personnel. Minnesota used to require inspection training and created several YouTube videos documenting the proper methods to perform and inspect various repairs for training inspectors. Georgia also used to provide extensive training for contractors and State personnel.

3. FDR Specifications

All of the States included in this study have up-to-date and highly effective specifications for FDR. These States have spent many years refining and improving these specifications. California began experimenting and refining FDR or individual slab replacement in the early 1970s, and other States developed theirs soon after, as truck traffic greatly increased. Table 11 summarizes the States' specifications, special provisions, and other documents.

Table 11. Summary of State specifications for FDR.

State	Specification
California	Section 41-9: Individual Slab Replacement, ISR (2015) Concrete Pavement Guide
Georgia	GA 452, 504, 609
Minnesota	MnDOT 2302 SP
Missouri	MoDOT Standard Specification 613.10. MoDOT Standard Drawing 613 (Sheet #1)
Utah	UDOT: PCC 3055 Precast: SP 02757S, PB4, 2753
Washington	Standard Plan A-60.10-03. WSDOT 5-01 Cement Concrete Pavement Rehab.

Specific States and other research studies made various recommendations regarding FDR of concrete surfaces that should be considered in specifications or guidelines.

FDR Portland Cement Concrete (PCC) and Base Removal (if needed)

The base course beneath the slab must obviously provide adequate long-term support to the new FDR. If this is not provided, the FDR will fail no matter how well all the other construction steps are performed. This key point is recognized by all of the States.

California: Saw cut full depth all four sides, with additional sawing allowed, and remove concrete within the saw cuts. Before removing any type of asphalt treated, cement treated, or concrete base, saw cut the outline of the base removal area using a power-driven saw with a diamond blade. Cut asphalt treated base at least 2 inches deep on a neat line perpendicular to the base surface. Cut cement treated or concrete base full depth.

Missouri: Saw cut full depth and lifts out concrete. Repair damage and, if needed, reconstruct the base. Missouri assumes 10 percent of the repair area will need base replacement and compaction in project documents, but this quantity is seldom required during construction.

Georgia: Saw cut full depth on four sides, remove concrete (lift out pieces), and clean and fill overcuts. During removal, the contractor avoids damaging any portion that will not be removed. Remove and replace any areas that are damaged from slab removal. Loose base material is removed and repaired to produce a well compacted (aggregate) base.

Minnesota and Utah: Saw cut full depth on all four sides, remove deteriorated concrete (lift out pieces), and restore & repair the base if needed.
Washington: Saw cut full depth along all joints. Adjacent concrete must be protected from spall damage at all costs. An additional vertical full depth relief saw cut located 12 to 18 inches from and parallel to the initial longitudinal and transverse saw cut locations is also required. Repair of the base course is also needed if it is disturbed.

Dowel Bar Anchoring (The Most Important Step)

Anchoring of the dowel bars into the existing slab is the critical construction step for FDR. If this is not done adequately, the dowels will eventually become loose, the FDR joint will lose joint LTE, and pumping and faulting will develop along with cracking and settlements. Figure 17 and Figure 18 show one of many poor anchoring examples, where the grout or epoxy material was not properly surrounding the dowel bar and the continual pounding of loads has completely eroded it away.

Figure 17. Poor anchoring of dowel into hole that resulted in faulting of the transverse FDR joint. (Photo Courtesy of Mark Snyder)

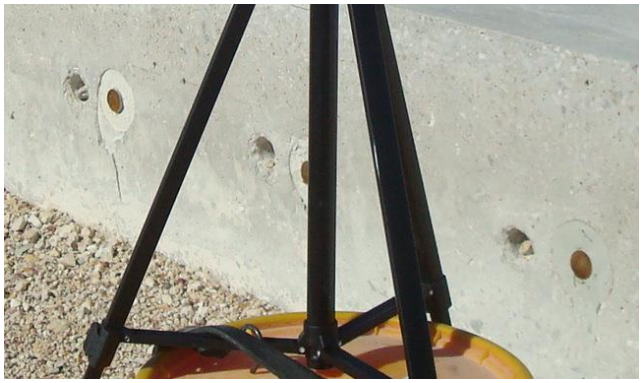

Figure 18. Use of grout retention disks provides good coverage. (Photo Courtesy of Mark Snyder)

California. Drilled hole is ¼ inch larger than dowel. Bars are anchored with epoxy and no grout retention disks are used. Figure 7 shows the dowel bar anchoring requirements in California.

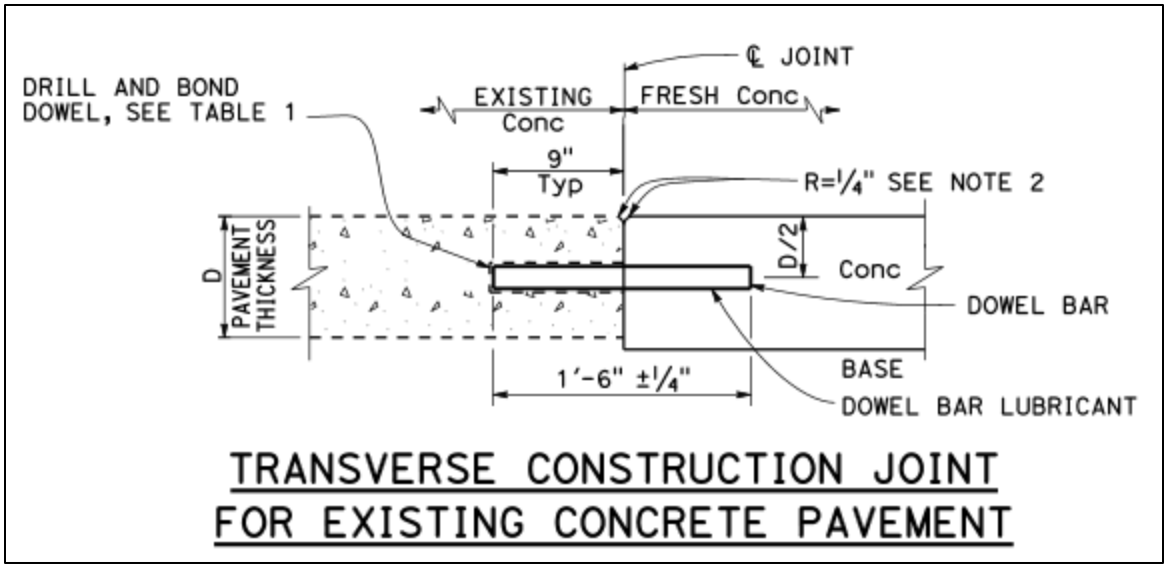

Figure 19. Dowel bar anchoring in California. (Caltrans Standard Plans, P10, 2015)

Missouri. Drill holes, clean, inject epoxy resin into hole, rotate bar during insertion, use grout retention rings, end caps on protruding dowels, coat bars with lubricant. The grout retention rings are an important part of a system of anchoring dowels, and that system includes the proper consistency and strength of anchor materials (cementitious grout or epoxy-based material), the right annular gap for the material being used, the proper injection of the material into the drilled hole, proper technique for inserting and rotating the dowel for good filling of the gap and the grout retention ring to force full grout coverage at the joint face and hold it all in place until it sets. Figures 8 and 9 show a typical dowel bar anchoring with a grout retention disk.

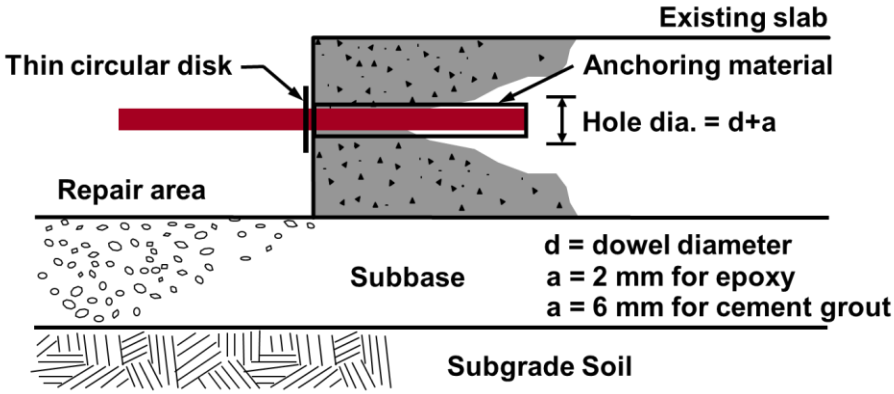

Figure 20. Dowel bar anchoring in slab face with grout retention disk. (FHWA, 1998)

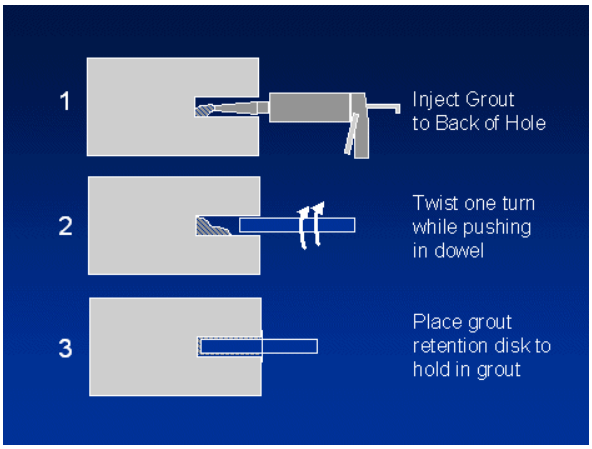

Figure 21. Illustration of recommended procedure to anchor dowels securely into face of slab. (Courtesy of Mark B. Snyder)

Figure 22. Grout retaining disk. (Photo Courtesy of Mark B. Snyder)

Georgia. For 1.5-inch dowel diameter, drill 1 5/8-inch diameter holes. Clean drilled holes of contaminants. Use epoxy adhesive in a cartridge with a mixing nozzle that thoroughly mixes the two compounds as dispensed into the back of the hole. Insert the dowels into the holes at correct horizontal and vertical alignment. Fill the cavity surrounding the dowel bar with epoxy. Coat the protruding portion of each dowel with bond breaking material.

Minnesota. Minnesota conducted a 2013 study on adequacy of anchoring dowels after some failures on a project and found a lack of bonding agent around the dowels from all contractors. They changed back from 8 dowels bars to 11 bars due to this problem with workmanship and increased the diameter to 1.25 inch, spread over 11 dowels uniformly spaced.
Drill hole diameter 1/8 inch > dowel bar diameter to obtain surrounding grout anchor.

- Grout anchoring capsule – A cementitious non-shrink grout, pre-mixed capsule, in a water permeable wrapping, is placed in water for 2 minutes, and then pushed into the hole by a dowel bar, which forces it out around the dowel surface.

- Epoxy adhesive material – An epoxy two-part mixing nozzle is placed in back of hole and epoxy is injected. Then the dowel bar is pushed into the hole, thus forcing epoxy around the dowel surface. The free end of dowel bars are coated with a debonding agent.

Minnesota has had success without using grout retaining rings. Performance results with the grout capsules were good, but the holes had to be 1/8 inch larger in diameter. Minnesota changed back from 4 dowels in each wheel path to 11 dowel bars spread across the lane due to problems with workmanship (could not get grout to surround the dowels so use more dowels were specified).

Washington. 1.75- to 2.00-inch diameter holes are drilled, cleaned, and epoxy resin is injected into the hole. The dowel bar is rotated during insertion and grout retention rings are required with end caps protruding from dowels. The bars must be coated with a lubricant.

Utah. Drill holes, clean, inject epoxy resin into hole, rotate bar during insertion, use grout retention rings, end caps protruding dowels, coat bars w/lubricant. Good anchoring of bars was reported.

Note that Missouri, Washington, Utah, and other agencies (including the precast industry) require the use of grout retention rings to provide a more consistent way for anchoring material to surround the dowel bar at the joint interface where bearing stress are highest. Extensive testing and evaluation work indicated the grout retention rings would provide improved coverage surrounding the dowel bars, especially at the interface where the bearing stresses will be the highest. (Snyder & Darter, 1990) A quote from the Wisconsin DOT FDR manual describes the disk:

> "A plastic or nylon epoxy retention disk that fits tightly over the dowel and effectively seals the gap around the hole is required to prevent flowable epoxy from running out of the hole. This disk may be about 2 inches larger in diameter than the dowel being used and should be manufactured to fit snugly over the bar and slide up against the face of the slab when the bar is being inserted into the hole. The retaining disk is inserted over the dowel bar and pushed to flush against the PCC surface to retain the epoxy. The disk will keep most of the material in the dowel hole and provide an excellent bearing surface at the face of the slab." (Titus-Glover & Darter, 2007)

Lubricated Ends of Dowels

All of the States require that the ends of dowels be lightly greased or lubricated to avoid joint lockup and subsequent cracking around the joint. A recent study showed that the pullout of ungreased dowels requires a significantly higher force than for greased dowels, suggesting that a lack of grease may restrain a doweled joint from opening and closing and cause joint lockup. (Khazanovich, Hoegh & Snyder, 2009).

Cast-in-Place Conventional Concrete Materials

Minnesota Standard Specifications 2461 specifies that the contractor designs concrete mixtures. Concrete Mix 3R52 requires minimum 28-day compressive strength of 4,000 psi. Missouri Standard Specification 613.10.2.3 requires either conventional or rapid strength PCC for conventional cast-in-place FDR. Washington uses both conventional and rapid strength PCC for FDR. Utah Standard Specification 3055 (conventional concrete) is used if opening time >48 hours. If less than 48 hours, precast concrete repairs are typically used.

Cast-in-Place High Early Strength Concrete Materials

RSC is a special or proprietary cement mixture that develops rapid strength. California Standard Specifications 2015 Section 41-9 includes RSC specifications. Caltrans carries out overnight lane closures to replace existing distressed slabs, so most of the FDR work is done at night due to the high traffic levels. RSC slab repair material is frequently used in California. Observations of RSC repairs indicate that durability problems often develop over time. Slabs constructed using RSC mixtures can be opened to traffic in less than 4 hours. RSC mixtures develop a flexural strength in excess of 400 psi within 4 hours. The 400 psi requirement 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 may be compromised. There are various RSC cement mixtures available on the market. Among these, “CTS Cement” and “4×4 Concrete System” commonly are used in California for slab replacement.

Missouri’s high early strength concrete is often used. However, early shrinkage cracking is a concern. The contractor interviewed for this study has had good experience with high early strength materials. These materials can increase the cost of a project greatly. Mobile mixers are commonly used because of early set up time.

Minnesota uses Concrete Mix 3A32HE when opening time is a minimum of 48 hours.

Georgia specifies 2,500 psi compressive strength within 24 hours. Slab replacements are scheduled so that the concrete will have a curing time of at least 4 hours. Concrete typically has at least 1,500 psi at this time.

Washington allows rapid hardening hydraulic cement meeting the requirements of ASTM C 1600. The maturity method for opening strength has had good success in Washington. However, the rapid set material often comes out with a high slump, and inspectors have problems with that. This goes back to training, the inspectors need to know what to expect. Utah typically uses precast slabs if opening time <48 hours. High early strength PCC slabs often develop durability problems and are not often used.

Curing and Avoiding Temperature Extremes and Extreme Upward Curling

Each State addresses proper curing procedures for cold and hot weather paving. One key aspect of hot weather placement of FDR that needs special attention is best illustrated with an FDR project in Ohio under Strategic Highway Research Program (SHRP) Project C-206. (Yu, Mallela & Darter, 2006) The 6-ft-long by 12-ft-wide FDR slabs included several different high early strength materials. The FDRs were placed typically about 10 am and opened to traffic by 5:30 pm. The temperature of the slab (top, middle, and bottom) was measured at the time of concrete hardening, as was the peak concrete temperature during that time. These four temperatures were then averaged for each material and day (e.g., for one mixture/day at time of concrete hardening: 132 °F top, 125 °F middle, and 100 °F bottom of slab were combined with the peak concrete temperature 177 °F during that day for an average of 133.5 °F). These daily averages of concrete slab temperature during curing varied from 101 °F to 133.5 °F. The overnight average low temperature of the concrete slab (again average of top, middle, and bottom) over these days was a constant 50 °F. The differences ranged from 51 °F (101 to 50 °F) to 84 °F (133.5 to 50 °F) for a variety of high early strength mixtures and days. These differences correlated strongly with longitudinal cracking of the FDR on this project.

Many of the Ohio FDRs developed longitudinal cracking over the weeks and months after opening to traffic. A strong positive correlation existed after 2 months of traffic between the "difference between average curing and overnight low temperatures" and "percent slabs longitudinally cracked at 2 months." Thus, the higher the temperature difference between curing temperature and overnight low temperature, the greater the amount of longitudinal FDR cracking. For example, for the extreme temperature differences, these results were produced at 2 months:

- FDR temperature difference: 101 to 50 °F = 51 °F; this material showed 12 percent FDR with longitudinal cracking (lower amount of upward curled slab).

- FDR temperature difference: 134 to 50 °F = 84 °F; this material showed 100 percent FDR with longitudinal cracking (highly upward curled slab).

The longitudinal cracks were top down and located 3 to 5 feet from the outer edge with the shoulder. This is the critical lateral distance where tensile stresses may occur when a heavy truck axle passes over the FDR near the outer edge. In addition, the weight of the upward curled slab would increase this stress. These cracks developed over time similar to top-down fatigue cracking.

Extreme temperatures require special actions that prevent an FDR from becoming significantly curled upward due to temperature extremes on sunny days. Extreme temperatures like this can be avoided simply by cooling the surface of the slab after placement with several sprays of cool water (without marring the surface) or wetted burlap. Wet burlap prevents rapid loss of moisture and maintains reasonable levels of concrete temperature during hydration. Wet burlap significantly decreases the potential for built-in negative temperature gradients during daytime placements. (Titus-Glover & Darter, 2008)

Bond Breaker

Caltrans uses a suitable bond breaker such as plastic sheeting or curing paper between the slab and lean concrete base course. The bond breaker allows slabs and base to move independent to each other to reduce reflection cracking and to provide flexibility for slab curling due to temperature difference between the top and bottom of the slab.

In Washington, the longitudinal and transverse joint faces of the surrounding slabs are lined with a bond breaking material such as polyethylene plastic sheeting to minimize friction between the old concrete slab face and the new repair concrete. This material is also placed on the existing base course to reduce friction. This material needs to be "fitted" along the joints to avoid problems. This material reduces tensile stresses and reduces cracks forming in the FDR.

When bonding is broken between two slabs placed on top of each other, the upper slab is free to curl upward. Due to its weight and any loading, this creates a high potential for slab corner cracks. Thus, careful use of a bond breaker is advised.

Minimum Time to Open Traffic

California. Standard Specifications 2015 Section 41-9.01D(6)(b) includes RSC specifications for minimum time to open. RSC slabs can be opened to traffic in 2 to 4 hours. The modulus of rupture for RSC must be at least 400 psi for opening to traffic and at least 600 psi at 3 days.

Minnesota. FDRs may be opened traffic at 3,000 psi compressive strength. Control cylinders are cured at the same temperature as the slab.

Georgia. Minimum traffic opening time is 4 hours, and minimum compressive strength is 2,000 psi for cast-in-place concrete. PCC typically has at least 1,500 psi compressive strength by then. A Long Term Pavement Performance (LTPP) program study on FDR with high early strength concrete in Georgia showed excellent performance over a 5-year period. However, similar test sections in Ohio showed longitudinal cracking within few weeks after opening to traffic. This premature cracking correlated strongly with the slab temperature difference between curing temperature and overnight temperature. (Yu, Mallela & Darter, 2005)

Missouri. 2,000 psi minimum to open to traffic.

Utah. 4,000 psi minimum and 48 hours minimum to open to traffic.

Washington. Used maturity meters to determine early opening strengths. Minimum opening compressive strength to traffic is 2,500 psi.
The SHRP C-206 Manual of Practice recommends the following after a major research study into opening times: (Yu, Mallela & Darter, 2006)

- Minimum concrete modulus of rupture of 300 psi by third-point testing.
- Minimum concrete compressive strength of 2,000 psi.

4. Inspection/Acceptance

The inspection and acceptance process for FDR focuses on proper removal of concrete, dowel alignment and anchoring, base course repair, smoothness, concrete modulus of rupture or compressive strength, cracks and other observable distresses in the FDR, and distress in surrounding concrete. These criteria are quite effective to control the construction quality of FDR, and they are practical to measure and fairly repeatable.

California Standard Specifications section 41-9.03H Noncompliant Individual Slab Replacement, ISR, describes inspection/acceptance criteria for slab replacement. The inspection/acceptance criteria requires replacement of an individual slab replacement or FDR that has any of the following defects:

- One or more full depth cracks.
- Concrete raveling.
- Noncompliant smoothness, except the contractor may request authorization for grinding and retesting.

- Noncompliant modulus of rupture.

California Standard Specification section 41-9.01D(6) describes the Department Acceptance. Section 41-9.01D(6)(a) states Individual Slab Replacement must pass visual inspection and have a coefficient of friction of at least 0.30 determined under California Test 342, and section 41-9.01D(6)(b) states RSC must have a minimum modulus of rupture of 400 psi at opening to traffic and at least 600 psi at 3 days..

Minnesota randomly chooses two separate repairs and mark two dowel bars for assurance coring and observation for each 1,500 feet of FDR. Potential damage is determined by sounding using a chain for the FDR and adjacent slab joints.

Georgia measures joint LTE of the FDR with a truck and does not allow any cracks in them. Epoxy surrounding the dowel bars must be set before FDR material is placed. All dowels must be in alignment. Concrete strength criteria must be met before FDR can be accepted.

Missouri, Utah, and Washington use minimum 28-day compressive strength criteria. In addition, no cracks should be visible on the surface. Utah and Washington also require base restoration or compaction.

Incentive/Disincentive

In the low-bid environment it is essential to ensure that a high-quality contractor is selected that knows how to provide a quality repair. If incentives are included for FDR in a contract, contractors may adjust their unit bid prices accordingly, believing they can meet the incentives. Incentives/disincentives are used by only a few States for FDR. Minnesota and Utah have incentives/disincentives for smoothness that is part of overall diamond grinding. California uses disincentives for strength. California Standard Specifications section 41-9.01D(6)(b) ISR specifies the following requirements for RSC modulus of rupture disincentives:

- If RSC modulus of rupture at opening age is at least 400 psi and at 3 days is greater than or equal to 500 psi but less than 550 psi, the Department deducts 10 percent of the payment for individual slab replacement—RSC.
- If RSC modulus of rupture at opening age is at least 400 psi and at 3 days is greater than or equal to 550 psi but less than 600 psi, the Department deducts 5 percent of the payment for individual slab replacement—RSC.

Since the success or failure of an FDR depends so highly on achieving proper dowel bar anchoring, an incentive could be placed on achieving this requirement. This could be done two ways:

- Through random coring of the dowels to check visual coverage (as in the DBR specifications of the States described above), as per Minnesota.
- Falling Weight Deflectometer load testing of the FDR joints for LTE, as per Georgia. This test would be very easy to perform once testing conditions were standardized. The higher the FDR joint LTE, the greater the incentive of course. A standard load testing procedure would need to be specified.

As a further aid to inspection/acceptance, Appendix A includes a "Full-Depth Repair of PCC Pavements Checklist" that was prepared for Wisconsin DOT. (Titus-Glover & Darter, 2008)

5. Performance/Survival

The performance of FDR in these States has been very good over many years. A summary of FDR performance by each of the six States are presented in Table 12.

Table 12. Summary of performance of FDR by State.

State	FDR Performance
California	Individual slab replacement is designed to last for 5 to 10 years and lane replacement for 20 years. Individual slab replacement with RSC is expected to last for less than 5-10 years due to durability issues. Note that the actual service life of these pavements is far more than this as indicated in a 2008 survey.
Georgia	FDR works very well when specifications are followed. Excellent success with the FDR since the mid 70's. Many slabs exceeded 20 years of service life.
Minnesota	Performance of FDR has been excellent over many years. Many FDR slabs had service lives of 15 years on heavily trafficked Interstates and more than 15 years on lower truck volume routes.
Missouri	Overall FDR strategy is showing good performance for Cast-In-Place PCC. Many FDR projects last for 15 to 20 years. Dowel design seems sufficient to prevent joint faulting.
Utah	Very good performance for Cast-In-Place and Precast FDR. Minor spalling developed in surrounding PCC as well as FDR slabs. Cracking and joint fault are also minor. Dowel design appears to prevent faulting of the transverse joints.
Washington	FDR performs very well overall with at least 15 years of service life. Some FDR have cracked initially and sometimes those cracks spalled. Joint faulting is not a problem for FDR slabs.

RSC slab repair material is frequently used in California for early opening to traffic. These individual slab repairs are designed to last only 5 to 10 years; however, in 2008, Caltrans surveyed 15 projects covering 5,430 RSC slabs across California and only 74 slabs (1.4%) showed some type of surface distress (see Figure 23). This statistic indicates excellent performance of RSC FDR in California, as these projects were 3 to 8 years old at the time. Thus, the mean service life of RSC FDR is expected to be far more than 5 to 10 years.

Of the few RSC FDR that were distressed, the majority exhibited midpanel and surface cracking as shown in Figure 24. Distresses in the FDR included spalls and corner cracking, midpanel and surface cracking, shrinkage cracking, aggregate pockets, and moving slabs. Figure 25 shows the percent distressed slabs on surveyed RSC projects.

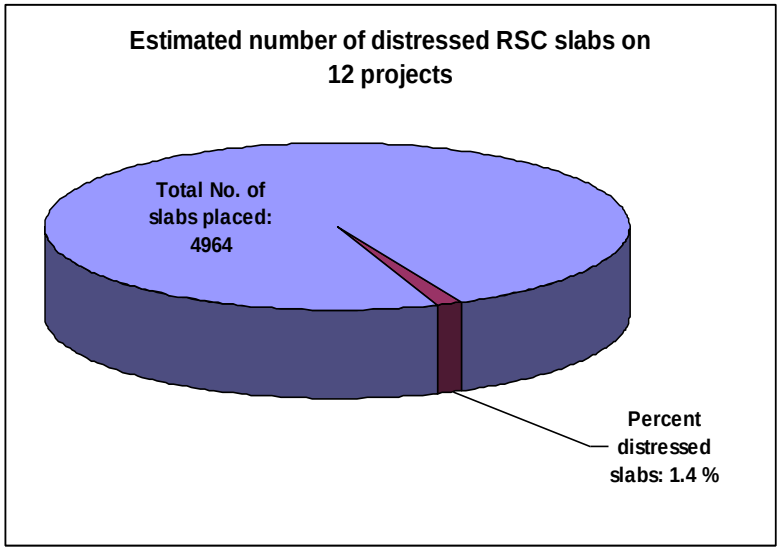

Figure 23. Percent distressed slabs on 12 surveyed RSC projects in California.

Figure 24. Midpanel cracking on an RSC slab on I-5 in Los Angeles County.

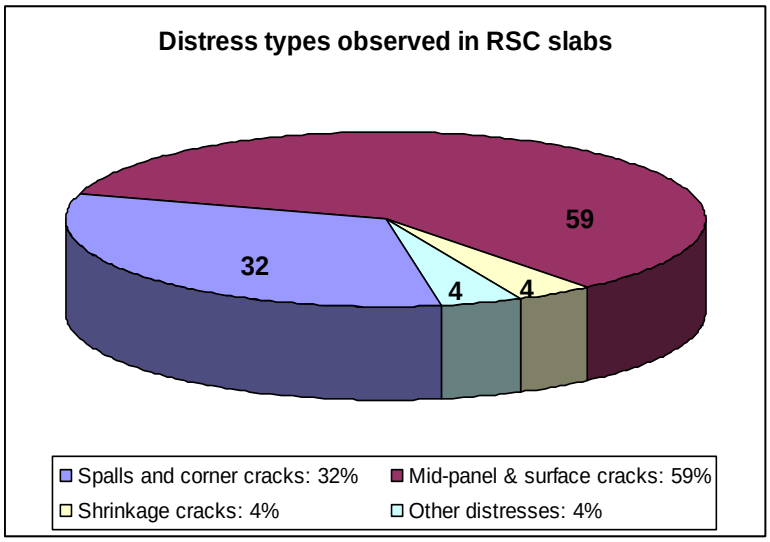

Figure 25. Distribution of distress types observed in the distressed slabs on RSC projects.

The following is a summary of the Caltrans RSC survey:

- Only 1.4% of the slabs surveyed showed premature distress.
- Midpanel and surface cracking was the predominant distress type, followed by corner breaks and spalling, respectively.
- RSC material composition/design did not cause panel distresses or failures. Very few minor distresses were observed where proper construction techniques where followed.
- Spalling was caused primarily by improper placement of the bond breaker foam/expansion joint material (or lack thereof).
- In a few instances, midpanel cracking resulted from excessive slab lengths or crack migration from adjacent slabs.
- Corner breaks may be attributed to lack of recompaction of the existing supporting material prior to RSC placement.
- Early age panel distresses and failures observed on all the sites can be attributed to construction errors.

Thus, overall these States are experiencing an FDR survival life of 10 to 20+ years over a wide range of climates, traffic loads, materials, and designs.

6. Summary of FDR

The FDR technology has evolved over many years and is now a reliable restoration technique that is critical to producing a significant increase in life for JPCP and JRCP. When combined with repairs such as dowel bar retrofit and diamond grinding, FDR can significantly enhance service life. California uses RSC for partial and full slab-width FDR in JPCP and has achieved excellent performance for a very high percentage of FDR. Mean service life should be well over

10 years for RSC FDR. Missouri has used FDR for JRCP for transverse joint and crack replacement with a service life of 15 to 20 years. The key factors to overall long life of FDR include the following:

- Existing pavement does not have serious durability problems (e.g., joint and crack deterioration), or if some exists, the FDR boundaries are enlarged to include significant deterioration of concrete.
- FDR of deteriorated slabs can be combined with dowel bar retrofit on transverse joints to eliminate the transverse joint faulting problem over the service life. Proper anchoring of the dowels into the slab is the critical construction step for FDR because, if this is not accomplished, the joint will fault, creating roughness, spalling, and cracking.
- Proper construction of other FDR steps must also be accomplished. California, Missouri, and the other States mentioned herein have excellent specifications, special provisions, and standards that provide long-life FDR projects. Nevertheless, the proper anchoring of dowels is the most critical construction issue for FDR. Every effort must be made to achieve a high LTE.

With proper project selection, design, construction, and inspection of CPR projects involving FDR, States have demonstrated that many restored pavements can last from 15 to beyond 20 years before another restoration or rehabilitation. Even after one of these projects has deteriorated to the point of needing additional restoration, a subsequent CPR project can be constructed with an acceptable service life.

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Appendix A: Full-Depth Repair of PCC Pavements Checklist

(Prepared for Wisconsin DOT, Titus-Glover & Darter, 2008)

This checklist for Wisconsin full-depth repair design and construction practices was adapted from the Federal Highway Administration (FHWA) and the Foundation for Pavement Preservation (FP2).

Preliminary Responsibilities

- Verify that pavement conditions have not significantly changed since the project was designed and that full-depth repair is appropriate for the pavement.
- Agree on quantities to be placed, but allow flexibility if additional deterioration is found below the surface.

Materials Checks

- Verify that the mix design for the material being supplied meets the criteria of the contract documents.
- Verify that full-depth repair material has been sampled and tested prior to installation, and is not contaminated.
- Verify that dowel bars and tied bars meet specifications and are properly coated with epoxy (or any other approved material). The surface must be free of any surface damage.
- Verify that epoxy meets specifications.
- Verify that bond-breaking board (typically asphalt-impregnated fiberboard)meets specifications.
- Verify that sufficient quantities of materials are on hand for completion of the project.

Equipment Inspections

<u>PCC Removal Equipment</u>

- Verify that PCC saws and blades are in good condition and of sufficient diameter and horsepower to adequately cut the required full-depth repair boundaries.

- Verify that all the equipment required for existing PCC removal is on-site and in proper working order and of sufficient size, weight, and horsepower to accomplish the removal process.

Full-Depth Repair Area Preparation Equipment

- Verify that the plate compactor is working properly and capable of compacting subbase material.

- Verify that gang drills are calibrated, aligned, and sufficiently heavy and powerful enough to drill multiple holes for dowel bars.

- Verify that air compressors are properly functioning.

- Verify the epoxy retention disks that fits tightly around the dowel bars and tie bars are available.

PCC Placement and Finishing Equipment

- Verify that handheld PCC vibrators are the proper diameter and operating correctly.

- Verify that all floats and screeds are straight, free of defects, and capable of producing the desired finish.

- Verify that sufficient polyethylene sheeting is readily available on-site for immediate deployment as rain protection of freshly placed PCC, should it be required.

Weather Requirements

- Verify that ambient air and PCC surface temperatures are within recommended range for PCC placement.

- Full-depth repairs should not proceed if rain is imminent. Full-depth repair that have been completed should be covered with polyethylene sheeting to prevent rain damage.

Project Inspection Responsibilities

PCC Removal and Cleanup

- Verify that the boundaries of the removal areas are clearly marked on the pavement surface and the cumulative area of the pavement to be removed is consistent with quantities in the contract documents.

- Verify that the full-depth repair size is large enough to accommodate a gang-mounted dowel drilling rig, if one is being used. Note: The minimum length (longitudinal direction) of full-depth repair is 6-ft.

- Verify that full-depth repair boundaries are sawed vertically the full thickness of the pavement.

- Verify that existing PCC is removed using either the break-up or lift-out method, minimizing disturbance to the base or subbase as much as possible. Note: The Saw cut and lift method is preferred to jackhammer removal.

- Verify that after existing PCC removal, disturbed base or subbase is recompacted, or preferably replaced entirely with PCC.

- Verify that PCC adjoining the full-depth repair is not damaged or undercut by the existing PCC removal operation.

Full-Depth Repair Preparation

- Verify that dowel holes are drilled perpendicular to the vertical edge of the existing PCC using a gang mounted drill rig.

- Verify that drilled holes are thoroughly cleaned using compressed air.

- Verify that approved epoxy is placed in dowel holes, from back to the front.

- Verify that proper epoxy-retention disks are available for the dowel bars. These must fit reasonably tightly around the dowel bars. It is absolutely essential that these be used to anchor dowel bars.

- Verify that dowels are inserted with a twisting motion, spreading the epoxy along the bar inside the hole. An epoxy-retention disk must be used to keep the epoxy from seeping out of the hole. Ensure that the disks are placed against the face of the bar as the bar is inserted with a twisting motion. This will guarantee that the epoxy surrounds the face of the PCC slab tightly.

- Verify that dowels are installed in transverse joints to the proper depth of insertion and at the proper orientation (parallel to the centerline and perpendicular to the vertical face of the saw cut excavation) in accordance with specifications. Typical tolerances measured perpendicularly to the sawed faced are ¼-in misalignment per 12-in of dowel bar length.

- Verify that tie bars are installed at the proper location and to the proper depth of insertion in accordance with contract documents. When the length of the longitudinal joint is 15-ft or greater, tie bars are typically installed in the manner used for dowels. When the length of the longitudinal joint is less than 15-ft, a bond-breaker board is placed along the length of the full-depth repair to isolate it from the adjacent slab.

- Ensure that tie bars are checked for location, depth of insertion, and orientation (perpendicular to centerline and parallel to slab surface).

<u>Placing, Finishing, and Curing PCC</u>

- Verify that the fresh PCC is properly consolidated using several vertical penetrations of the PCC surface with a handheld PCC vibrator.

- Verify that the surface of the full-depth repair is level with the adjacent slab using a straightedge or vibratory screed in accordance with contract documents.

- Verify that the surface of the fresh full-depth repair is finished and textured to match adjacent surfaces.

- Verify that adequate curing compound is applied to the surface of the fresh PCC immediately following finishing and texturing in accordance with contract documents. Best practice suggests that two applications of curing compound be applied to the finished and textured surface, one perpendicular to the other.

- Ensure that insulation blankets are used when ambient temperatures are expected to fall below 40°F. Maintain blanket cover until PCC attains the minimum strength required. Wet burlap must be used when PCC is placed in hot temperatures. Other curing techniques such as fogging can be used to maintain a moist/wet surface.

Common Problems and Solutions

Undercut spalling (deterioration on bottom of slab) is evident after removal of PCC from full-depth repair area:

- Saw back into adjacent slab until sound PCC is encountered.
- Make double saw cuts, 6 inches apart, around full-depth repair area to reduce damage to adjacent slabs during PCC removal.
- Use a carbide-tipped wheel saw to make pressure-relief cuts 4 inches wide inside the area to be removed.

Lifting out existing PCC damaged adjacent slab:

- Adjust lifting cables and re-position lifting device to assure a vertical pull.
- Re-saw and remove broken section of adjacent slab.
- Use a forklift or crane instead of a frontend loader.

Slab disintegrates when attempts are made to lift it out:

- Complete removal of full-depth repair area with backhoe or shovels.
- Angle the lift pins and position the cables so that fragmented pieces are bound together during lift out.
- Keep lift height to an absolute minimum on fragmented slabs.

Full-depth repair area becomes filled with rainwater or groundwater seepage, saturating the subbase:

- Pump the water from the full-depth repair area, or drain it through a trench cut into the shoulder.
- Re-compact subbase to a density consistent with contract documents, adding material as necessary.
- Allow small depressions in subbase to be filled with aggregate dust or fine sand before full-depth repair PCC material is placed.
- Permit the use of aggregate dust or fine sand to level small surface irregularities (1/2 in or less) in surface of subbase before full-depth repair PCC is placed.

Epoxy around dowel bars flows back out of the holes after dowels are inserted:

- Pump epoxy to the back of the hole first.
- Use a twisting motion when inserting the dowel.
- Add a epoxy retention disk around the bar to prevent epoxy from leaking out.

<u>Dowels appear to be misaligned once they are inserted into holes:</u>

- If misalignment is less than 1/4 in per 12 in of dowel bar length, do nothing.
- If misalignment is greater than 1/4 in per 12 in of dowel bar length on more than three bars, re-saw full-depth repair boundaries beyond dowels and re-drill holes.
- Use a gang-mounted drill rig referenced off the slab surface to drill dowel holes.

#5 Partial Depth Repair Case Study for Minnesota and Other Leading States

Abstract

This document presents a case study for partial depth repair (PDR) for the lead State of Minnesota as well as specs and experience from Georgia, Missouri, Washington, California, and Utah. PDR technology has been used for many years to repair a concrete slab's upper surface that has experienced spalling for various reasons. Nevertheless, there have been a number of failures of PDRs over the years on projects across the country. This case study is based primarily on specifications and interviews with lead State and contractor staff relative to their experience with PDR, focusing on how to provide long-term performance.

The first key to long-term performance is to limit usage of PDR to the appropriate slab locations and conditions. Minnesota and other States recommend that PDR should only be used in joints and cracks that exhibit spalling or disintegration in the upper one-half slab thickness. Minnesota also found that PDRs do not last very long in wheel paths unless the underlying portland cement concrete is very sound, or if the PDR is on a low volume highway. It is recommended to extend the PDR beyond the wheel path to give it more structural stability. Bottom line: Spalling of the lower half of slabs cannot be repaired successfully by PDR and must be addressed using full depth repair (FDR).

A second key is the detail and effectiveness of the specifications and special provisions of the State, along with the inspection/acceptance effectiveness. Substantial information is provided regarding States' specs for PDR, including field layout, removal of concrete, forming of joints, and inspection/acceptance. A third key aspect is the PDR material itself. Conventional cementitious mixtures have generally shown good long-term performance, while many States report relatively poor performance of some proprietary early opening materials. States are in search of more durable, reliable, and rapid opening to traffic material for PDR.

PDR has produced a significant increase in the life of spalled jointed concrete pavements whenever the location is proper, specifications are effective, materials are durable, and inspection/acceptance procedures are followed. In these cases, a life of 10 to 20 years has been achieved in most States. However, installation must go properly. There can be few deficiencies, or the PDR service life will be reduced. This makes a case for increased training of State and

contractor staff. Warranties (even 30 days, as in Minnesota) may be an effective procedure for achieving better PDRs.

1. Introduction

Partial depth repair (or PDR) is a well-established technique applied to an existing concrete pavement that has spalls along joints and cracks. Nevertheless, there have been a number of failures of PDR over the years on projects across the country. Interviews with State and contractor personnel indicated that one key to successful PDR is to limit their usage to the appropriate slab locations and conditions. These experts made clear that the key to success is to only place PDR in joints and cracks that exhibit spalling or disintegration in the upper one-half the slab thickness. Spalling of the lower one-half of slabs cannot be repaired successfully by PDR and must be addressed using full depth repair (FDR) technology. Causes of spalling and appropriate repairs are summarized as follows:

- **Incompressibles infiltrate into the joint or crack** during cooler weather and then are compressed during warmer weather, creating shear forces that often spall off the upper portion of the slab along the joint or crack. This is the type of spall that can be successfully repaired using PDR technology.
- **Misalignment of dowel bars or high tie bars along the joint or crack**. During temperature changes, a misaligned bar create shear or tension stress in the concrete that eventually lead to a shearing of the concrete. Experience has shown that if a spall caused by misaligned dowel bars is repaired with a PDR, the PDR will fail quickly. A FDR is the appropriate repair for spalling caused by misaligned dowels.
- **Concrete durability.** Frost areas experience deterioration of the lower portion of slabs first. The extent cannot be seen on the surface until it becomes significant and causes spalls of the surface. This type of distress and location in the slab can only be identified by coring or nondestructive testing such as ground penetrating radar. If the durability deterioration extends below mid-depth of the slab or extends too far from the joint horizontally, PDR is not appropriate and an FDR should be specified. PDR is appropriate if the durability deterioration occurs in the upper portion of the slab. Areas where frost is not significant typically do not experience lower slab deterioration.

This case study focuses on PDR in Minnesota but also includes information from Missouri, Utah, Washington, Georgia, and California. The Minnesota specifications and acceptance procedures have evolved over many years into a reliable and cost-effective process that has produced long-term performance. The other States' PDR specifications are also excellent, and each adds some valuable information. This report first describes the pre-PDR design considerations, then specifications and construction, inspection and acceptance are next, and finally performance is described.

2. Pre-PDR Considerations

Minnesota and the other States typically perform PDR to provide a long-term repair to localized concrete spalling and deterioration at transverse joints, longitudinal joints, and cracks to alleviate a roughness, maintenance, and/or safety problem. The key question that should be asked first is, how long does the agency want the PDR to last? This case study focuses on the longer term PDR requirements (e.g., 10+ years). Two key pre-PDR construction considerations include an appropriate existing pavement condition and the establishment of proper boundaries.

Appropriate Existing Condition for PDR

Minnesota evaluates transverse joints and cracks throughout the depth of the slab. Coring and visual inspection is required to ensure there exists sound portland cement concrete (PCC) in the lower portion of the slab. If there is significant lower slab deterioration (lower portion of core falls apart), then PDR is not placed. If a field change from PDR to FDR is made after work has begun on the PDR, Minnesota will pay 40 percent of the PDR to give credit to the contractor for work accomplished. Minnesota has also found that if the jointed plain concrete pavement (JPCP) or jointed reinforced concrete pavement (JRCP) is a high volume road, PDRs do not perform well, especially in the truck wheel paths. Thus, for this situation, PDR is not recommended and FDR is typically used. If PDR is used, it is recommended to extend the PDR beyond the wheel path to provide more structural stability.

Georgia only uses PDR if the spalling is less than 4 inches deep into the slab; otherwise, FDR is used. Missouri repairs joint and crack spalling and high steel for JRCP. Missouri also checks to see if the slab is deteriorated < ½ thickness. If the deterioration extends deeper into the slab, FDR is specified. A strong consensus among States and contractors was that PDRs work well with typically good service life when they are applied to the appropriate distress conditions.

California utilizes PDR for spall repair to restore localized surface deterioration in joints, cracks, or miscellaneous areas within the upper 1/3 of the concrete slab depth. Spall repair is commonly used to repair isolated spalling between 6 inches and 6 feet long caused by incompressible material in joints or cracks, localized areas of weak material from poor consolidation, curing, finishing practices, and joint inserts. California makes clear that deterioration beyond 1/3 slab thickness should be repaired using FDR. (California DOT, 2015) A contractor comment covering several States is that the State marked locations are very often not sufficient to cover all of the deterioration. Thus, the quantity of PDR work is underestimated on many projects; creating project performance issues since not everything that needs PDR get repaired. Inspection staff should be checking PDR's for additional deterioration before the Contractor places backfill.

Marking of PDR Boundaries

MnDOT does the coring to determine if a pavement is a good candidate for a CPR project. If it is a CPR candidate, then the cores are used to estimate PDR/FDR quantities. If MnDOT finds the lower portion of the PCCP is unsound they will then skew the repairs more towards FDRs. A sample graphic and instruction sheet for PDR is shown in Figure 26.

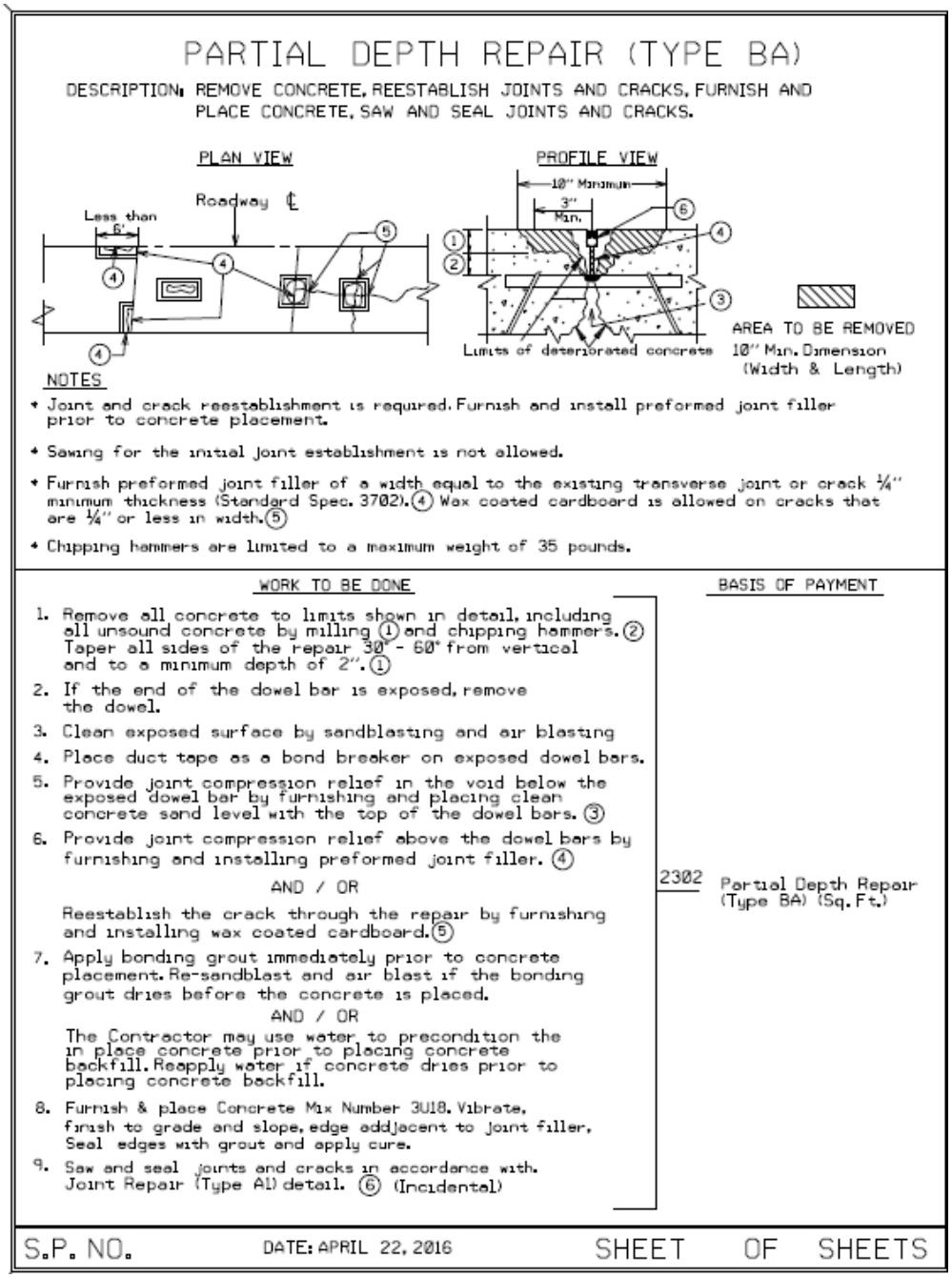

Figure 26. Minnesota PDR BA diagram for milling. (MnDOT, 2017)

Georgia (as does MN) sounds each joint with a visual defect to help establish boundaries. The pavement surfaces along the sides of each joint are struck with a hammer, chain drag, or similar tool to detect unsound concrete that sounds flat or hollow. Georgia marks the limits of defective areas by a rectangle 2 inches beyond the outer limits of unsound concrete. Missouri selects the repair boundary that is 1 to 2 inches beyond the visible surface deterioration. Washington uses both visual and sounding techniques. A rectangular boundary is sawed at least 3 inches beyond the deterioration. Utah utilizes visual and sounding and then saws the boundaries 6 inches beyond the deterioration to ensure sound concrete. California determines the unsound concrete by sounding (dull, hollow sound) using a hammer, steel rod, or dragging chain. At least 2 inches are required beyond the unsound limits, but 5 inches are required beyond the edge of visible spalling.

Several comments were received regarding the size of the PDR. Due to the many complications involved, only general guidelines can be provided:

- Some believe that, if a PDR is too small, it often seems to fail more readily than a larger repair. One contractor recommended that the minimum partial depth repair is 12x12 inches and 2 inches deep. The length allowed along a longitudinal or transverse joint would be based upon the type and extent of deterioration. If the area is deteriorated far into the slab for 30 percent or more of the length of the patch area, then an FDR is recommended. Otherwise, there is no restriction on length. Minnesota's experience with short PDR placed in wheel paths failing sooner and their recommendation to extend (lengthen along the joint) the repairs to reduce the failure potential is an example of this phenomenon.
- PDR at joints needs to be large enough and deep enough (most States require >2 inch depth, and one contractor requires 2.5 inches) to provide sufficient mass to bond and perform over a long time period under heavy repeated loads.
- More PDR failures were observed in larger sized repairs. For example, some Washington State staff suggested limiting the size to 3x4 feet for cementitious materials. Some States have limits on sizes for specific materials. For example, Utah has a maximum 9 square feet for elastomeric PDRs.
- Width is another story, relates a contractor; again, it should be based on the type of deterioration. Repairs that are 3 to 5 feet wide should be no problem if the deterioration was limited to 1/3 the thickness of the slab. The width of the PDR should be at least 5 inches from the joint. The exception would be if an elastomeric type material was used, as these materials do not work well when grinding is specified. These types of repairs should be limited in length and width to minimize diamond grinding problems. If the repair is going to be full depth of the slab, then the elastomeric material does not work well and is too expensive.

3. PDR Specifications

Minnesota and all of the other States appear to have excellent specifications and special provisions for PDR. Contractors who have worked in these States affirm they are reasonable and effective. Table 13 summarizes the various specifications, special provisions, and other documents from these States. Each of the key topics of PDR construction will be covered in this section.

Training of State and Contractor Staff

Training was a major concern for PDR for all of the States and contractors included in the survey. If anything goes wrong, the PDR is likely to fail early. Pre-construction training in Minnesota was very beneficial and allowed the engineers to explain the mechanics behind the repairs. Training is an important aspect for both inspectors and contractors personnel. Inexperience and turnover have been an issue at the State level.

Minnesota used to require inspection training that utilized YouTube videos documenting the proper methods to perform and inspect various repairs for training inspectors, including PDR (being updated 2017). Georgia likewise conducted many training courses for State and contractor personnel in the 1980s and 1990s, and it paid off greatly in terms of long-lasting PDRs. Training will especially help with situations where everything doesn't follow the specifications exactly. Utah indicated that the success of the PDR depends on the training of the designers, inspectors, and contractors. Thus, there is a consensus for pre-construction training with regard to PDR, as there are so many steps involved.

Table 13. Summary of State Specifications for Partial Depth Repair (PDR)

State	Specification/Documents
Minnesota	MnDOT 2302 Special Provision MnDOT 2302 Special Provision and 3105
California	Section 41-1: Spall Repair (Polyester Concrete) Section 41-4: Spall repair (fast-setting concrete for pre-overlay repair, anticipated life <5 years) California "Concrete Pavement Guide"
Georgia	GA 451, 435
Missouri	MoDOT 613.20. Standard Drawing 613 (Sheet #2)
Washington	WSDOT 5-01 Cement Concrete Pavement Rehabilitation
Utah	UDOT 2751 Partial Depth (cementitious material). UDOT 2751S Partial Depth (Hot Applied Proprietary Flexible Material).

Removal of PCC in PDR Area

Historically, the most common procedure to remove deteriorated concrete is to saw the boundaries of a rectangular area and then use light chipping hammers to remove the concrete. Today, Minnesota (who pioneered this procedure), Missouri, California, and others either specify or allow milling of the area for a PDR. Minnesota began using this innovative approach to more quickly and cost-effectively remove deteriorated concrete starting in the 1980s and has used this procedure ever since. The concrete is removed by milling and typically some minor chipping hammer (< 35 lbs) work needed after the milling. This is especially true at the intersection of transverse and longitudinal joints. Milling machines used for concrete removal are equipped with a device for stopping at preset depths to prevent damage to the dowel bars. The maximum partial depth thickness is recommended to be 1/3 of slab thickness. A thickness of 1/2 slab thickness can be used for areas with material-related issues. The area is cleaned by sandblasting and air blasting, which by all accounts is critical to strong bonding. A minimum of 2 inches is generally recommended.

Missouri also removes spalled concrete by milling, with good success. Then either sand or shot blasting is used to clean the surface. The maximum depth of removal is ½ slab thickness. If deeper, an FDR is specified because PDR without a solid base to bond to tend to deteriorate.

Washington uses a traditional vertical saw cut to a minimum depth of 2 inches. PCC material is removed to sound concrete, which is about mid-depth of the slab. A maximum of 30 lb chipping hammer is required in this operation. The surface is sandblasted and all loose material is removed. Spall repair is not done in dowel areas.

Utah specifies sawing the boundaries and use of a maximum 30 lb jackhammer. Sand/water blasting is used to clean the surface. The maximum depth of removal is ½ of the slab thickness. Sand or water blasting with 2,000 psi is used to clean the surface. For the hot applied elastomeric repair material, Utah specifies a maximum of 9 square feet for each PDR. This limit is critical to good performance and to make it possible to diamond grind the hot applied elastomeric PDR.

Georgia saws the rectangular marked areas with vertical faces at least 2 to 3 inches deep. They remove unsound material within the sawed area with a maximum 30 lb light chipping hammer. If unsound material is >4 inches deep, a 6-ft FDR is placed.

Forming Joint in PDR

All States reestablish the existing crack that runs through or alongside the PDR and consider this a critical step to avoid future PDR failure. There just simply has to be complete relief along the repair for the joint or crack to work, or the repair will fail. Minnesota reestablishes the existing

crack with wax-coated cardboard and joint filler, then saw cut the joint to ensure it is as wide as the rest of the joint. Missouri maintains the existing joint with ≥ ¼-inch fiber board or other material before placing concrete patch material. California uses either closed cell foam or rigid foam material insert, which is stable in the presence of the freshly mixed polyester concrete.

Georgia places a 0.25-inch-wide piece of closed cell polyethylene foam shaped to fit the saw cut in the joints bordering the repair areas. Polyethylene foam must be supported in a straight line. Washington forms the new joint to the same width as the existing joint or crack. Compressible joint material is placed into existing joint 1 inch below the depth of repair. Utah maintains the existing joint with fiber board or other material before placing concrete patch material. However, the existing joint or crack is not sealed for repair with elastomeric (Flexcrete) material.

A Utah contractor stated that there is always a foam/fiber insert problem of maintaining a straight up and down material that goes deep enough through the patch material. The insert just does not stand up and slumps or bends over, creating a very poor or no joint at all. This contractor prefers to green saw the joint on all their projects. This is a much more reliable methodology to getting a proper joint. This will work for a straight line like a joint, but a typical crack cannot be followed with a saw. If placing patches in cool weather, the joint should be sawed thicker so that it will not close up in the hot summer and spall the repair.

Partial Depth Repair Material

States use both conventional concrete and early opening rapid set materials. Minnesota uses Concrete Mix 3U18 Type B repair placed, vibrated, finished to slope, edges sealed with grout (when this step is skipped the repairs tend to pull away from the edges and a premature failure develops), and a cure applied. They occasionally use ultra-high early strength (2 to 4 hours) materials (e.g., CTS, Futura 15 are both cementitious products), packaged, dry, non-shrink, rapid hardening concrete material for backfilling. In other words, Because MN has the 30-60 deg. tapered edges from milling the slurry helps the outside taper edge from drying/curing issues. The following new paragraph will be added to the 2018 MnDOT specification.

> After radius edging all inserts, final finishing and providing broomed surface texture; apply a heavy application of bonding grout/slurry at the surface interface (around the perimeter) of the Type B repair and the in place concrete pavement. Position the grout/slurry band so 1 inch is over the in place pavement and 3 inches is located over the newly placed repair.

California repairs spalls using polyester concrete with a high molecular weight methacrylate (HMWM) bonding agent. Polyester concrete is being used for spall repairs (as well as dowel bar retrofit). Contractors indicate that polyester concrete does a great job and holds up well. A

contractor states that the polyester concrete often outlasts the surrounding original concrete slab. California's concrete manual describes polyester concrete as follows:

> Polyester concrete consists of an unsaturated isophthalic polyester-styrene copolymer resin binder and dry aggregate. A silane coupler is used to increase the resin bonding strength, and a high-molecular-weight methacrylate (HMWM) bonding agent is applied to penetrate microcracks in the substrate surface and increase shear strength at the bond interface.
>
> Despite higher cost, polyester concrete is preferred when compared to fast-setting concrete materials for most applications due to generally superior performance over a wider range of conditions. Polyester concrete cures rapidly, developing high compressive strength and good concrete adhesion for placement over a wide surface temperature range between 40 and 130 °F. The polyester resin gel time can be adjusted for conditions anticipated in the field by adjusting the initiator percentage according to manufacturer recommendations.
>
> Minimum polyester concrete material property requirements for viscosity, specific gravity, elongation, tensile strength, styrene content, silane coupler, saturated surface dry bond strength, and static volatile emissions are in 2010 Standard Specification Section 41-1.02C. (California DOT, 2015)

California is evaluating proprietary materials (e.g., Fastpatch, Fibercrete, Techcrete). These materials are currently used on a case-by-case basis using non-standard special provisions.

Cleaning of the PDR surface area is critical to achieve strong bonding. States generally require sandblasting or high-pressure water blasting that is followed by compressed air blasting to clean the exposed PDR concrete. If this step is not performed properly, the PDR will debond and experience a short life.

Utah participated in a Strategic Highway Research Program (SHRP) H-106 study on PDR, where several material types were installed on I-15 in 1991 and monitored for 7 years. These PDRs were properly installed (similar to the concepts described herein) by experienced contractors as observed by the researchers. (ERES, 1999) The Utah JPCP was sound concrete with no underlying deterioration at the joints, and spalling was in the upper one-half of the slab. Results showed that most of the repair materials performed exceptionally well, with 100 percent survival rate, including Type III PCC, Set 45, Five Star HP, MC-64, Sika Printo 11, and PERCOL FL.

This same experiment was repeated in three other States, and Arizona and South Carolina had similar results—both had no underlying concrete deterioration. Results from Pennsylvania

showed a much lower PDR survival rate (35 to 80 percent) at 7 years. Overall, these results indicate the level of success possible when existing concrete is durable and that good construction inspection and knowledgeable contractors perform the work.

Utah now uses a proprietary elastomeric material (Utah Spec Section 02751S), hot-applied flexible polymer modified patch material (e.g., Crafco "TechCrete"). These materials have been found to reliably remain in the hole over time, but the downside is that they settle under repeated loading, causing a permanent deformation creating roughness within 2 to 7 years. These PDRs then have to be topped off to reduce roughness. If the JPCP will be diamond ground in the future, special precautions must be taken as described by the State and the manufacturer. Utah limits the size of the repair to < 9 square feet and specifically uses a special mixture that includes aggregates mixed with the resin to stiffen the mix and make it more possible to diamond grind. One manufacturer procedure for producing an appropriate diamond ground surface on TechCrete patches consists of adding 20 to 30 percent aggregate into the resin for the main layer and then applying the surface aggregate in a single layer over the TechCrete surface for a grindable surface. (Crafco, 2015) See the Diamond Grinding Case Study report for more on this topic.

Washington has used rapid strength prepackaged cementitious repair materials in the past (with some poor performance) but is now transitioning to polymer concrete products looking for improved performance. Washington has looked at several polymer type products, utilizing either epoxy or polyester resin binders. The benefits of polymer concrete over cementitious repair products include fast cure times, improved bond strength, greater elongation properties, and no wet or special curing requirements. These benefits should result in faster return to traffic times and fewer repair failures.

Regarding use of a bonding agent, a contractor in Washington believes that this is more problematic than helpful. For example, if epoxy sets up before the patch material is placed in the hole, the bond will not be good resulting in patch early failure. A bonding agent adds another dimension PRD that can cause major problems. A surface that is saturated surfaced dried is good for bonding.

Georgia "Repair Method 1" includes a 24 hour accelerated strength concrete. The concrete surface areas are coated with a film of Type II epoxy. PCC is placed in repair while the epoxy is still tacky. "Repair Method 2" is a rapid setting patching material. The operation is performed based on manufacturer recommendations.

Minimum Time Open Traffic

For conventional concrete PDR, the following opening strengths or times are required:

- Minnesota permits opening to traffic after 12 hours and > 3,000 psi compressive strength.
- Georgia requires >2,500 psi compressive strength for conventional concrete.
- Missouri requires > 2,000 psi strength achieved for conventional concrete.

For proprietary or rapid set materials, the following are required:

- California requires for polyester concrete at least 2 hours after the time of final setting under ASTM C403/403M.
- TechCrete (Crafco), which is placed hot, is used by Missouri and Utah, and these States require just the time to cool down.
- States follow manufacturer recommendations for other proprietary materials.

4. Inspection/Acceptance

The inspection and acceptance process for PDR focuses on observations and testing:

- Proper marking and then milling or saw cutting boundaries and locations.
- Removal of deteriorated material but not damaging underlying material.
- Cleaning of the repair area. Sandblast is highly recommended. A contractor from Minnesota states that there is no other good way to clean and prep for a PDR.
- Sealing off the existing joint or crack to prevent repair material from infiltrating.
- Applying bonding material where applicable.
- Forming of the joint through the repair. <u>This is critical and, if not done properly, will cause failure of the PDR</u>.
- Placing repair material.
- Proper curing of patch material surface.
- Time to open to traffic.
- 30-day warranty period (Minnesota).
- Early cracking of PDR. Georgia and some other States require the removal and replacement of the PDR if cracked.
- Check delamination of PDR from existing concrete through testing. Minnesota, Missouri, and Washington tests for bonding of PDRs using chains or other devices. Georgia occasionally tests for bonding of the PDR. These States require the removal and replacement of the PDR if debonding is indicated. Missouri checks repairs for debonding 30 days after construction, which in effect is a warranty without being called one.

Thus, the observation of the specified process being followed plus testing for a delamination of the PDR are the keys to inspection and acceptance of PDR. Minnesota believes that the check for bonding is extremely critical, because no PDR will remain in place that becomes debonded.

Incentive/Disincentive

None of the States use incentives/disincentives for PDR. Minnesota, however, has a 30-day warranty. Minnesota feels that, if a PDR is going to have a bond failure, it often happens very quickly, and the 30-day warranty will often catch this occurrence. Missouri and Utah also agreed that a short-term warranty may be a valuable idea for PDR.

In addition, it is conceivable that an incentive/disincentive specification for bonding of PDRs could be developed. Bonding of the PDR requires that attention be paid to every step in the process of producing a good PDR.

If a high percentage of sounded PDRs indicate full bond, an incentive could be paid. Conversely, a disincentive could be specified for a lower percentage of sound PDRs. A random sample of project PDRs within a section could be tested and the results used to establish incentive/disincentive pay scale versus percent of fully bonded PDRs.

5. Performance & Survival of PDR

Performance of PDR in Minnesota and the other States is summarized below.

Minnesota. Performance has been good when PDRs are installed properly in durable surrounding concrete. Service life depends heavily on existing pavement condition. Many pavements are >25 years old and have major concrete damage and deterioration (particularly in the lower portion of the slab at joints), which limits the life of PDR. Obviously, some of these repairs should have been FDRs to begin with, and then they would have shown good performance. If an existing concrete pavement is <20 years, then typically a 20-year life of PDR is possible. The most common failure modes are often related: debonding and cracking. However, having sound concrete below the PDR for at least ½ the slab depth is very important in Minnesota. Placement within the wheel path reduces the life of PDR. Overall, 10 to 15 years is typical for many PDRs in Minnesota.

Missouri: The key problem controlling life of PDRs is surface preparation and properly reforming joints or cracks. A 10- to 15-year life has been obtained when the specifications and procedures are followed. Milling to remove the deteriorated concrete has worked well.

California: Service life of the polyester PDRs has been good. Contractors stated that spall material (polyester concrete) often outlasts the surrounding original PCC.

Washington: In recent years, some projects have been unsuccessful due to deterioration of the PDR material. Causes include poor installation procedures (material delamination) and materials durability. Washington is evaluating new materials, including polymer concretes.

Utah: Cementitious repairs consisting of concrete with Type III cement to Set 45 and others materials placed in the 1991 SHRP study showed 100 percent survival over 7 years. The mean life of all these PDRs on I-15 north of Salt Lake City probably had a mean life more than double this life (e.g., 14+). This is partly due to durable existing concrete slabs and very good inspection and contractors placed the PDRs. Hot elastomeric materials have been used extensively for PDR of concrete pavement in recent years. The most significant distress associated with these PDRs is rutting or settlement of the hot applied elastomeric material over a 2- to 7-year period that causes roughness and requires re-topping of the mixture, which requires reheating the surface first so that topping aggregate will bond. This is not a long-term PDR.

Georgia: PDR performs very well when placed properly by following the construction and materials specifications. The expected life of PDR is 20+ years when this occurs, and much shorter otherwise.

Overall, the performance of PDRs has been highly variable, both within and between States. If properly installed using the procedures described for Minnesota and the other States surveyed, and placed by knowledgeable contractors, a PDR typically lasts 10 to 15 years or more. However, many times something goes wrong and the PDR ends up with a short life of < 5 years. If a PDR is placed in a situation where there is surrounding concrete deterioration, then it will not last long, and an FDR should have been placed. Basically, everything has to go properly or the PDR will fail early.

6. Summary of PDR

The PDR technology has been around for a long time, but performance has been highly variable. To be successful and perform 10+ years, a PDR must meet the following requirements:

- **Sound concrete both surrounding and below the PDR.** Otherwise, early failure will occur. FDR should be placed if sound concrete is not available.
- **Proper milling and removal or sawing and removal of deteriorated material** must occur. In Minnesota, Missouri, and elsewhere, milling has proven to be a rapid and cost-effective process.
- **Cleaning (sandblasting) of the repair area** and placement of an effective bonding agent if specified.
- **Sealing of any existing joint or crack** below or along the side of the PDR is required to prevent material from infiltrating the joint or crack and causing future failures.
- **Forming of the joint/crack** either through or at the edge of the PDR properly.

- **Proper curing** of the PDR to ensure a durable surface. Minnesota has found very poor performance with standard water-based cures for PDR. Linseed cure is highly recommended by MnDOT.
- **PDR material must be durable and long lasting** and not shrink significantly. Regular concrete has been found to be the most durable material. Rapid setting and other specialty materials carry with them the benefits of rapid curing and early opening but serious problems associated with durability (cracking and disintegration) or rutting and roughness of elastomeric flexible type material.
- **Inspection is in part observational** and must be monitored sufficiently. Inspection is also testing for bonding between the PDR and concrete slab is an effective and needed acceptance procedure. Testing for bonding is an effective and needed acceptance procedure that has proven effective in several of the States. **Warranties (e.g., 30 days)** are believed to be an effective approach to achieving improved quality and longer lasting PDRs.
- **Service life** has been highly variable, ranging from < 5 years (due to surrounding deteriorated concrete and non-durable PDR materials) to 10 to 20 years (when everything goes right). Often, the short service life of a PDR can be attributed to cases in which an FDR should have been placed. In addition, there have been durability problems with some rapid early opening proprietary materials.

PDR has produced a significant increase in the life of spalled JPCP and JRCP in the States interviewed whenever the PDR location and selection was proper, the specifications were followed, materials were durable, and inspection/acceptance procedures were followed. In these cases, a life of 10 years to 20 years has been achieved. However, everything must go right. There can be no deficiencies, or failure of the PDR with a short service life will occur. This points to the importance of just-in-time pre-construction training for the State and contractor staff prior to start of work.

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#6 Slab Stabilization Case Study for Missouri

Abstract

A case study for slab stabilization for the lead State of Missouri is presented. This document also brings in the specifications and experience of Georgia and contractors from these States. Slab stabilization is the pressure insertion of a highly flowable material beneath the slab or stabilized base to restore the support beneath transverse and longitudinal joints that has been eroded away by repeated heavy axle loads, poor joint load transfer efficiency (LTE), excess free water, and erodible materials in the base, subbase, and subgrade. Slab stabilization is not used to significantly raise the slab, but to restore support. This case study is based on specifications and interviews with the States and contractor staff as well as past research studies focusing on restoring and providing long-term slab support.

Four key aspects of successful slab stabilization were identified. **The first key is to limit usage to the slab locations where deflection testing indicates loss of support**. Missouri and Georgia use load deflection testing at slab corners to determine if erosion and loss of support has developed. Many slab corners do not develop loss of support along a project. The presence of joint faulting is an obvious sign that erosion has occurred and loss of support beneath the approach, the leave, or both slab corners. Faulting typically occurs at transverse joints without dowels and at working transverse cracks due to poor joint or crack LTE and high deflections.

The second key is the detail and effectiveness of the specifications for slab stabilization. Substantial information is provided from Missouri, Georgia, and other sources regarding their successful approaches and results.

The third key is the material that is used to pump beneath slabs, how well the deflections are reduced at the slab corners, joint and crack LTE is increased, and how long the material will maintain full support without pumping and eroding over the long term.

The fourth key is the inspection/acceptance procedures and their effectiveness. The best acceptance criterion is to conduct deflection testing of joints that were stabilization along the project to verify that full support has been achieved. This is a straightforward procedure using deflection testing at a specified corner with a range of loads and corresponding measured deflections that are plotted on a load versus deflection graph to establish that the best fit line passes near the origin. If not, the corner can be restabilized and tested to verify full support, which is the ultimate proof that the slab stabilization has worked.

1. Introduction

Slab stabilization through pressure injection of a flowable material beneath the slab or stabilized base has been conducted since the 1970s. It is applied to an existing concrete pavement that has developed loss of support due to erosion creating relatively thin voids (0.005 to 0.25 inches thick) beneath the slab or the treated base course. Some voids actually develop beneath a concrete treated base (CTB) or lean concrete base (LCB) when the layer is bonded to the concrete slab, while others form directly beneath the slab when the bond is broken or when an aggregate base course is in place. Other names for slab stabilization are subsealing or undersealing.

Slab stabilization included in Missouri and Georgia specifications strive not to lift the slab significantly, but simply to fill in the voids and restore full support beneath the slab. This is the same definition as provided by the American Concrete Pavement Association in 1994. (ACPA, 1994) Thus, slab stabilization is not the same as "slabjacking," which is used to "lift" a slab at a depression usually caused by subgrade settlement or bridge approach slab to restore the original profile of the highway. Slabjacking is a valuable and often used technique that provides profile restoration in specific locations such as bridge ends or various localized depressions along the project, but this technique is not discussed in this document.

Missouri and Georgia use diamond grinding to provide a very smooth profile after slab stabilization is complete. Utilizing an effective material may also reduce future erosion and joint faulting through increasing the load transfer efficiency (LTE), and this would be a significant long-term advantage. Projects in Texas and Pennsylvania have shown increased LTE after slab stabilization with polyurethane. (Brown & Reed, 2011 and 2014)

Why is loss of support such a destructive occurrence in pavement slabs? Most of the voids develop near slab corners at joints and cracks from high deflections, causing erosion and pumping. The loss of support causes larger and larger corner deflections (particularly at the leave corner) and critical slab stresses which lead to faulting of the joint, corner cracks, rocking slabs, and finally complete breakup of the slab. (Darter, Barenberg & Yrjanson, 1985) That distress can be repaired by filling the voids through proper slab stabilization and perhaps also performing dowel bar retrofit (DBR) to reduce future faulting.

Slab stabilization is usually performed in conjunction with other needed concrete pavement restoration (CPR) work such as full depth repair, partial depth repair, dowel bar retrofit, and diamond grinding. Slab stabilization is an important component of overall CPR if loss of support has developed beneath slab corners.

The primary cause of loss of support goes back to the design and materials used in the jointed concrete pavement projects built from the 1950s through the 1980s. Jointed plain concrete

pavement (JPCP) often did not include dowel bars, tied shoulders, non-erodible base courses, shorter joint spacing, and subdrainage. Jointed reinforced concrete pavements (JRCP) often did not include sufficient reinforcement to hold transverse cracks tight over time, tied shoulders, adequate sized dowel bars, and non-erodible base courses. In addition, truck traffic began to increase far greater than designers had anticipated over this time period. Pumping and erosion began to develop on many projects across the U.S., leading to early maintenance and rehabilitation starting in the 1970s. States and contractors developed slab stabilization procedures in the 1970s and have continued to improve them, along with the other CPR techniques, to extend the service lives of these deficient JPCP and JRCP.

In the 1980s, designs began to improve by including adequate sized dowels, tied portland cement concrete (PCC) shoulders, improved stabilized base courses, subdrainage, widened slabs, and shorter joint spacings. States also stopped building JRCP. All of these design changes reduced the potential for high joint and crack corner deflections leading to erosion of underlying materials. These new designs have performed far better and have required far less CPR-type work in Missouri, Georgia, and other States.

Missouri conducted its first full-scale slab stabilization project with polyurethane in 1999, but grout slurry usage goes back several decades before that. Missouri has completed several projects with polyurethane since 1999 with good success. Georgia also began slab stabilization with cementitious grout in the 1970s and completed many successful projects during the next 30 years.

Georgia began developing pressure grouting procedures in the 1970s that involved pressure injection of a slurry type grout mixture through holes drilled in the pavement into voids beneath the slab to stabilize and underseal JPCP. This spec was developed for older JPCP designs with 30-ft joint spacing, no dowels, aggregate-soil base course, asphalt shoulders, etc. Today's JPCP design is totally different and removes many of the high deflection and water entry problems (dowels, non-erodible base, 15-ft joint spacing, tied PCC shoulders, sealed joints, and thicker slabs). Slab stabilization is only rarely done today on Georgia JPCP projects because there is no significant loss of support.

This case study first describes the pre-slab stabilization design considerations, then specifications and construction, followed by inspection and acceptance, and finally performance.

2. Pre-Slab Stabilization Considerations

Pre-Construction Evaluation

Missouri conducts surveys 1 to 3 years ahead of construction to identify suitable candidates for slab stabilization. They consider visual conditions such as pumping and faulting as well as Falling Weight Deflectometer (FWD) testing on some projects at slab corners to determine if loss of support has occurred. The results from FWD testing can be used directly to estimate quantities of material, which is increased about 10 percent in the bid plan quantities to account for increased erosion.

The erosion process begins at slab corners (joints or working cracks) over time and with increased traffic, particularly under unfavorable design and climatic conditions. These include free water beneath the slab (if aggregate base) or sometimes the base (if cement stabilized) that flows under tremendous pressure when a heavy truck load passes over the joint and deflects the slab. Fine materials are eroded and pumped backwards (opposite to traffic flow) due to the way in which the load crosses the approach (upstream) side and then abruptly loads the leave (downstream) side forcing water backwards, carrying fine material from the base and/or subgrade. The high pressure erodes almost any type of material beneath the leave slab and forces it backwards under the approach slab with every axle load. If this process is not limited or stopped, serious joint faulting, from loss of support, will develop over a period of a few months or years, creating unacceptable roughness. As erosion proceeds, more and more loss of support under the leave side slab corner develops, increasing the bending stresses, often resulting in corner or diagonal cracking.

"Slab stabilization" specifically means to fill in any air void spaces that develop below the slab or treated base course to lower deflections of slab corners. Slab corners (especially the leave corner) are the critical location where deflections are always much higher and, thus, pumping and erosion is much higher.

If an effective slab stabilization process occurs and joint or crack support can be restored beneath the corner, and the stabilization material will not erode, then future faulting may be slowed. Stabilization can be combined with DBR and diamond grinding to prevent faulting and provide a very smooth surface for highway users for many years.

Appropriate Existing Condition for Slab Stabilization

Missouri JPCP is doweled and does not often develop loss of support and faulting at transverse joints, but transverse cracks sometimes develop that do develop faulting and loss of support. If some rocking slabs exist (where slab deflection can be observed as a heavy axle load passes over

the joint or crack), this becomes a prime candidate for slab stabilization. Missouri JRCP often develops working transverse cracks that fault and spall, and are potential candidates for slab stabilization. If erosion and pumping can be slowed through stabilization of the slab corners/joints and increased crack LTE, then the service life of these pavements will be extended.

Georgia looks for signs that pumping and joint faulting is occurring along the non-doweled JPCP. Of course, rocking slabs would be a clear clue of loss of support. Georgia considers deflection testing at slab corners the ultimate determination of loss of support beneath the slab. Significantly high deflections at the corners indicate a loss of support and a void beneath the slab.

Rapid Void Detection Procedure

The key indicator measured in Missouri and Georgia is the slab corner deflection along the project. Deflection data can then be used to determine if there is significant loss of support beneath the joint or crack that needs to be restored. This can be accomplished 1 to 3 years ahead of construction. The rapid void detection procedure is used by Missouri and is more fully explained in the 1993 AASHTO Guide, Part III, Chapter 3, Section 3.5. (AASHTO, 1993)

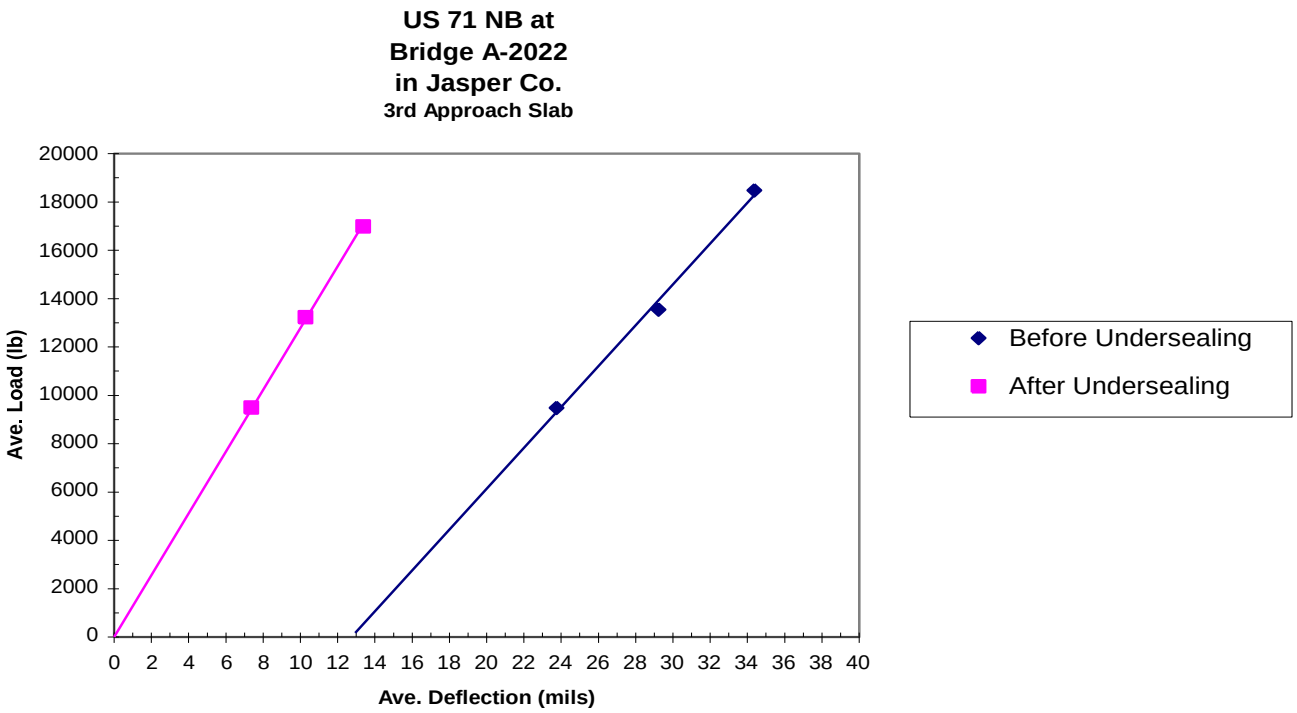
- The standard test method for measuring support in the past has been proof rolling, as defined in MoDOT Test Method T64. Currently, this is done by the FWD, which has many advantages. LTE can be easily measured and the load can be varied over a range and identify loss of support.
- FWD deflection testing should avoid midday summer hot temperatures to minimize the chance of joint lockup and slab curl. FWD testing must not be performed in freezing temperatures either.
- Each individual joint (approach and/or leave side, which typically has more loss of support) can be tested, or a statistical sample of joints can be tested with the FWD to give an estimate of the percentage of joints that exhibit loss of support. The procedure provides a loss of support result for each slab corner tested.
- FWD testing consists of placing the loading plate as close to the slab corner joints without crossing them and loading it at a minimum of three drop heights. Missouri uses 9, 12, and 16 kip loadings.
- The center of plate deflections are measured and used at each load level.
    - A simple plot is made of center of FWD plate deflections (X-axis) versus plate load (Y-axis) for a given joint corner and a best fit straight line is plotted.
    - If the extended line passes near or through the origin, as in Figure 27, then the corner is fully supported and does not require stabilization.
    - If the line extends through the X-axis, as shown in Figure 27, then the corner will likely have loss of support and that joint/corner is marked for stabilization. The

magnitude of the loss of support (representing the thickness of the void) depends on how far from the 0.0 deflection axis the line passes. Missouri's interpretation:

- If the best fit extended line passes < 0.030 inches along the deflection axis (X-axis), then the slab corner is fully supported and does not need to be stabilized from this standpoint.
- If the best fit extended line is > 0.03 to 0.07 inches along the deflection axis, then the joint becomes a marginal subsealing candidate.
- If the best fit extended line is > 0.007 inch, the corner becomes a strong undersealing candidate.

Some of the findings and conclusions from this field study are as follows (MoDOT, 2004):

> The FWD is an efficient tool to use for void detection under PCC pavement slabs, assuming the rapid void detection procedure is correct. Plotting the linear trend for load-versus-deflection before and after undersealing is simple. The assumptions about linearity, within the 9,000 to 16,000 (lb) range, appear to be accurate based on the plots.
>
> In 73 percent (16 of 22) of the slabs that were undersealed there was clear evidence in the plots that an improvement had been made. Two of the six slabs, whose load deflections had not improved, still had a change in deflection slope, thus indicating that some betterment had occurred. Another one of the six slabs may have been tested improperly on a cracked piece after undersealing. The remaining three slabs showed negligible changes for unexplained reasons, assuming that they had been properly undersealed….
>
> Although it is the primary intent of another study to evaluate the effectiveness of the polyurethane undersealing process, it can be concluded from the FWD tests that a positive improvement in support did occur under most of the slabs. It may be a more reliable process than grout undersealing, which, although cheaper, has in the past on occasions caused uneven filling of voids, because of its high viscosity, and created greater support problems than existed before.



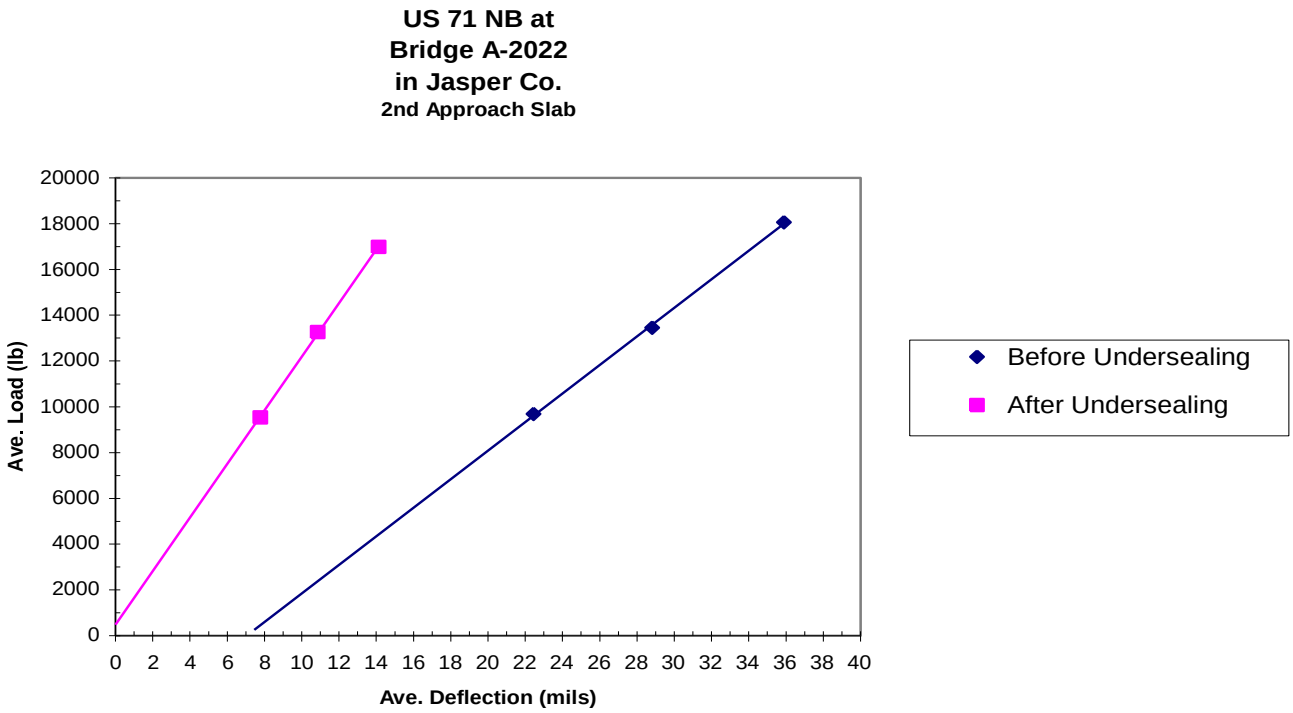


Figure 27. One Missouri slab corner tested by FWD at 9,000, 13,000, and 16,000 lb before and after slab stabilization showing impact of stabilization to restore support.

Estimation of Material Quantities

A large issue related to slab stabilization is the quantities needed to stabilize the slab. A few projects have been over pumped, raising slabs, and sometimes resulting in cracking. Too little material will not fill all the voids, and loss of support will still exist. The optimum amount of material required is what ensures that slab corner deflections are back to normal and that the load versus deflection line goes near the origin of the plot.

The various companies that perform slab stabilization work have their own ways of estimating quantities needed based on their extensive experience. Research conducted in the 1980s under National Cooperative Highway Research Program (NCHRP) Project 1-21 also derived a procedure to estimate cementitious grout quantities based on FWD deflection tests previously described. (AASHTO, 1993; Darter, Barenberg & Yrjanson, 1985, Darter et al, 1993).

Missouri Field Project Example

An example of the AASHTO procedure is provided using data from Missouri projects. (MoDOT, 2004) The objective of this study was to evaluate the efficiency of the FWD for determining the loss of support (or voids) under jointed concrete pavement slabs near bridge approach locations. Then, after slab stabilization of the slabs with polyurethane, the extent of loss of support was evaluated. The deflection improvement (reduction) after the slabs with voids had been undersealed with polyurethane was then assessed as referenced above.

Both of the joints in Figure 27 show that the slab stabilization process was very effective in reducing the deflection of the slab as the load vs deflection plots end up going directly through the origin of the plot indicating full support. These joints were all in the regular JPCP joints from the bridge approach joint.

Results from many other deflection testing and slab stabilization studies are available that show similar results. Typically, the leave corner shows loss of support while the approach does not, but both can show loss of support on some joints.

Georgia Procedure

Georgia developed the original procedure for slab stabilization in the 1970s. GA 450.3.05 requires deflection testing to detect slab corners having a deflection greater than 0.030 inch (non-doweled joints). Testing must be done between 3 am and 9 pm to avoid joint lockup in warm temperature. An 18,000-lb single axle load with dual tires is required. The truck axle (outside of outer tire) is placed 12 inches from the outer edge of the slab. Deflection measurements are made by gauges set on slab transverse joint corners on both the loaded and unloaded sides of the

transverse joint. The truck axle is moved to the approach joint and the loaded side gauge is read. The truck is then moved forward across the joint to the leave side and the forward gauge is read. This operation is repeated for each joint to be tested. Slab corners showing > 0.030 inch deflection are marked for grouting.

3. Slab Stabilization Specifications and Construction

Grouting Procedures, Grout Holes, and Materials

Missouri specifies slab lift measuring equipment that measures to 0.001 inch. Maximum lift allowed is 0.125 inch during undersealing. The contractor must provide the slab stabilization hole pattern 7 days in advance. Drilled hole diameter is a maximum of 1.5 inches. Holes are drilled through the slab only and not into the aggregate base.

Proof of full slab stabilization (undersealing) includes material seeping from adjacent joints and cracks, vertical slab movement, or other visual indications.

Materials used for slab stabilization over the years include cementitious grouts, asphalt, and polyurethane, as described below:

- **Cementitious grouts** have been used successfully since the 1970s by agencies such as Georgia. They include water, cement, flyash, and soil that flows under pressure beneath slabs and stabilized base courses to restore support. Usage of cementitious grout has been phased out in recent years due to its potential to erode over time.
- **Asphalt cement** commonly used for slab stabilization must have a low penetration and a high softening point. It must also have a viscosity suitable for pumping when heated to temperatures from 400 to 450 °F. A few agencies allow the use of asphalt cements for slab stabilization, including Missouri. Due to the extremely high temperatures there is some safety concern.
- **Expansive, high-density, two-component, water-resistant polyurethane material** is the primary slab stabilization material used today. (Brown & Reed, 2014) It is pressurized through a mixing nozzle where the two components combine at 120 °F and begin to set in 30 seconds. Ninety percent strength is achieved within 3 minutes, and ultimate strength is achieved in 24 hours. The material can flow into very fine voids and expands several times its liquid size, forcing its way into smaller or larger voids.

Comments from interviews on comparing cementitious grout to polyurethane indicate that the process control is better using the recommended injection pattern with smaller holes more tightly spaced together. By injecting polyurethane in smaller holes grouped together every 4 to 5 feet or so, there seems to be more uniform coverage across the slab bottom. Cementitious grout slurry

was usually forced through one or two larger holes, and the material didn't always flow where it should have, sometimes creating voids as the slab lifted. The success of grout slurry stabilization was more dependent on the skill of the practitioner. Another factor is getting the consistency of the grout just right—it cannot be too stiff or too watery, whereas the polyurethane chemical ratios are relatively easy to achieve.

Missouri permits both polyurethane materials and asphalt cement materials for injection beneath slabs to restore support. Both materials have apparently provided good results on a number of projects for years. A recent Utah application of polyethylene material for a non-doweled JPCP with joint faulting included three holes drilled per joint. The design included a 10-inch portland cement concrete (PCC) slab over a 4-inch bonded CTB. Holes were drilled 22 inches, and the subgrade was about 18 inches deep. The depth of the holes was required to get below the bonded CTB where the erosion was taking place.

Georgia Materials and Procedures

Georgia defines five types of cementitious grout mixtures for slab stabilization. Dry mixtures include a variety of cement, limestone dust, flyash, and fine aggregate. Georgia does not currently allow polyurethane injection. Contractors interviewed recommended that this material be added to the specifications, as it has performed well.

Georgia requires slab lift measuring equipment. The contractor must simultaneously detect movement of the two outside slab corners adjacent to a joint and the adjoining shoulder. Equipment must measure to 0.001 inch accuracy. Temperature must be above 35 °F and rising. Grouting must be stopped if the temperature is 40 °F and falling. A 1.5-inch hole is drilled in the corner of a slab with a deflection > 0.030 inch. A 1.5-inch test hole shall be drilled 18 inches from the transverse and shoulder joint. The test holes are be filled with water and observed. If the test holes drain water, the slab is pressure grouted as determined by the engineer. Plans show the hole pattern and pumping sequence. Drilled holes are extended approximately 8 inches deep beneath the bottom of the concrete slab. Slabs cracked during the undersealing operation are replaced.

During pumping, the lift measuring device is monitored to prevent excessive pumping and rapid lifting of slabs, or rising of the adjacent shoulders. Pumping shall cease when cavities or voids are filled within the expected volume range. Grout flowing out of an adjacent hole or joint or the edge of the slab is also sufficient evidence that voids and cavities are filled and pumping should stop. The slab must not be lifted more than 0.050 inch at the outside joint corner.

The stabilized JPCP should not be opened to traffic for 4 to 6 hours until grout has set up.

4. Inspection/Acceptance

Inspection of a slab stabilization project should focus on getting the optimum amount of polyurethane material injected into the holes located near a given slab corner with loss of support, so that the corner will be fully supported. Too little injected material and support will not be restored. Too much, and the slab will lift up and perhaps lead to cracking (this was identified as the major concern for inspection). For slab stabilization, over pumping is indicated by too much slab lifting. Thus, there are two key inspection points:

- **Post-stabilization load deflection testing of the slab corners** along the project is the key test to ensure that sufficient material has been injected. The Missouri project provides an example of this test. If the load versus deflection plot shows the line passing close to the origin, the joint will pass inspection. If not, then the contractor would need to return and inject materials into the slab again.
- **Lifting of the slab is the key test to ensure that too much material is not injected into the hole.** Missouri and Georgia have specification limits when the slab begins to lift to control the amount.

Missouri, as most States, has limited FWD resources to test every slab for void potential, and this inhibits its wide usage. A sampling plan can be developed to measure only a small percentage of corners and from there extrapolate over the entire project. During the interviews on slab stabilization it was suggested by some that, if there is no time to perform the FWD test illustrated in Figure 27, then slab stabilization should not be performed because there is no way to determine if it is done properly. Deflections of slab corners are the key result of slab stabilization that should be measured.

Georgia follows the same procedure as Missouri. After pumping the grout into a designated slab and permitting traffic over the slab for at least 12 hours, the slabs are tested for stability. Based on test results and criteria, slabs will be accepted or designated for further undersealing.

Incentive/Disincentive

There are no incentives/disincentives used by any of the States for slab stabilization. The best potential factor to place incentives/disincentives on is the percentage of corners that were retested and actually achieved full restoration of support. The higher this percentage, the higher the incentive could be set to encourage a higher quality and consistency of stabilization.

5. Performance & Survival of Slab Stabilization

The actual life of slab stabilization would be until either significant joint faulting or loss of support returned to a project. If DBR has also been installed in an undoweled joint, the slab stabilized corners are less likely to deteriorate over time, due to greatly reduced deflections.

Therefore, the service life depends on whether joint load transfer was also performed with DBR. A badly faulted non-doweled JPCP project, slab stabilized without DBR, will not prevent the return of faulting as long as if DBR had been included. Performance of slab stabilization in Missouri and Georgia is briefly summarized below.

Missouri. The typical service life of slab stabilized JRCP (mostly at working transverse cracks) in Missouri was estimated by experienced State personnel at 5 to 10 years. A contractor who has conducted many CPR projects estimates the life of slab stabilization using polyurethane injection material as 10 to 15 years.

Georgia. When performed according to the specifications for cementitious grout, the support to the JPCP non-doweled joints was restored for 5+ years (no DBR at transverse JPCP joints).

6. Summary of Slab Stabilization

The first key to successful slab stabilization is to limit stabilization to the slab locations where deflection testing (Figure 27) indicates loss of support exists. There is no need to inject joints or working cracks where full support exists unless there is an obvious settlement that needs stabilization.

The second key is the detail and effectiveness of the specifications, special provisions, and standard drawings (design) for slab stabilization. The items that need close attention are included in this document.

The third key is the material pumped beneath the slabs. The material should reduce deflections at the slab corners and maintain full support without pumping and eroding. Polyurethane is currently the most popular material being used to restore support and reduce future erosion and pumping, but liquid asphalt and grout slurry have also been used with success, as long as their specific product installation procedures have been followed.

The fourth key is the inspection/acceptance procedures and their effectiveness. The use of deflection equipment (e.g., FWD) to test undersealed joints for load support and elimination of voids is highly recommended. If a joint still indicates loss of support, it can be reinjected with material and then retested to ensure compliance.

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