CONCRETE PAVEMENT PRESERVATION GUIDE

THIRD EDITION

IOWA STATE UNIVERSITY
Institute for Transportation

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National Concrete Pavement Technology Center

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About This Guide

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Reference Information for this Guide

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The Concrete Pavement Preservation Guide provides guidance and information on the selection, design, and construction of cost-effective concrete pavement preservation treatments. Its overall aim is to assist highway agencies in effectively managing their concrete pavement network through the application of timely and effective preservation treatments. The preservation approach typically uses low-cost, minimally invasive techniques to improve the overall condition of the pavement.

To reflect advancements and new developments in the concrete pavement preservation arena, this third edition of the guide has been revised to include the following:
• Information on new pavement evaluation equipment, technologies, and protocols
• Information on new materials and techniques for partial-depth repairs
• New information on full-depth repairs, including updated information on precast and utility cut repairs
• Updated information on diamond grinding and grooving, including information on slurry-handling procedures
• An updated chapter on joint sealing with an introduction to the use of surface sealers
• An abbreviated chapter on concrete overlays with links to detailed information in the Guide to Concrete Overlays (4th edition)
• Discussion of general sustainability considerations in the selection of pavement preservation treatments

After the introductory chapter summarizing the guide and referring the reader to additional resources, 11 chapters cover the following topics: pavement preservation concepts; concrete pavement evaluation; slab stabilization and slab jacking; partial-depth repairs; full-depth repairs; retrofitted edgedrains; dowel bar retrofit, cross-stitching, and slot-stitching; diamond grinding and grooving; joint resealing and crack sealing; concrete overlays; and treatment strategy selection.

This guide is aimed at state and local design and material engineers, construction managers, quality control personnel, contractors, material producers and suppliers, technicians, and tradespeople who have some familiarity with concrete pavement behavior and pavement preservation treatments. However, the guide is expected to also be of value to those who are new to the field.</td></tr>
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Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ACPA	American Concrete Pavement Association
ADA	Americans with Disabilities Act
ASR	alkali-silica reactive/reactivity/reaction
BCR	benefit-to-cost ratio
CDG	conventional diamond grinding
CBR	California bearing ratio
CPE	corrugated polyethylene
CRCP	continuously reinforced concrete pavement
CSA	calcium sulfoaluminate
CTE	coefficient of thermal expansion
dB	decibels
dBA	A-weighted decibels
DBR	dowel bar retrofit
DCP	dynamic cone penetrometer
DPI	DCP penetration index
DOT	department of transportation
DRIP	Drainage Requirement in Pavements
ESAL	equivalent single-axle load
EDC	Every Day Counts
FAST Act	Fixing America's Surface Transportation Act
FDR	full-depth repair
FHWA	Federal Highway Administration
FWD	falling weight deflectometer
GPR	ground-penetrating radar
GPS	Global Positioning System
HMA	hot-mix asphalt
IRI	international roughness index
IRPS	inertial road profiling system
JPCP	jointed plain concrete pavement
JRCP	jointed reinforced concrete pavement
LCCA	life-cycle cost analysis
LCP	life-cycle planning
LiDAR	light detection and ranging
LTE	load transfer efficiency
LTPP	long-term pavement performance
MAP-21	Moving Ahead for Progress in the 21st Century Act
MEPDG	Mechanistic-Empirical Pavement Design Guide
MIT	magnetic imaging tomography
MRD	materials-related distress
MSDS	material safety data sheets
MUTCD	Manual on Uniform Traffic Control Devices
NDT	nondestructive testing
NGCS	Next Generation Concrete Surface
NHS	National Highway System
OBSI	on-board sound intensity
OSHA	Occupational Safety and Health Administration
PCC	portland cement concrete
PCI	pavement condition index
PDR	partial-depth repair
PGED	prefabricated geocomposite edgedrains
PMS	pavement management system
PPE	personal protective equipment
PSR	present serviceability rating
PVC	polyvinyl chloride
QPL	qualified products list
RSI	remaining service interval
SCRIM	Sideway-force Coefficient Routine Investigation Machine
TSD	traffic speed deflectometer
TPAD	total pavement acceptance device
UAV	unmanned aerial vehicle
w/cm	water-to-cementitious materials

Contents

Chapter 6

Figures

Chapter 2

Chapter 3

Chapter 4

Chapter 5

Chapter 6

Chapter 7

Chapter 8

Chapter 9

Chapter 10

Tables

Chapter 6

Chapter 7

Chapter 8

Chapter 9

Chapter 10

Chapter 11

Chapter 12

Chapter 1

Introduction

1. Introduction

The effective management of the nation's roadway network is critical to the safe and efficient movement of travelers, goods, and commodities. With more than 4 million miles of public roads, including more than 1 million miles of Federal aid roadways (Van Dam et al. 2019), this roadway network represents an enormous financial investment whose impact touches nearly every component of the nation's socioeconomic vitality.

As the roadway network continues to age and carry ever-increasing traffic levels and as funding levels routinely fall short of those needed to address all requirements, state and local highway agencies are working continually to maintain the condition of their roadway facilities to meet current and future demands.

Pavements left to deteriorate without timely preservation treatments will require more costly and invasive rehabilitation and reconstruction measures, resulting in reduced performance, shortened service lives, and more frequent interventions (leading to greater disruption and inconvenience to the users of the facilities and greater exposure to risk for roadway workers)—with the ultimate result of increased overall costs. Recognizing this, agencies are looking to apply effective, proactive treatments as one element in the resourceful management of their roadway networks.

Various preservation treatments are available to help an agency effectively manage its pavement network. For concrete pavements, whose hallmark characteristic is long service life, pavement preservation activities play an integral role in ensuring that performance expectations are met (and often exceeded). Pavement preservation activities include treatments that accomplish one or more of the following:

- Correct localized distress (e.g., spalling, cracking, and faulting)
- Improve slab support conditions
- Improve load transfer capabilities
- Improve smoothness and rideability
- Reduce water infiltration into the pavement structure
- Prevent the intrusion of incompressible materials into joints or cracks
- Remove water from beneath the pavement structure
- Improve friction and safety
- Reduce noise
- Improve and manage the overall conditions of a pavement network

However, in order for preservation treatments to be effective, they must be (1) applied to the right pavement at the right time, (2) effectively designed for the existing design conditions and prevailing design constraints, and (3) properly constructed or installed using proven construction practices and procedures. These three elements are the very foundation of the *Concrete Pavement Preservation Guide*.

This document focuses primarily on preservation treatments that are applicable at the project level rather than at the network level, the latter of which is where pavement management activities function, and addresses such issues as prioritizing and budgeting. Effective pavement management programs are critical in identifying and forecasting the need for timely pavement preservation treatments, and that important link between forecasting need and optimal pavement management is highlighted in this document.

The second edition of the *Concrete Pavement Preservation Guide* was published in 2014 (Smith et al. 2014). Notable additional resources for guidance on concrete pavement preservation have been published by the Federal Highway Administration (FHWA) over the years and are available from the FHWA website (Geiger et al. 2003, Van Dam et al. 2019). These include a pavement preservation checklist series developed for the FHWA by the National Concrete Pavement Technology (CP Tech) Center (FHWA 2002, 2019). To reflect advancements and new developments in the concrete pavement preservation arena, this third edition of the guide has been revised to include the following:

- Information on new pavement evaluation equipment, technologies, and protocols
- Information on new materials and techniques for partial-depth repairs (PDRs)
- New information on full-depth repairs (FDRs), including updated information on precast and utility cut repairs
- Updated information on diamond grinding and grooving, including information on slurry-handling procedures
- An updated chapter on joint sealing with an introduction to the use of surface sealers
- An abbreviated chapter on concrete overlays with links to detailed information in the *Guide to Concrete Overlays* (4th edition) (Fick et al. 2021)
- Discussion of general sustainability considerations in the selection of pavement preservation treatments

From its first edition, the *Concrete Pavement Preservation Guide* has been prepared to address high-type concrete facilities (Interstates, freeways, and primary roadways), but the information is applicable to all in-service concrete pavements, including collectors, arterials, and local roads and streets. Moreover, the applicability of the pavement preservation treatments to the major concrete pavement types (jointed plain concrete pavement [JPCP], jointed reinforced concrete pavement [JRCP], and continuously reinforced concrete pavement [CRCP]) is addressed.

The intended audience for this guide includes state and local design and material engineers, construction managers, quality control personnel, contractors, material producers and suppliers, technicians, and tradespeople. While this guide is aimed at those who have some familiarity with concrete pavement behavior and pavement preservation treatments, it is expected to be of value to those who are new to the pavement field as well.

Concrete Pavement Types

- Jointed plain concrete pavement (JPCP) consists of short-jointed pavements (with transverse joints generally spaced 15 ft apart) that contain no reinforcing steel distributed throughout the slab. JPCP may, however, contain steel dowel bars across transverse joints and steel tie bars across longitudinal joints. JPCP is the most common type of concrete pavement constructed.
- Jointed reinforced concrete pavement (JRCP) employs longer joint spacing (typically about 30 to 40 ft) and contains steel reinforcement (welded wire fabric or deformed steel bars comprising about 0.10% to 0.20% of the cross-sectional area) distributed throughout the slab. The steel reinforcement is designed to hold tightly together any transverse cracks that develop in the slab. Dowel bars and tie bars are also used at all transverse and longitudinal joints, respectively. JRCP was commonly constructed in the 1960s and 1970s (mostly in the Midwestern states) but sees little construction activity today.
- Continuously reinforced concrete pavements (CRCP) have no regularly spaced transverse joints but contain a significant amount of longitudinal steel reinforcement (typically 0.6% to 0.8% of the cross-sectional area). This high steel content both influences the development of transverse cracks within an acceptable spacing (about 3 to 8 ft) and serves to hold them tightly together. CRCP is considered a premium pavement and is constructed by several agencies on high-volume, urban roadways.

2. Document Organization

In addition to this introductory chapter, this guide contains the following chapters:

Chapter 2 provides background information, including an introduction to general concrete pavement preservation concepts, a summary of anticipated benefits of preservation, a brief overview of preservation treatments, and a review of the data needed to help manage concrete pavements. This is followed by Chapter 3 on pavement evaluation, which describes the basis for determining the suitability of a pavement for preservation (including a description of condition surveys), nondestructive testing, roughness and friction assessment, and materials and laboratory testing). These two chapters establish the foundation for the description of the specific concrete pavement preservation treatments covered in Chapters 4 through 10. Each of those chapters shares the following general elements:

- Introduction
- Purpose and Project Selection
- Limitations and Effectiveness
- Materials and Design Considerations
- Construction
- Quality Assurance
- Troubleshooting
- Summary
- References

Chapter 11 presents an abbreviated discussion on concrete overlays, highlighting their potential for use in the pavement preservation environment. The application of a properly designed and constructed concrete overlay not only increases load-carrying capacity but also improves pavement surface characteristics and extends the service life of pavements.

Finally, Chapter 12 describes factors to be considered in the selection of pavement preservation strategies and provides an approach to help identify suitable strategies for a given pavement project.

3. Additional Information

Although this guide presents a considerable amount of information on the use of concrete for pavement preservation treatments, numerous references are cited throughout the document to provide readers with additional sources of information. General sources of information on concrete pavement preservation are listed in Table 1.1.

Table 1.1. Sources of additional information

Federal Highway Administration	
National Highway Institute (NHI) 2600 Park Tower Drive, Suite 500 Vienna, VA 22180 www.nhi.fhwa.dot.gov	Office of Highway Policy Information 1200 New Jersey Avenue SE Washington, DC 20590 www.fhwa.dot.gov/policyinformation
Office of Preconstruction, Construction, and Pavements 1200 New Jersey Avenue SE Washington, DC 20590 www.fhwa.dot.gov/pavement www.fhwa.dot.gov/preservation	Office of Research, Development, and Technology Turner-Fairbank Highway Research Center 6300 Georgetown Pike McLean, VA 22101 highways.dot.gov/research
Office of Stewardship, Oversight, and Management 1200 New Jersey Avenue SE Washington, DC 20590 https://www.fhwa.dot.gov/asset	
Industry	
American Concrete Pavement Association (ACPA) 9450 W. Bryn Mawr Avenue, Suite 150 Rosemont, IL 60018 www.acpa.org	International Grooving & Grinding Association (IGGA) 12573 Route 9W West Coxsackie, NY 12192 www.igga.net
Portland Cement Association (PCA) 5420 Old Orchard Road Skokie, IL 60077 www.cement.org	
Other	
American Association of State Highway and Transportation Officials (AASHTO) 555 12th Street NW, Suite 1000 Washington, DC 20004 www.transportation.org	American Society of Civil Engineers (ASCE) 1801 Alexander Bell Drive Reston, VA 20191 www.asce.org
Foundation for Pavement Preservation (FP^2) 8100 West Court Austin, TX 78759 fp2.org	National Center for Pavement Preservation (NCPP) 2857 Jolly Road Okemos, MI 48864 www.pavementpreservation.org
National Concrete Pavement Technology Center (CP Tech Center) Iowa State University 2711 South Loop Drive, Suite 4700 Ames, IA 50010 cptechcenter.org	National Precast Concrete Association (NPCA) 1320 City Center Drive, Suite 200 Carmel, IN 46032 precast.org

4. References

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Chapter 2

Pavement Preservation Concepts

1. Introduction

In the 1990s, the FHWA launched a formal initiative promoting the use of pavement preservation as a cost-effective way of managing the country's roadway (pavement) network. At the time, this was a radically different approach to managing pavement networks than what had been previously used (i.e., a "worst-first" type of approach) and it spurred a nationwide movement towards the adoption of pavement preservation and preventive maintenance programs as agencies began to focus on being proactive rather than reactive.

The FHWA helped solidify the role of pavement preservation by publishing a memo in 2005 that defined pavement preservation and laid out its many potential benefits and advantages (Geiger 2005). The agency issued a revised definition in 2016 (FHWA 2016a), and a more recent definition pertaining to concrete pavements is now being advanced (Van Dam et al. 2019), indicating the evolving nature of pavement preservation.

The enactment of the Moving Ahead for Progress in the 21st Century Act (MAP-21) in 2012 reinforced the importance of pavement preservation and recognized it as a valuable component in the Federal highway program. That transportation reauthorization bill invested in an expanded National Highway System (NHS), with more than half of the funding going to preserving and improving the country's most important highways (FHWA 2012). In addition, MAP-21 promoted a performance-based approach to surface transportation with a focus on improving safety, maintaining infrastructure condition, reducing traffic congestion, improving system efficiency, protecting the environment, and reducing delays in project delivery—considerations that correlate strongly with pavement preservation impacts.

On the heels of MAP-21, the Fixing America's Surface Transportation Act (FAST Act) was signed into law in December 2015 to build on the initiatives of its forerunner legislation (FHWA 2016b). One ongoing important element of the MAP-21 and FAST Act legislation is the National Highway Performance Program (NHPP). The NHPP provides support for the condition and performance of the NHS, provides for the construction of new facilities on the NHS, and ensures that investments of Federal aid funds in highway construction are directed to support progress toward the achievement of the performance targets established for the NHS in a state's asset management plan on both state and local routes. In this way, the FAST Act continues to recognize that pavement preservation is a vital component of achieving and sustaining highway facilities in a desired state of good repair (FHWA 2016b).

This chapter provides an overview of pavement preservation concepts and introduces the different types of concrete pavement preservation treatments, including their general use and applicability.

Evolving Definitions of Pavement Preservation

Geiger (2005):

Preservation is a program employing a network-level, long-term strategy that enhances pavement performance by using an integrated, cost-effective set of practices that extend pavement life, improve safety, and meet motorist expectations.

FHWA (2016a):

Preservation consists of work that is planned and performed to improve or sustain the condition of the transportation facility in a state of good repair. Preservation activities generally do not add capacity or structural value but do restore the overall condition of the transportation facility.

Van Dam et al. (2019):

Concrete pavement preservation is a strategy of extending concrete pavement service life for as long as possible by arresting, greatly diminishing, or avoiding pavement deterioration processes.

2. Description of Pavement Preservation

Pavement preservation is often described as "providing the right treatment to the right pavement at the right time." Figure 2.1 shows the relative timing of treatment activities that can be applied to a pavement—including pavement preservation, rehabilitation (where minor rehabilitation is a treatment or series of treatments placed reactively and major rehabilitation is a structural overlay), and reconstruction.

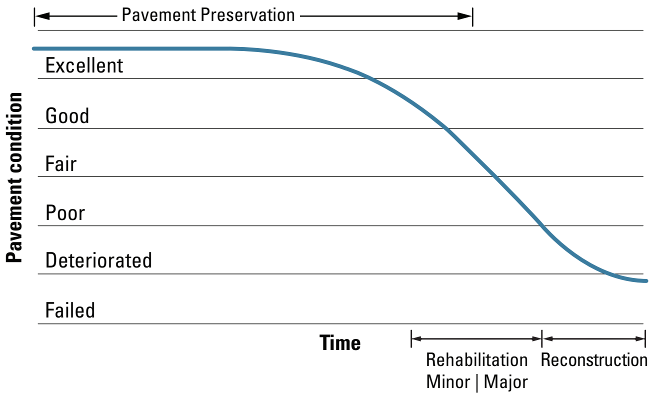

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Figure 2.1. General applicability of pavement preservation, rehabilitation, and reconstruction activities

The preservation area of the curve in Figure 2.1 is the portion that covers the early years of the constructed pavement while it is still in good to very good condition. For concrete pavements, common pavement preservation treatments may include slab stabilization, FDRs and PDRs, retrofitted edgedrains, dowel bar retrofit (DBR), diamond grinding, diamond grooving, joint resealing, and, in some instances, thin concrete overlays. Note that preservation treatments can be applied again periodically after a major rehabilitation (e.g., a structural concrete overlay) to enhance the performance of the rehabilitation activity and extend its service life.

Adopting a pavement preservation approach means promoting and applying proven and effective engineering solutions in the management of pavement structures. Under the definition of concrete pavement preservation proposed by Van Dam et al. (2019), effective engineering solutions can include the following:

- Designing and constructing durable, long-lasting concrete pavements:

  Many agencies have adopted long-life concrete pavements, which focus on the use of quality foundations, durable materials, and excellent workmanship to achieve extended performance. Such designs effectively defer major structural or materials failures, allowing relatively noninvasive preservation techniques to be utilized to maintain functionality while delaying or eliminating significant structural deterioration. Although the construction of long-life concrete pavements typically comes at a higher initial cost, a life-cycle perspective reveals that reductions in future maintenance and rehabilitation costs (as well as reductions in user costs) can more than offset that increased initial investment.

- Using overlays to maintain the structural capacity and serviceability of a pavement:

  Overlays have been demonstrated effective in preventing or arresting the development of distress in the underlying concrete pavement, thereby preserving the existing pavement (Fick et al. 2021). An overlay also adds additional load-carrying capacity while imparting excellent surface characteristics (improved smoothness and friction along with reduced noise) to serve the traveling public.

- Maintaining serviceability using targeted and timely concrete pavement preservation treatments:

  The application of preservation treatments can be a highly effective solution if the existing pavement is in sound structural condition and free of materials-related distress (MRD). When placed in a timely fashion before severe levels of deterioration have developed, treatments can be targeted to specific distresses and shortcomings in an existing pavement, can be applied with minimal disruption to traffic, and can be extremely cost-effective (generally quantified in terms of lower life-cycle costs).

3. Benefits of Pavement Preservation

Pavement preservation provides not only a logical approach to preserving assets but an approach offering measurable benefits to agencies.

- **Higher customer satisfaction**—Customer satisfaction is at the heart of successful pavement preservation practices. From project selection to treatment selection to construction, an effective preservation program will benefit roadway users in several areas, including the following:

  - **Enhanced smoothness**—Most roadway users cite smoothness as the primary indicator they use to assess the overall condition of a pavement.

  - **Increased safety**—Although not readily apparent to roadway users, effective pavement preservation maintains adequate friction and can reduce hydroplaning potential; in addition, pavements with higher smoothness levels typically have fewer distresses, which contributes to safer operating conditions.

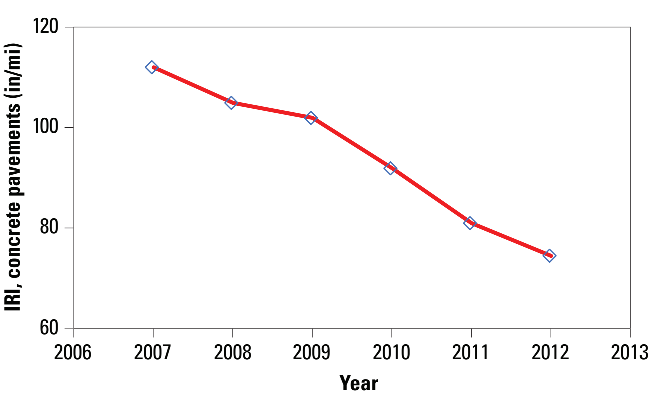
  - **Reduced traffic disruptions**—Through effective, less invasive treatments, the number and duration of associated lane closures is reduced.

  - **Increased proportion of the pavement network at a high level of functional condition**—With an increased proportion of the pavement network at a high level of functional condition, roadway users can reap greater benefits over the course of their travels.

- **Improved pavement condition**—Pavement preservation is a proactive approach intended to preserve a pavement and extend its useful performance period or cycle. The idea is that if pavements in good condition are kept in good condition longer (delaying the need for more substantial rehabilitation or reconstruction), then an obvious benefit is overall improved conditions. Such a benefit was realized by the state of Kentucky in a concrete pavement preservation initiative that began in 2007 (Dispenza 2015). Over a five-year period, the state focused on preserving 536 lane miles of Interstate concrete pavement through an aggressive program of diamond grinding, patching, and joint sealing to achieve a remarkable improvement in the overall rideability of the network (see Figure 2.2).

- **Cost savings**—From an agency standpoint, probably the most sought-after benefit of pavement preservation is financial. Such savings are in the form of less expensive treatments, deferment of more substantial rehabilitation, and pavements with extended service lives. In the case of the Kentucky program, the state realized savings of more than $1 billion by adopting a program of preservation in lieu of more expensive and invasive rehabilitation and reconstruction alternatives. However, indirect cost savings can also be achieved in the form of decreased user costs that result from reduced time delays (due to shorter work zone durations), lower vehicle operating costs (due to shorter work zone durations and smoother roads), and lower crash-related costs.

- **Enhanced sustainability**—Recent years have seen a growing number of transportation agencies and organizations embracing principles of sustainability, in which key environmental, social, and economic factors are considered together in the decision-making process (Van Dam et al. 2015). The pavement preservation approach fits well into this sustainability framework and is inherently a sustainable activity. Preservation treatments typically have a lower environmental footprint (often expressed in terms of greenhouse gas emissions and energy consumption throughout the material production and treatment installation process), offer important social benefits (e.g., increased smoothness, increased safety, reduced noise, and shorter lane closure durations), and provide cost-effective solutions when applied at the right time and using effective procedures. One example of this is the application of diamond grinding relatively early in the life of a pavement before its roughness has reached excessively high levels. Diamond grinding performed at this time will cost less because the conditions are not as severe—and the resulting high levels of smoothness will provide customer satisfaction while increasing fuel efficiency (saving money and resources) and reducing emissions. Thus, not only does it make sense economically to maintain existing pavement assets before they reach a point requiring major rehabilitation or reconstruction, but there are compelling environmental and social justifications for pavement preservation as well.



Adapted from IGGA 2015, used with permission

Figure 2.2. Improvement in Kentucky concrete Interstate pavements as the result of a preservation program

However, for any of the above benefits to be realized, the treatments must be placed on pavements that are good candidates for preservation, and all treatments must be properly designed, properly constructed, and properly maintained throughout their service lives. These prerequisites were the underlying drivers for a recent pavement preservation initiative undertaken by the FHWA under the fourth round of its Every Day Counts (EDC) program. Under EDC-4, two components of pavement preservation were championed (FHWA 2017):

- **When and where**—This component of pavement preservation emphasized the importance of targeting appropriate pavements in a proactive manner. As part of this, the use of life-cycle planning (LCP) was highlighted as a cost-effective means of managing assets (including pavements) at the network level over their whole service lives (Zimmerman et al. 2019). Pavement preservation plays a critical role in LCP, not only by reducing agency costs but also in helping agencies achieve performance targets. The incorporation of LCP concepts has become an integral part of the Transportation Asset Management Plans (TAMPs) that state transportation departments are now required to produce.

- **How**—This component of pavement preservation focused on effective application procedures for several pavement preservation treatments, including (among others) FDRs, PDRs, diamond grinding, and DBR. The proper installation of these treatments not only minimizes early failures but also contributes to improved pavement performance, fewer disruptions, and increased cost savings.

In support of these when and where and how components of effective pavement preservation, the FHWA, under its EDC initiative, conducted summit meetings and workshops, worked with state transportation departments in implementing proven practices, developed updated pavement preservation checklists and technology briefs, and delivered outreach training and guidance.

4. Introduction to Concrete Pavement Preservation Treatments

A range of treatments can be used in the preservation of concrete pavements, as shown in Tables 2.1 and 2.2.

These various pavement preservation treatments use different materials (or, in some cases, no materials), may be applied either globally across the pavement or locally wherever specific distresses or other issues exist, and may involve the removal of a small amount of the existing pavement and/or the placement of new material.

Table 2.1. Primary concrete pavement preservation treatments

Treatment type	Treatment description
Crack sealing	Sawing, power cleaning, and sealing cracks (typically transverse, longitudinal, and corner-break cracks wider than 0.125 in.) in concrete pavement using high-quality sealant materials
Diamond grinding	Removal of a thin layer of concrete (typically 0.12 to 0.25 in.) from the pavement surface, using special equipment fitted with a series of closely spaced, diamond saw blades
Diamond grooving	Cutting of narrow, discrete grooves into the pavement surface, either in the longitudinal direction (i.e., in the direction of traffic) or the transverse direction (i.e., perpendicular to the direction of traffic)
Dowel bar retrofit	Placement of dowel bars across joints or cracks in an existing concrete pavement to restore load transfer
Full-depth repair	Cast-in-place or precast concrete repairs that extend through the full thickness of the existing slab, requiring full-depth removal and replacement of full or partial lane-width areas
Joint resealing	Removal of existing deteriorated transverse and/or longitudinal joint sealant, refacing and pressure cleaning of the joint sidewalls, and installation of new material (e.g., liquid sealant and backer rods, preformed compression seal)
Partial-depth repair	Removal of small, shallow (typically up to half of the slab thickness) areas of deteriorated concrete and subsequent replacement with a cementitious or polymeric repair material

Table 2.2. Additional concrete pavement preservation treatments

Treatment type	Treatment description
Concrete overlay	Placement of a thin concrete layer (typically 4 to 6 in. thick) on a milled or prepared surface
Cross-stitching	Placement of deformed tie bars into holes drilled at an angle through cracks (or, in some cases, joints) in an existing concrete pavement
Slab stabilization	Filling of voids beneath concrete slabs by injecting polyurethane, cement grout, asphalt cement, or other suitable materials through drilled holes in the concrete located over the void areas
Slab jacking	Raising of settled concrete slabs to their original elevation by pressure-injecting cement grout or polyurethane materials through drilled holes at carefully patterned locations
Slot-stitching	Grouting of a deformed bar into slots cut across a longitudinal joint or crack
Retrofitted edgedrains (and maintenance)	Cutting of a trench along the pavement edge and placement of a longitudinal edgedrain system (pipe or geocomposite drain, geotextile lining, bedding, and backfill material) in the trench, along with transverse outlets and headwalls

Although each treatment is generally applicable to all major concrete pavement types (JPCP, JRCP, and CRCP), there are some obvious exceptions. For example, transverse joint resealing is not performed on CRCP because this pavement type contains no regularly spaced transverse joints.

An important consideration regarding many of these concrete pavement preservation treatments is that, under the Americans with Disabilities Act (ADA), they are considered “maintenance” and not an “alteration” and are therefore not subject to additional accessibility requirements. The ADA defines an “alteration” as a change that affects or could affect the usability of all or part of a building or facility. With regard to pavements, ADA-defined “alterations” typically include activities such as reconstruction, rehabilitation, resurfacing, and widening (Iowa SUDAS 2021). In contrast, pavement preservation treatments such as joint and crack sealing, diamond grinding, dowel bar retrofit, joint repairs, and pavement patching are all considered as “maintenance” items under ADA legislation.

Table 2.3 indicates the unique capabilities and functions of each of various pavement preservation treatments in terms of its impacts on the structural and/or functional performance of the existing pavement.

The impacts of these various pavement preservation treatments may be in the form of preventing or delaying the occurrence of new distresses, slowing the development of existing distresses, restoring the integrity and functionality/serviceability of pavements, and improving surface characteristics related to user safety and comfort.

However, even though preservation treatments can be used to address a range of pavement distress types and conditions, the presence of a significant amount of certain key structural distresses may suggest that an existing pavement is not an appropriate candidate for preservation. General indicators of structural adequacy for different existing concrete pavements based on key distress types are provided in Table 2.4, with a range of values presented to reflect different facility types and traffic levels.

Table 2.3. Primary functions of concrete pavement preservation treatments

Treatment	Seal pavement/ minimize pumping	Fill voids, restore support, address pavement deterioration	Remove moisture beneath structure	Prevent intrusion of incompressible materials	Remove/ reduce faulting	Improve texture for friction	Improve profile (lateral surface drainage and ride)	Improve texture for noise
Slab stabilization	✓	✓			✓			
Slab jacking		✓			✓		✓	
Partial-depth repair		✓					✓	
Full-depth repair	✓	✓		✓	✓		✓	
Retrofitted edgedrains (and maintenance)			✓		✓			
Dowel bar retrofit					✓		✓	
Cross-stitching					✓		✓	
Slot-stitching					✓		✓	
Diamond grinding					✓	✓	✓	✓
Diamond grooving						✓		
Joint resealing	✓			✓				
Crack sealing	✓			✓				
Concrete overlay						✓	✓	✓

Source: Adapted from Peshkin et al. 2011

Table 2.4. General indicators of structural adequacy for Interstate and primary roadways

Distress by pavement type	Adequate structural distress level	Marginal structural distress level	Inadequate structural distress level
JPCP—Medium- and high-severity cracking and corner breaks (% of slabs)	Less than 5 to 8	5 to 15	More than 10 to 15
JPCP—Mean joint/crack faulting (in.)	Less than 0.10 to 0.125	0.10 to 0.20	More than 0.15 to 0.20
JRCP—Medium- and high-severity cracking and corner breaks (# per lane mile)	Less than 15 to 20	15 to 50	More than 40 to 50
JRCP—Mean joint/crack faulting (in.)	Less than 0.15 to 0.175	0.15 to 0.35	More than 0.30 to 0.35
CRCP—Medium- and high-severity punchouts (# per lane mile)	Less than 5 to 8	5 to 15	More than 10 to 15

Source: Adapted from Harrington and Fick 2014

5. Pavement Management Data for Successful Preservation

As described previously, pavement preservation programs rely on proper treatment selection as well as proper treatment timing to be successful. To manage a pavement network effectively, the following information must be compiled and analyzed:

- Structure and condition of existing pavement
- Current and projected traffic
- Local climatic conditions
- Expected performance and anticipated service life extension after application of the preservation treatment
- Expected costs (initial and life-cycle) of the treatments, both direct (agency costs) and indirect (user costs)
- Construction considerations and other factors associated with the treatment that affects selection

The availability of the above information is an essential part of the process of managing a successful pavement preservation program. Successful programs exploit the data available from the agency's pavement management system (PMS) to help in the decision-making process. Although state and local highway agencies collect and analyze pavement management data in different ways, some of the types of data used to assess needs and to program treatments include the following:

- Existing pavement structure/history (including past preservation treatment applications)
- Traffic loadings
- Distress types (e.g., faulting, cracking, and spalling) along with severity levels and amounts
- Overall condition indexes/ratings
- Surface profile/smoothness
- Surface friction and macrotexture
- Falling weight deflectometer (FWD) data (e.g., deflections, load transfer efficiencies, and backcalculated layer moduli)
- Ground-penetrating radar (GPR) data (e.g., thickness, voids, and steel location)

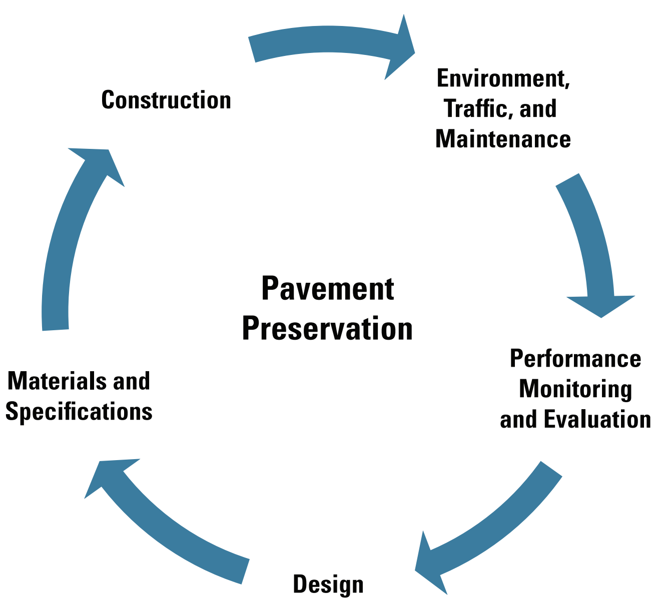
In the pavement preservation realm, pavement management data are very important in determining (1) whether a project is a suitable candidate for preservation, (2) which treatments are feasible for a project, and (3) which treatment is most ideal in terms of cost-effectiveness and other considerations. Performance indicators—such as overall condition indexes/ratings, smoothness indexes, and key distress measures—can be used to establish the pavement preservation window that defines when preservation should be considered for a project. Likewise, these same performance indicators can be used to set trigger and threshold levels for individual treatments that govern when they should be considered.

Although condition indexes and ratings are useful in planning and programming, it is important when developing a preservation program to manage the actual distresses separately, as targeting known deficiencies in a pavement helps to ensure the effectiveness of preservation treatments. Moreover, there is a need to consider other conditions or indicators—such as nondestructive testing (NDT) data, voids, or delamination—that may suggest a more proactive approach to applying pavement preservation treatments. Also, follow-up visits to candidate projects are still needed to ensure their feasibility for preservation.

Predicting Performance

The historical condition and performance data contained in a PMS can be used to develop time-series pavement performance models that can assist in the selection of appropriate preservation treatments based on expected performance and cost-effectiveness. The development of improved distress-based pavement performance prediction models (for such things as cracking, faulting, spalling, and roughness) can be used to provide insight into the appropriate timing for a given pavement preservation treatment and to allow comparisons of predicted to observed performance conditions—which in turn contributes to improvements in the design, materials, construction, and maintenance aspects of pavement structures. Using performance data as a feedback loop to continually improve the performance of pavement structures is an important component in the effective use of pavement management systems, and this same concept is applicable to agencies aiming at improved pavement preservation (see Figure 2.3).

A valuable resource in the evaluation of the need for and timing of pavement preservation treatments is the AASHTO *Mechanistic-Empirical Pavement Design Guide* (MEPDG). The most recent edition of the MEPDG was released in 2020 (AASHTO 2020) and is accompanied by the software program AASHTOWare Pavement ME Design. The MEPDG's mechanistically based design procedure predicts the performance of new and rehabilitated concrete pavements in terms of smoothness and key distress parameters, such as faulting, cracking, and punchouts. The MEPDG design procedure can be used to forecast rehabilitation and preservation needs using established trigger values for distress and/or smoothness—and, as described earlier, the MEPDG procedure can also be used to compare predicted to actual performance, enabling improvements over time to an agency's overall pavement design and construction process.



Recreated from Larry Scofield, IGGA, used with permission

Figure 2.3. Continuous feedback to improve pavement preservation application and performance

In recognition of the importance of preservation to improved pavement performance, a National Cooperative Highway Research Program (NCHRP) study has investigated different approaches for incorporating pavement preservation into the AASHTOWare Pavement ME Design software (APTech 2015). This study identified several approaches and procedures for designing concrete and asphalt pavement structures that allow the pavement design to account for the effects of future scheduled preservation treatments on pavement life. By designing a pavement to include preservation at key points in its life and carrying through with the application of those treatments once the pavement has been put into service, the pavement can be kept in better overall condition.

Treatment Performance

Treatment performance is commonly characterized in terms of the life of the preservation treatment (i.e., how long the treatment lasts until another treatment is needed). An alternative way of characterizing treatment performance involves quantifying the effectiveness of a treatment in improving the existing pavement performance in terms of either pavement life extension or performance benefits:

- Pavement life extension is simply the number of years of additional pavement life (or additional traffic loadings) obtained as a result of applying the treatment. The added life (or added traffic loadings) can be evaluated from the standpoint of structural and/or functional performance, as characterized by key surface distresses (e.g., cracking, faulting, punchouts, and spalling) and/or key pavement surface characteristics (e.g., smoothness, friction, texture, and tire-pavement noise).
- Pavement performance benefits can be quantified in terms of the area under the pavement condition/performance curve—that is, the greater the area under the curve, the more benefit provided by the treatment. Like pavement life extension, the performance benefit area is evaluated from the standpoint of structural and/or functional performance.

Pavement life extension represents a simpler and more straightforward calculation than the performance benefit area, but its use in evaluating concrete pavement preservation treatments has been very limited. Another metric that could be used in evaluating the impact of a preservation treatment is examining the cost per lane mile, an economics-based procedure described further in Chapter 12.

For many preservation treatments (e.g., FDRs, PDRs, and DBR), the goal should be that the life of the treatment is the same as the remaining life of the pavement. For others, such as joint resealing, while shorter treatment lives are obtained, the treatment is still expected to contribute to the performance and longevity of the pavement.

Typical performance lives associated with selected pavement preservation treatments are summarized in Table 2.5, with additional discussion on these treatments available in the cited chapters.

Achieving these typical performance lives is strongly dependent on selecting the appropriate pavement, using durable materials (where applicable), and following quality construction practices.

The condition of the existing pavement (and hence the timing of the application of a preservation treatment) can have a significant effect on the performance of the treated pavement. However, there are inevitably lags between the time of pavement data collection/analysis and actual treatment application (from as little as one to two years to as many as four to five years). Clearly, significant changes in pavement conditions can occur over these periods, which underscores the importance of identifying preservation needs and getting them programmed and implemented as expeditiously as possible.

Table 2.5. Typical range of expected performance life for selected concrete pavement preservation treatments

Treatment	Typical range of expected performance (treatment life)
Slab stabilization	5 to 10 years (see Chapter 4)
Partial-depth concrete patching	10 to 20+ years (see Chapter 5)
Full-depth concrete patching	20+ years (see Chapter 6)
Dowel bar retrofit	15 to 20+ years (see Chapter 8)
Cross-stitching	10 to 20+ years (see Chapter 8)
Diamond grinding	15 to 25+ years (see Chapter 9)
Joint resealing	8 to 16+ years (see Chapter 10)

6. Summary

Formal pavement preservation practices largely emerged in the 1990s, and over the ensuing decades the applications of and technologies for pavement preservation have continued to grow and evolve. Today, pavement preservation is viewed as an important component of an agency's approach to managing its pavement network and is defined as a strategy of extending concrete pavement service life for as long as possible by arresting, greatly diminishing, or avoiding pavement deterioration processes.

The benefits of pavement preservation are usually considered in terms of overall cost savings to a given agency, but there are other important benefits including improved pavement condition, extended service life, and increased customer satisfaction (in regard to smoothness, safety, noise reduction, etc.). In addition, there are indirect cost savings that can be realized from pavement preservation treatments in terms of decreased user costs resulting from reduced time delays (due, for example, to shorter work zone durations), lower vehicle operating costs (due to shorter and fewer work zones, smoother roads, etc.), and lower crash-related costs. Finally, most pavement preservation treatments are inherently sustainable as they typically exhibit lower environmental impacts than conventional rehabilitation treatments.

Several concrete pavement preservation treatments are available to meet a range of conditions. These treatments are often applied in combination to address several deficiencies and to maximize overall effectiveness—and when applied in a timely fashion, preservation treatments can also significantly improve pavement performance and extend service life.

Key factors influencing the impact of preservation treatments are applying them to the appropriate pavement at the appropriate time, using durable materials, and adhering to proven construction practices. Agency pavement management systems provide valuable information in planning and programming pavement preservation treatments and can enable ongoing improvements in agencies' performance prediction capabilities.

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Chapter 3
Concrete Pavement Evaluation

1. Introduction

Prior to selecting a preservation or rehabilitation treatment, a pavement evaluation to determine the causes and extent of pavement deterioration should be conducted. The approach to pavement evaluation described in this chapter is consistent with that presented in the AASHTO *Guide for Design of Pavement Structures* (AASHTO 1993) and in the AASHTO *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice* (AASHTO 2020).

The size of a project often dictates the time and funding levels that can justifiably be spent on pavement evaluation. At times, however, the funding and timing limitations may also dictate the type and extent of the preservation treatments to be applied, as well as the extent of project limits (i.e., the project may be scaled down in length or scope to meet available funding levels). Additionally, critical projects on major roadways and projects subjected to high traffic volumes require more comprehensive pavement evaluations than those on low-volume roadways. This is not because data collection is less important on lower volume roadways but because the ramifications of premature failures on the higher volume roadways is much more serious. Furthermore, some data items (such as roughness or noise levels) may not be as important on lower volume roads where often both traffic volumes and speeds are lower.

This chapter first presents a summary of all possible data that could be included in a pavement evaluation and then provides an overview of the steps involved in the project evaluation process, including the following:

- Pavement historical data
- Distress and drainage surveys
- NDT
- Evaluation of pavement surface characteristics (including noise, roughness, friction, and texture)
- Field materials sampling and testing

Not all of these evaluation components are needed for every project. The distress and drainage surveys generally drive much of the evaluation process and decision-making, but some of the other testing items may be needed depending on the characteristics exhibited by the existing pavement.

2. Data for Pavement Evaluation

Depending on the condition of the pavement, the location, the type of facility, and so on, data from these major categories will need to be collected:

- Pavement condition (e.g., distress, deflections)
- Surface characteristics (e.g., roughness, friction, noise)
- Shoulder condition
- Pavement design (e.g., layer thicknesses, layer properties, structural characteristics, construction requirements)
- Pavement layer materials and soil properties
- Traffic volumes and loadings (current and projected)
- Climatic conditions
- Drainage conditions
- Geometric factors
- Safety aspects (e.g., crashes, surface friction)
- Miscellaneous factors (e.g., utilities, clearances)

In many cases, the specific data needed also depend upon the treatment alternatives being considered. For example, if the grinding of a concrete pavement is to be considered, then the hardness of the aggregate and the faulting condition must be known. Table 3.1 provides a summary of suggested data collection items for various concrete treatment alternatives.

In summary, for pavement preservation to be maximally effective, the initial pavement evaluation data collection effort should accomplish the following:

- Provide the *qualitative* information needed to determine the causes of pavement deterioration and to develop appropriate alternatives for repairing the deterioration and preventing its recurrence.
- Provide the *quantitative* information needed to make quantity estimates associated with different treatment alternatives, to assess the rate of pavement deterioration, and to perform life-cycle cost comparisons of competing treatment alternatives.

The design engineer's overall objective in pavement evaluation is to make the most efficient use of data collection resources so that sufficient information can be obtained to identify feasible alternatives and to develop cost-effective designs.

Table 3.1. Suggested data collection needs for concrete pavement preservation treatment alternatives

Data collection item	Full-depth repair	Partial-depth repair	Concrete overlay	Diamond grinding	Diamond grooving	Slab stabilization	Slab jacking
Pavement design	Yes	Yes	Yes	Helpful	Helpful	Yes	Yes
As-constructed thickness	Helpful	Helpful	Helpful	Yes	No	Helpful	Helpful
Age	Helpful	Helpful	Helpful	Helpful	Helpful	No	No
Materials properties	Helpful	Helpful	Yes	Yes	Yes	Helpful	Helpful
Subgrade	Helpful	No	Yes	No	No	Helpful	Helpful
Climate	No	No	Yes	No	No	No	No
Traffic loads and volumes	Helpful	Helpful	Yes	Yes	Yes	Helpful	Helpful
Distress	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Safety (friction, texture, crashes)	No	No	Helpful	Yes	Yes	No	No
Potential NDT	Helpful (FWD, GPR, MIT, MIRA)	No	Yes (FWD) Helpful (GPR, MIT, MIRA)	Yes (friction)	Yes (friction)	Yes (FWD) Helpful (GPR, MIRA)	Yes (FWD) Helpful (GPR, MIRA)
Potential destructive testing/sampling	Yes (coring)	Yes (coring)	Yes (coring, DCP, SMT/C, ST, MRD)	Helpful (coring)	Helpful (coring, ST, MRD)	Helpful (coring, DCP, SMT/C)	Helpful (coring, DCP, SMT/C)
Roughness	No	No	Helpful	Yes	Helpful	Helpful	Helpful
Transverse surface profile	No	No	Yes	Yes	Helpful	Helpful	Helpful
Drainage (roadway and subsurface)	Helpful	No	Yes	No	No	Yes	Yes
Previous maintenance	Helpful	Helpful	Helpful	Helpful	Helpful	Helpful	Helpful
Bridge transitions	Helpful	Helpful	Yes	No	No	Helpful	Yes
Utilities	Yes	No	Yes	No	No	Yes	Yes
Traffic control options	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Vertical clearances	Helpful	No	Yes	Helpful	Helpful	No	No
Geometrics	No	No	Yes	No	No	No	No

Table 3.1 continued on following page

Key:

Yes—Definitely needed in the pavement evaluation process
Helpful—Would contribute to the pavement evaluation but not required
No—Not normally needed to evaluate this treatment
FWD = falling weight deflectometer
GPR = ground-penetrating radar
MIT = magnetic imaging tomography
DCP = dynamic cone penetrometer
SMT/C = subsurface materials testing/characterization
ST = strength testing
MRD = materials-related distress
MIRA = ultrasonic tomography device

For more details on potential NDT, see Section 5 of this chapter
For more details on potential destructive testing/sampling, see Section 7 of this chapter

Source: Adapted from AASHTO 1993

Table 3.1 continued from previous page

Data collection item	Retrofitted edgedrains	Joint resealing	Crack sealing	Dowel bar retrofit	Cross-stitching	Slot-stitching
Pavement design	Yes	Yes	Yes	Yes	Yes	Yes
As-constructed thickness	No	No	No	Yes	Yes	Yes
Age	Helpful	No	No	No	No	No
Materials properties	Yes	Yes	Yes	Helpful	Helpful	Helpful
Subgrade	Yes	Yes	Yes	No	No	No
Climate	Yes	Yes	Yes	Helpful	No	No
Traffic loads and volumes	Helpful	Helpful	Helpful	Yes	Helpful	Yes
Distress	Yes	Yes	Yes	Yes	Yes	Yes
Safety (friction, texture, crashes)	No	No	No	No	No	No
Potential NDT	Helpful (FWD, GPR, MIT, MIRA)	No	No	Yes (FWD) Helpful (GPR, MIRA	Helpful (FWD, GPR, MIT, MIRA)	Yes (FWD) Helpful (GPR, MIT, MIRA)
Potential destructive testing/sampling	Yes (coring, DCP, SMT/C)	No	No	Helpful (coring, DCP, SMT/C, ST, MRD)	Helpful (coring, ST, MRD)	Helpful (coring, ST, MRD)
Roughness	No	No	No	No	No	No
Transverse surface profile	No	No	No	No	No	No
Drainage (roadway and subsurface)	Yes	Yes	Yes	Helpful	Helpful	Helpful
Previous maintenance	Helpful	Helpful	Helpful	Helpful	Helpful	Helpful
Bridge transitions	No	Yes	No	Helpful	Helpful	Helpful
Utilities	Yes	No	No	No	No	No
Traffic control options	Yes	Yes	Yes	Yes	Yes	Yes
Vertical clearances	No	No	No	No	No	No
Geometrics	Yes	No	No	No	No	No

Key:

Yes—Definitely needed in the pavement evaluation process
Helpful—Would contribute to the pavement evaluation but not required
No—Not normally needed to evaluate this treatment
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MRD = materials-related distress
MIRA = ultrasonic tomography device

For more details on potential NDT, see Section 5 of this chapter
For more details on potential destructive testing/sampling, see Section 7 of this chapter

Source: Adapted from AASHTO 1993

3. Pavement Evaluation Overview

The overall pavement evaluation process can be broadly divided into the following general steps (Hoerner et al. 2001, AASHTO 2020):

- Historical data collection and records review
- Initial site visit and assessment
- Field testing activities
- Laboratory materials characterization
- Data analysis
- Final field evaluation report

A brief introduction to each of these pavement evaluation steps is presented in the following sections, with more detailed discussions on specific field and laboratory testing activities included later in the chapter.

Step 1: Historical Data Collection and Records Review

The first step of the evaluation process is to review the available historical records that are associated with the project. The goal is to collect as much information on the existing pavement as possible; potential data sources include the following:

- Design reports
- Construction plans/specifications (new, rehabilitation, and widening)
- Materials and soils properties from previous laboratory test programs and/or published reports
- Past pavement condition surveys along with NDT and/or destructive sampling investigation results
- Maintenance/repair histories
- Traffic measurements/forecasts
- Environmental/climate studies
- Pavement management system records

The information gathered in this step can be used to divide the pavement into discrete sections with similar design and performance characteristics in preparation for the initial pavement evaluation site visit.

In the early stages of the pavement evaluation process, it may also be useful to perform an assessment of the remaining structural capacity of the pavement by comparing the original design traffic loadings to those that have occurred to date. Although past pavement loading data can be difficult to obtain and may have questionable accuracy, it can provide the first indication as to whether the pavement is performing as intended, and it may provide direction as to whether preservation is, in fact, an appropriate solution. To aid in the assessment of the pavement's current condition, a reevaluation of the existing pavement design (with the applied pavement loading) can be conducted using the AASHTO *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice* (AASHTO 2020), the results of which can then be compared to the actual pavement performance to see how much of the expected service life has been consumed.

Step 2: Initial Site Visit and Assessment

An initial site inspection is conducted to gain a general knowledge of the performance of the pavement and to help determine the scope of the field testing activities to be conducted in Step 3.

Satellite mapping images as well as video logs or right-of-way videos are useful as an initial review tool. Many of these tools even offer time-series videos that allow the reviewer to see the development of distress over time. Agencies' pavement management systems can also be accessed beforehand for condition and segmentation data. Technology now also allows for multiple locations to remotely access these data and videos so central office, district, maintenance, and consultant personnel can all review and provide input ahead of the initial site visit.

As part of the initial site inspection, pavement information such as distress, surface roughness, surface friction, shoulder conditions, and moisture/drainage problems should be gathered. Data regarding on-site conditions can be collected through "windshield" surveys or shoulder surveys, supplemented with unmanned aerial vehicle (UAV) or dash camera video. Light detection and ranging (LiDAR) technology can be used from a stationary, vehicle, or UAV mount to identify and quantify distress, roadway drainage, longitudinal and transverse profiles, clearances, and so on. In addition, an initial assessment of traffic control constraints, obstructions, right-of-way zones, the presence of bridges and other structures, ADA compliance, and general safety concerns should be made during this visit.

Information obtained from this initial site visit and assessment should be used to formulate the type and extent of the field testing activities that may be needed under Step 3. For example, observations of moisture/drainage problems (e.g., pumping, corner breaks, standing water, and so on) may indicate the need for a more intensive deflection testing program or a more detailed investigation of subsurface drainage conditions.

Depending on the designer's familiarity with the project, discussions with local design and maintenance engineers may also be beneficial to understanding the overall performance of the pavement and whether it has exhibited any recent changes in condition.

Step 3: Field Testing Activities

Under this step, detailed field measurements and testing are conducted to better characterize the pavement's performance. The specific field testing activities performed are guided by the information obtained from the initial site visit and assessment as well as by the potential preservation strategies. Field testing may therefore include the following:

- **Distress and drainage surveys**—These surveys provide a visual indication of the structural condition of the existing pavement and will have the greatest impact on the selection of the appropriate preservation or rehabilitation treatment. While most roadway agencies have developed their own manuals for quantifying pavement distress to best fit local distresses and needs, the FHWA's *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014) is a long-standing and accepted source of information on pavement distresses and distress identification. A guide document specific to concrete pavements is also available from the FHWA that focuses on the identification of distress causes and appropriate repair treatments, *Guide for Concrete Pavement Distress Assessments and Solutions: Identification, Causes, Prevention, and Repair* (Harrington et al. 2018).
- **Nondestructive testing**—This commonly refers to deflection testing, but it may also include specialized testing using technologies such as magnetic imaging tomography (MIT), ultrasonic tomography (e.g., MIRA), and GPR. These technologies may be used to evaluate the overall structural condition of the pavement, to assess its joint load transfer capabilities, to determine the depth of its steel reinforcement, and to determine its layer thicknesses. The scope of the NDT program to be conducted should be established by the design engineer during or after the initial site visit.
- **Surface characteristics testing**—This testing focuses on the functional performance of the pavement, that is, how well the pavement is meeting noise, roughness, and safety (friction, longitudinal and transverse profiles and curvature, and hydroplaning) requirements established for the project.
- **Field material sampling and testing**—Field sampling and testing activities serve several purposes, such as the confirmation of layer materials and thicknesses, the determination of the in situ strength of unbound layers, and the retrieval of cores and subsurface samples for subsequent laboratory testing. Most pavement preservation projects will require limited field sampling or testing programs, if any.

Specific details associated with each of these different types of field testing activities are discussed later in this chapter.

Step 4: Laboratory Materials Characterization

Laboratory testing is not required on every project. When included as part of the pavement evaluation process, however, laboratory testing may be conducted to confirm or clarify certain results from the distress surveys or the NDT program, to provide additional insight into the mechanisms of distress, or to provide additional information needed for the identification of feasible treatment alternatives. Although not typically needed, examples of the types of information that can be determined from laboratory testing include the following:

- Concrete strength data
- Stiffness of concrete and of bound support layers
- Presence of MRD, such as alkali-silica reactivity (ASR) or durability cracking (D-cracking)
- Petrographic analysis of the concrete layer
- Resilient modulus of the unbound layers and of the subgrade
- Density and gradation of underlying granular layers

It is again emphasized that these types of information are not typically needed for most pavement preservation projects.

Step 5: Data Analysis

For each field data collection activity, there is a corresponding element of analysis required. For the pavement condition data—such as distress, roughness, and friction—the data can be plotted in various graphical formats to illustrate varying conditions, often inside an agency's PMS software system. If prepared in bar chart form, these profile plots can depict both the extent and severity at each measurement interval. Slab cracking, corner breaks, faulting, and spalling are candidate distresses that can be expressed in these types of illustrations, along with plots of load transfer, noise, pavement texture, roughness, and friction. In addition, areas of poor drainage, significant changes in topography (cut/fill sections), and changes in traffic levels or patterns can also be overlaid on the strip chart to provide insight into observed conditions.

The collected pavement condition information helps define when pavement preservation activities may or may not be appropriate. Table 3.2 presents examples of action values for different pavement performance indicators. Action values define the starting point when pavement preservation may be considered. It should be noted that the values shown in Table 3.2 are provided as example guidance, and agencies should adjust these action values to fit their strategies and processes.

The interpretation of NDT on concrete pavements can be used in a number of ways, including in the development of pavement deflection profiles, the backcalculation of layer properties, the assessment of the structural capacity of the pavement, the determination of load transfer capabilities, the evaluation of void potential, and the determination of layer thicknesses.

Step 6: Final Field Evaluation Report

The final step in the initial pavement evaluation process is the field evaluation report, which summarizes the results of the overall evaluation process (i.e., records review, site visit, field and laboratory data collection, and data analyses). In addition, all critical nonpavement factors that could impact the selection of treatment alternatives should be identified as part of the final field evaluation report; this could include such items as shoulder condition, ditches, right of way, geometrics, curves, bridges, ramps, ADA requirements, and traffic patterns.

Table 3.2. Example of pavement preservation action values

Pavement type	Performance measure	Action value for Interstates, other freeways, and expressways	Action value for other principal arterials	Action value for minor arterials, collectors, and local roads
Jointed	Low- to high-severity fatigue cracking (% of slabs)	1.5	2.0	2.5
Jointed	Deteriorated joints (% of joints)	1.5	2.0	2.5
Jointed	Corner breaks (% of joints)	1.0	1.5	2.0
Jointed	Average transverse joint faulting (in.)	0.08	0.08	0.08
Jointed	Durability distress (severity)	Medium–high	Medium–high	Medium–high
Jointed	Joint seal damage (% of joints)	>25	>25	>25
Jointed	Load transfer (%)	<50	<50	<50
Jointed	Surface friction	Minimum local acceptable level	Minimum local acceptable level	Minimum local acceptable level
Jointed	IRI (in./mi)	80	90	110
CRCP	Failures (punchouts, FDRs) (no./mi)	3	5	N/A
CRCP	Durability distress (severity)	Medium–high	Medium–high	N/A
CRCP	Surface friction	Minimum local acceptable level	Minimum local acceptable level	N/A
CRCP	IRI (in./mi)	80	90	N/A

Source: Adapted from ACPA 1997

4. Pavement Distress and Drainage Surveys

Section 3 of this chapter provided an overview of the pavement evaluation process. The remaining sections describe the specific field testing components of that evaluation process, namely pavement distress and drainage surveys, NDT, surface characteristics (i.e., noise, longitudinal and transverse profiles, surface friction, and texture) evaluation, as well as materials sampling and laboratory testing.

Project-level pavement distress surveys are the first step in the overall pavement evaluation process, and they serve as the cornerstone for evaluating the suitability of a pavement to receive a given preservation treatment. Distress surveys record visible distresses on the surface of the pavement and are performed to accomplish the following:

- Document pavement condition
- Identify types, quantities, and severities of observed distress
- Group areas of pavement with similar construction exhibiting similar performance
- Gain insight into causes of deterioration (e.g., structural versus functional versus material)
- Identify additional testing needs
- Identify potential treatment alternatives
- Identify repair areas and repair quantities

Pavement distress is any visible defect or form of deterioration in a pavement or on the surface of a pavement, and it is a key measure of the performance of an existing pavement. To fully describe pavement distress, the following three factors must be considered:

- **Type**—The type of distress is determined primarily by similar mechanisms of occurrence and appearance. By identifying the types of distress, a great deal of information can be inferred regarding the underlying causes of deterioration.
- **Severity**—The severity of distress represents the criticality of the distress in terms of progression; more severe distresses may preclude preservation and instead require more extreme rehabilitation measures.
- **Extent**—The quantity and severity level of each type must be measured and recorded.

Examples of some of the more common concrete pavement distress types are illustrated in Figure 3.1.

As part of the distress survey, it is recommended that a drainage survey be conducted if moisture is suspected to be contributing to the distresses. In a drainage survey, visible signs of poor drainage are noted and can be coupled with information from material sample testing and NDT to provide insight into the role that moisture is playing in the performance of the pavement. Items to consider in drainage surveys include the following:

- Depth of ditch compared to pavement profile grade
- Ditch condition
- Longitudinal and transverse profile
- Presence of pumping and/or faulting at transverse joints or lane-shoulder joints
- Condition of edgedrains and their outlets

Distress Survey Procedures

To be consistent in how the distress type, severity, and extent are determined during a distress survey, distress measurement protocols need to be adopted by the agency conducting the survey. Most state and local highway agencies have developed their own protocols or adopted various AASHTO standards for assessing the condition of their pavement structures.

As part of the FHWA's Long-Term Pavement Performance (LTPP) program, a detailed distress survey procedure and standardized distress definitions are available (Miller and Bellinger 2014). The LTPP *Distress Identification Manual* describes the appearance of each distress type, depicts the associated severity levels (where defined), and describes the standard units in which the distress is measured. Figures and photographs of the distress type at various levels of severity are also provided to aid in the distress identification process. Table 3.3 summarizes the distresses defined for concrete pavements in the LTPP *Distress Identification Manual* and notes whether the distresses are primarily related to traffic or climate/materials.

Because the *Distress Identification Manual* was developed for the LTPP program, it should be noted that the manual is research oriented and consequently requires that pavement distress data be collected in considerable detail and at relatively high levels of precision.

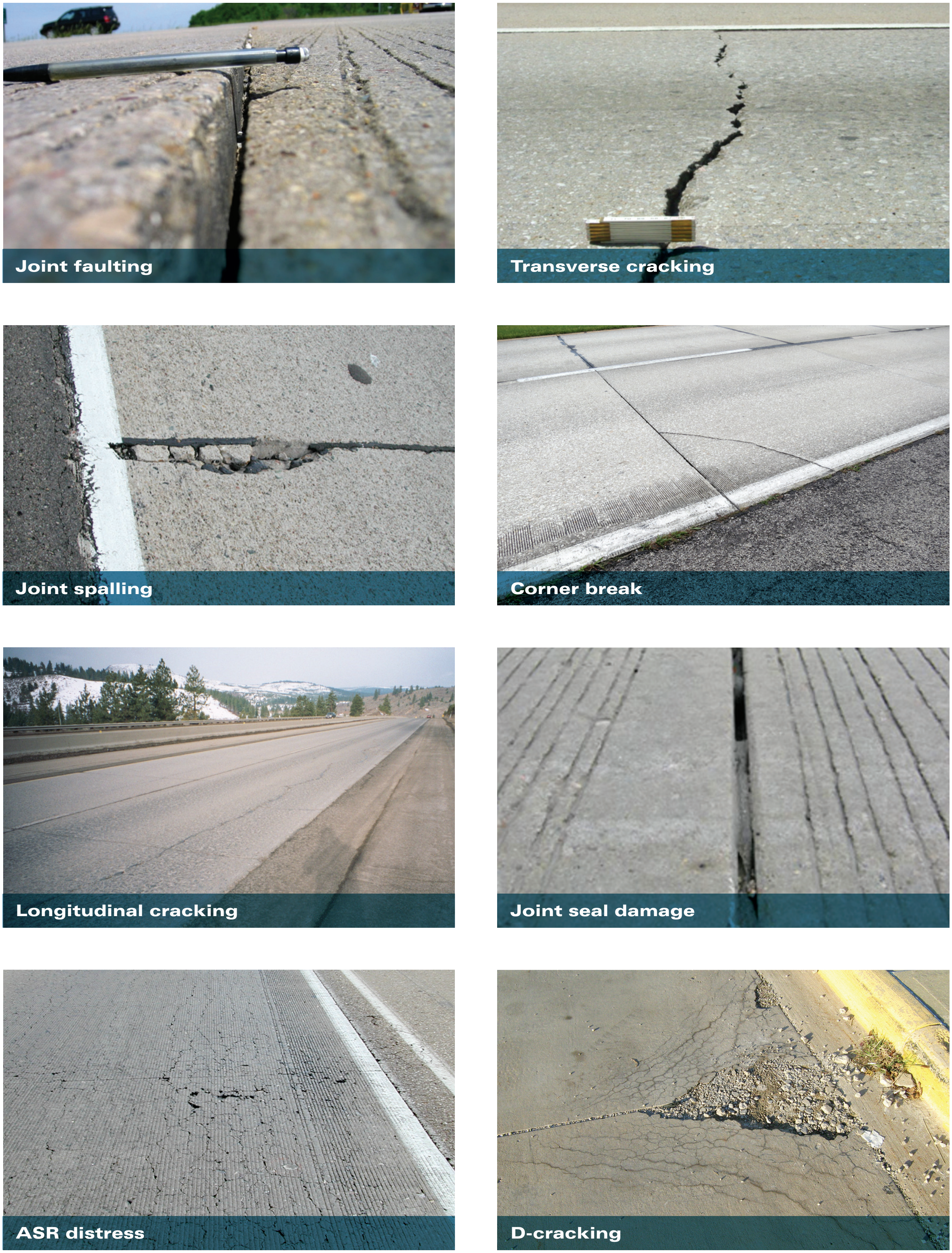

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Figure 3.1. Common concrete pavement distress types

Table 3.3. Concrete pavement distress types as defined in the LTPP *Distress Identification Manual*

Distress type	Unit of measure	Severity levels?	Primarily traffic/load?	Primarily climate/ materials?
Corner breaks	Number	Yes	Yes	No
Durability cracking	Number of slabs, number of square feet affected	Yes	No	Yes
Longitudinal cracking	Length in feet	Yes	Yes	Yes
Transverse cracking	Number, length in feet	Yes	Yes	Yes
Transverse joint seal damage	Number	Yes	No	Yes
Longitudinal joint seal damage	Number	No	No	Yes
Spalling of longitudinal joints	Length in feet	Yes	No	Yes
Spalling of transverse joints	Number, length in feet	Yes	No	Yes
Map cracking	Number, number of square feet affected	No	No	Yes
Scaling	Number, number of square feet affected	No	No	Yes
Polished aggregate	Number of square feet affected	No	Yes	No
Popouts	Number/square feet	No	No	Yes
Blowups	Number	No	No	Yes
Transverse construction joint deterioration	Number	Yes	No	Yes
Faulting of transverse joints/ cracks	Distance in inches	No	Yes	No
Lane-to-shoulder dropoff	Distance in inches	No	No	Yes
Lane-to-shoulder separation	Width in inches	No	No	Yes
Patch/patch deterioration	Number, number of square feet affected	Yes	Yes	No
Punchouts	Number	Yes	Yes	No
Water bleeding and pumping	Number, length in feet	No	Yes	No

Source: Adapted from Miller and Bellinger 2014

Another common pavement distress survey procedure is the pavement condition index (PCI) procedure as defined in ASTM D6433-18, Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys. Extensive work went into the development of a numerical index value that is used to represent a pavement's structural integrity and its surface operational condition based on the observed distress. The resulting index value, the PCI, ranges from 0 (failed pavement) to 100 (perfect pavement) and accounts for the types of distresses, the severity of the distresses, and the amount or extent of the distresses. The associated effects of these factors are combined into a composite PCI value through established weighting factors so that the composite value reflects the overall performance of the pavement (Shahin and Walther 1990).

The PCI procedure is intended primarily for network-level pavement management purposes, not only in documenting the current condition of pavements but also in developing prediction models to forecast future pavement condition (Shahin and Walther 1990). The methodology, however, is sufficiently comprehensive and flexible enough that it can be used in project-level analyses.

Finally, extensive work has recently been conducted by AASHTO, in cooperation with the FHWA, to develop protocols and standards in relation to pavement distress surveys. The AASHTO standards related to concrete pavement distresses include the following:

- AASHTO R 36, Standard Practice for Evaluating Faulting of Concrete Pavements, which provides a method for fault measurements at highway speeds
- AASHTO R 86, Standard Practice for Collecting Images of Pavement Surfaces for Distress Detection, which provides a method for automated collection of pavement surface images for network- and project-level analysis
- AASHTO R 87, Standard Practice for Determining Pavement Deformation Parameters and Cross Slope from Collected Transverse Profiles, which provides cross slope and transverse deformation definitions

Guidelines for Conducting Pavement Distress Surveys

Although modern technology has made automated distress data collection a feasible alternative at the network level, some pavement managers still prefer a manual distress survey at the project level. A manual distress survey is a "walking" survey of the pavement in which the entire limits of the project are evaluated and all distresses are measured, recorded, and mapped. Automated surveys, on the other hand, use specially equipped vehicles that collect video images of the roadway surface and of the drivers' perspective, as well as transverse (used to determine cross slope and surface wear) and longitudinal (used to determine roughness and faulting) profiles, all measured at the posted speed limits.

If an automated survey is conducted, its level of detail should be sufficient to quantify the pavement condition to the degree necessary for preservation treatment type selection. If needed, a limited manual survey can be used to augment the automated survey. However, the detail required for the manual survey must be balanced against the safety risk to the raters in obtaining the data, especially for high-speed, high-volume roadways. In either case, distress surveys serve as a cornerstone in the documentation of pavement condition and in the development of feasible treatment alternatives.

Equipment needed for a manual distress survey is readily available and should include the following:

- Measuring wheel for measuring distances along the project
- String line or straightedge for measuring depressions and/or drop-offs
- Small scale or ruler for fine measurements
- Marking paint or lumber crayon to mark distresses or record stationing along the project
- Mid- to full-sized vehicle
- Fault meter or other means for measuring joint/crack faulting and lane-shoulder drop-off
- Notebook computer or tablet (or data collection forms or sheets) for recording distresses
- Mobile device with a Global Positioning System (GPS) app for recording specific repair locations
- Agency-approved distress identification manual
- Camera for capturing representative distresses and conditions
- Hard hats and safety vests
- Traffic control provisions

Presurvey Activities

Prior to any fieldwork, a preliminary records review should be conducted on the project as outlined in Section 3. Pavement Evaluation Overview, Step 1: Historical Data Collection and Records Review. Complete historical information on the project is desirable, although it may not always be available.

Arrangements for the provision of traffic control should also be made prior to any fieldwork. Although some distress survey work can be performed from the shoulder (or the curb/sidewalk in urban locations), the survey crew must be allowed on the pavement with the freedom to closely inspect the entire pavement. In addition, any sampling and testing activities that will be conducted will require unrestricted access to the entire pavement.

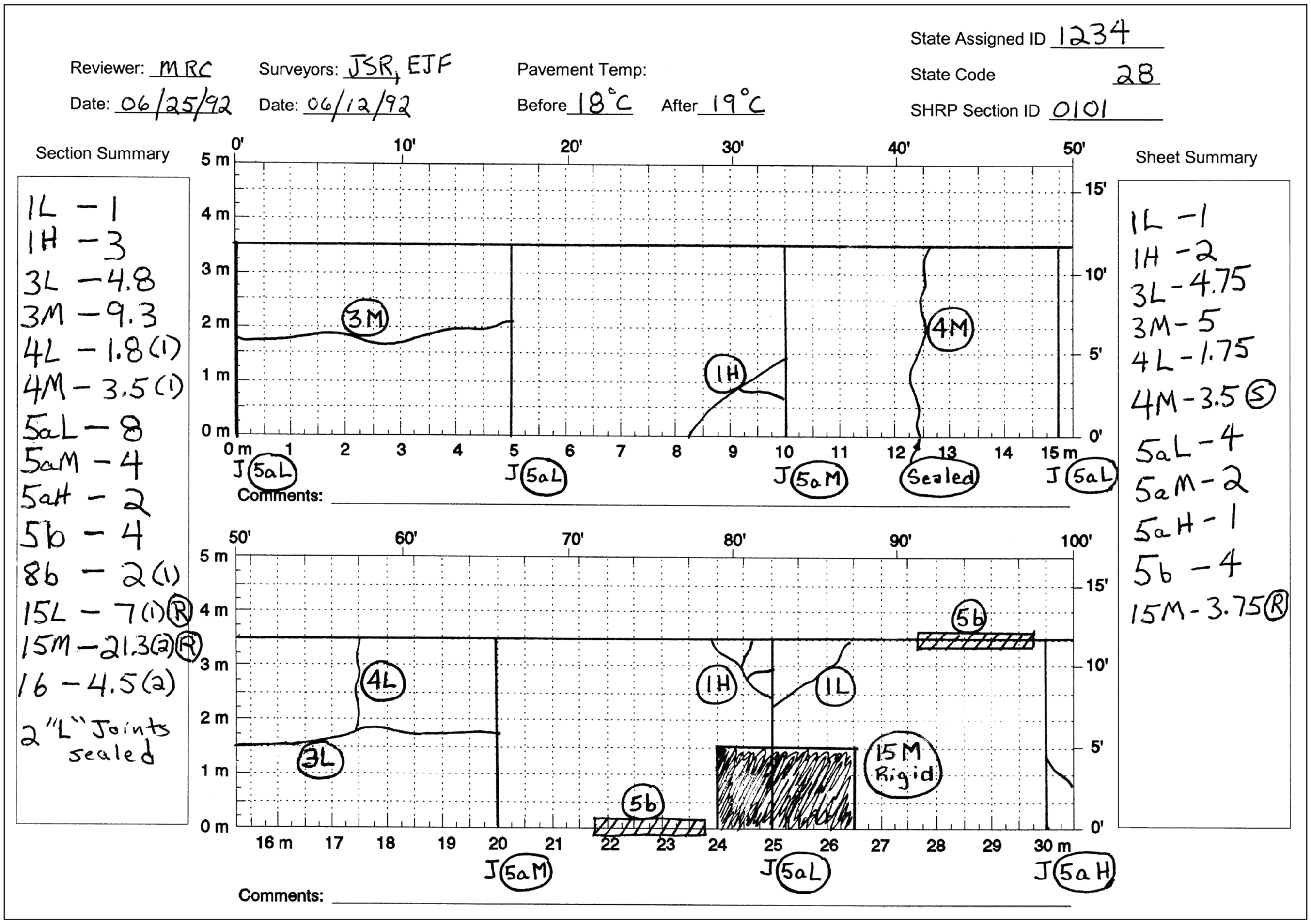
In the case of higher volume roadways, road closures are generally limited to nighttime closures or not at all, depending on traffic volumes and patterns. In these instances, the assessment of pavement condition may be limited to surveys conducted using high-speed data collection vehicles or to windshield surveys. All traffic control arrangements should be scheduled as far in advance as possible and should adhere to the guidelines provided in the *Manual on Uniform Traffic Control Devices* (MUTCD) (FHWA 2009) or the agency's governing requirements.

Manual Distress Survey

As a first step in the manual distress survey, the entire project should be driven in each lane from both directions at the posted speed limit to obtain an overall "feel" or impression for how the pavement is performing. This is also the easiest way to get a measure of the overall rideability of the pavement. During these passes, any swells, depressions, or other sources of discomfort should be recorded and their location noted by milepost. Also, significant changes in overall pavement condition or performance over the length of the project should be noted.

The manual distress survey then follows, typically using a two-person crew that walks or drives along the shoulder or sidewalk to note and record all distresses. In most cases, both travel lanes and shoulders are included in the survey. As previously described, if the project is on a busy roadway and a manual survey is conducted, traffic control is strongly suggested for the safety of the survey crew.

Manual condition surveys have traditionally used paper forms to map the pavement distresses and identify critical repair areas, as shown in the example of a completed field survey form in Figure 3.2.



Miller and Bellinger 2014, FHWA

Figure 3.2. Example of LTPP field data collection form

The project-level distress mapping detail as shown in Figure 3.2 would be prepared for a project proceeding to construction and is more detailed than a preliminary distress survey. Roadway agencies may use the detail of such surveys in the preparation of their construction documents.

Today, more and more agencies are using notebook computers, tablets, or other mobile devices to aid in the collection of distress data. Users can input into these devices directly distress measurement values, which can then be downloaded for further evaluation. Such technologies can be convenient for reducing paperwork and are also effective in reducing transcription errors; some models also allow mapping of actual distresses. Field surveys using computers may proceed at a slightly slower pace than surveys using data collection forms, but the time is made up during data processing since surveys via computer can produce not only maps but also a tabular summary of distresses.

At the conclusion of a manual distress survey, it is recommended that a complete photo or video summary of the project be performed. The purpose of this photo summary is to document the condition of the pavement, as well as to record the prevailing foundation and drainage characteristics of the roadway. There are numerous low-cost technologies available (handheld or vehicle mounted) that collect and record video along with geospatial location.

Automated Distress Survey

Although developed originally for network-level condition surveys, distress surveys employing automated methods are now also used for evaluating project-level distress and pavement condition. Automated surveys are typically conducted using vans equipped with specialized data and video collection equipment. Pavement condition data related to ride and faulting are typically collected using noncontact sensor equipment, whereas surface distress data are typically collected using high-speed, high-resolution cameras or lasers. Recent developments have concentrated on three-dimensional (3D) systems that capture both the image intensity of two-dimensional (2D) systems and data from the 3D range (since it is elevation that identifies cracks, spalls, potholes, etc.) (Pierce and Weitzel 2019). Typical images from a 3D system are shown in Figure 3.3.

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Figure 3.3. Typical images from a 3D data collection van showing a cross-stitched longitudinal crack and the right-of-way (left), the range in the vertical direction (center), and the downward-facing camera view (right)

Data collected from automated distress surveys require postprocessing using either automated or semiautomated methods, as defined below:

- **Automated data processing**—Sensor and video image data are interpreted, reduced, and/or analyzed using computer processing technology. Computer algorithms are used with digital recognition software to identify types and quantities of detectable surface distress.
- **Semiautomated data processing**—Sensor data are analyzed automatically as in the automated data processing methodology. Surface images, however, are viewed by a human using a computer workstation for identifying and quantifying the surface distress information, as shown in Figure 3.4.

Pierce and Weitzel (2019) reported that of 44 survey respondents with jointed concrete pavements, 23 agencies performed fully automated condition and distress surveys, while 22 agencies performed semiautomated condition and distress surveys. (One of the agencies performed both.) For CRCP, 18 agencies performed fully automated condition and distress surveys, while 12 agencies performed semiautomated condition and distress surveys.

Serigos et al. (2014) noted that for equipment tested by the Texas Department of Transportation (TxDOT), there were significant improvements in the accuracy of its distress measurements after they applied manual postprocessing consisting of visual interpretation and correction of the results produced by their systems' algorithms. Additionally, several types of distresses, such as patching, punchouts, spalling, and joint damage, were reported only after manual postprocessing of crack maps. For concrete pavements, the identification and differentiation of joints and transverse cracks is key to identifying, measuring, and calculating faulting.

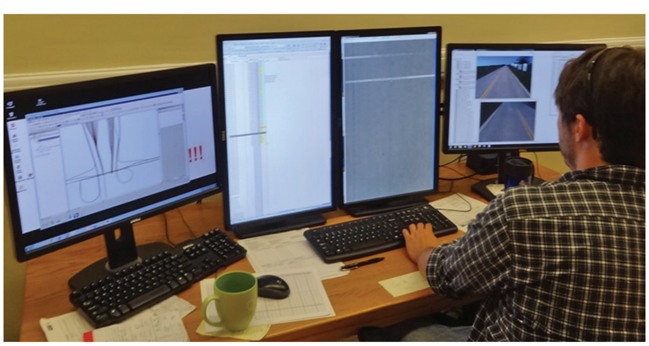

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Figure 3.4. Semiautomated distress processing

In addition to obtaining surface condition information, automated survey vehicles can be outfitted to obtain right-of-way images, grade and cross slope information, GPS coordinates, and 3D images using LiDAR technology.

Most automated survey vehicle manufacturers provide manuals and guidelines for conducting pavement condition data collection. Also, as previously mentioned, AASHTO has developed several standards to provide guidance in this area.

Guidelines for Conducting Pavement Drainage Surveys

As part of a pavement distress survey, it is also important to assess the overall drainage conditions of the existing pavement. This is because poor drainage conditions have long been recognized as a major cause of distress in pavement structures, and unless moisture-related problems are identified and corrected where possible, the effectiveness of any preservation treatment will be reduced. Thus, the purposes of conducting a drainage survey are as follows:

- Identify the presence of moisture-related distresses
- Document the prevailing drainage conditions of the pavement (e.g., cross slopes, cut/fill areas and depths, and condition and depth of ditches or storm drains)
- Assess the condition and effectiveness of edgedrains (if present)

The detection of possible drainage problems as evidenced from a drainage survey may suggest the need for an in-depth analysis of the drainability of the pavement structure. The Drainage Requirement in Pavements (DRIP) computer program (Mallela et al. 2002) can be used to assist in such an analysis and is available from the FHWA and AASHTO (AASHTO 2015). Neshvadian et al. (2017) have reported on pavement moisture infiltration along with needed improvements to the DRIP moisture infiltration model for jointed concrete pavements.

Ideally, the drainage survey should be conducted at the same time as the distress survey. Particular attention should be given to the severity and extent of those distresses that are known to be moisture related to help assess the degree to which excess moisture may be contributing to the pavement deterioration. The location of these distressed areas should be clearly noted. In addition, the following drainage-related items should be recorded as part of the drainage survey:

- **Topography of the project**—The overall topography and the approximate cut/fill depth should be noted along the length of the project to help identify potential distress/topography relationships.
- **Transverse slopes of the shoulder and pavement**—These should be evaluated to ensure that the pavement surface and shoulder are not ponding water or preventing the effective runoff of moisture from the surface. Typical recommendations for pavement surface drainage are a minimum of a 2% cross slope for mainline pavements and a 3% cross slope for shoulders (Anderson et al. 1998).
- **Movement of slabs under traffic**—Accumulated fines or staining on the shoulder next to transverse joints and working cracks are evidence of pumping and possible voids. Slabs may also be seen to visibly move under truck loads during windshield surveys.
- **Condition of the ditches**—The condition of the ditches and the embankment material adjacent to the shoulder should be noted along the length of the project, with inspectors looking for signs of standing water, debris, or vegetation that might otherwise impede the flow of water. The presence of cattails or willows growing in the ditch is a sign of excess moisture.
- **Geometrics of the ditches**—The depth, width, and slope of the ditches should be noted along the length of the project to ensure that they facilitate the storage and movement of water. It is generally recommended that ditches be 4 ft below the surface of the pavement, be at least 3 ft wide, and have a desirable minimum longitudinal slope of 1.5% to 2.0% in urban areas and 1.0% in rural areas (Federal Lands 2018).
- **Condition of drainage outlets (if present)**—These should be assessed over the entire length of the project to ensure that they are clear of debris and set at the proper elevation above the ditch line. The overall condition of the outlets and headwall (if present) should also be assessed and the presence or absence of outlet markers noted.
- **Condition of drainage inlets (if present)**—Many urban projects incorporate drainage inlets to remove surface water, and these should also be inspected over the length of the project. These should be free flowing and clear of debris.

If edgedrains are present, their effectiveness should be evaluated by observing their outflow either after a rainfall or after water is released from a water truck over pavement discontinuities. Another way of assessing the effectiveness of edgedrains is through the use of video inspections (Daleiden 1998, Christopher 2000), in which a camera attached to a pushrod cable is inserted into the drainage system at the outlets. In this way, any blockages, rodent nests, or areas of crushed pipes can be located. Several roadway agencies have adopted video edgedrain inspections as part of new pavement construction.

All of the information collected from the drainage survey should be marked and noted on a strip chart and then examined together to obtain a visual picture of what moisture is doing to the pavement, where any moisture damage is occurring, and what factors are present that allow this moisture damage to occur.

Collective Evaluation of Distress and Drainage Survey Results

Upon completion of the distress and drainage surveys, the critical distresses and drainage conditions should be summarized for the project. One useful way of summarizing the results is through a strip chart that shows the occurrence of various distresses over the length of the project. Primary distresses such as slab cracking are often plotted, but other important performance parameters such as joint load transfer, roughness, texture, and surface friction could also be included. Also, when other important pavement evaluation information—such as deflections, soil types, and traffic volumes—are added to the strip chart, a more complete picture of the overall pavement condition is obtained and additional insight into possible causes of distress is gained. In addition, a strip chart can assist in identifying particularly troublesome areas for more detailed materials and pavement testing.

An example of a strip chart that plots the severity of concrete slab cracking along the length of a project is shown in Figure 3.5.

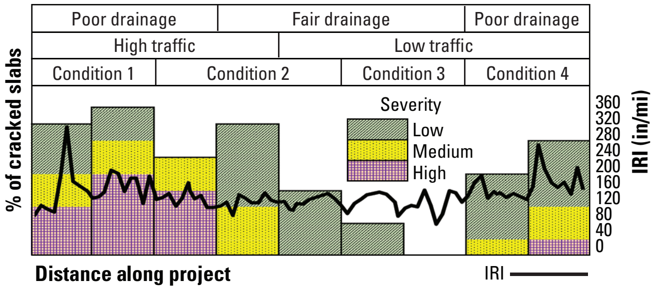

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Figure 3.5. Example of a project strip chart

Three different slab cracking conditions are noted in the strip chart over the length of the project, and it is observed that the worst condition (Condition 1) occurs in an area with high traffic levels and a silty clay subgrade. The best performance is observed in an area with low traffic levels and a granular subgrade. Other factors, such as cut and fill areas, depth of ditches, and condition of drainage outlets (if present), could also be added to the strip chart to provide additional insight.

A summary of the uses for the information obtained from the pavement distress and drainage surveys is listed below:

- Distresses and other deficiencies requiring repair can be identified and the corresponding repair quantities can be estimated. (If there is a significant delay between the collection of pavement distress and drainage field survey data and the construction, a follow-up survey may be needed to ensure that contract quantities are still valid.)
- An overall examination of the data collected over the length of the project will reveal whether there are significantly different areas of pavement condition along the project. In addition, the inner lanes of multilane facilities may exhibit significantly less distress or lower severity levels of distress than the outer lane, suggesting the possibility of targeted lane-by-lane solutions.
- The condition survey data provide permanent documentation of the condition of the existing pavement. This lends itself to several uses, including the monitoring of the pavement performance over time, the comparison of pavement performance before and after treatment, and the development of performance prediction models.
- The data provide an excellent source of information with which to plan structural, functional, and materials testing, if required.
- The pavement distress and drainage survey data provide valuable insight into the mechanisms of pavement deterioration and, consequently, the type(s) of treatment alternatives that may be most appropriate.
- If time-series condition data are available (that is, performance data collected on a pavement at different points in time), then information can be obtained regarding the time that the various deficiencies began to appear and their relative rates of progression. Such information can be extremely valuable in identifying causes of condition deficiencies and in programming appropriate treatment alternatives.

5. Nondestructive Testing

Several NDT technologies are available to assist in the evaluation of concrete pavements. Although surface distress can provide valuable information and indications of structural or subsurface issues, NDT can be used to quantify structural condition, determine layer thicknesses, establish the location of reinforcing steel, and identify the presence and location of underlying voids, thereby providing valuable information in determining the applicability of potential preservation treatments. Table 3.4 provides a summary of selected NDT technologies, each of which is further described in the following sections.

Deflection Testing

Pavement deflection testing is an extremely valuable engineering tool for assessing the uniformity and structural adequacy of existing pavements. Over the years, a variety of deflection testing equipment has been used for this purpose, from simple beam-like devices affixed with mechanical dial gauges to more sophisticated equipment using laser-based technology. Nevertheless, all pavement deflection testing equipment basically operates in the same manner in that a known load is applied to the pavement and the resulting surface deflection is then measured.

Table 3.4. Overview of selected NDT technologies

NDT device	Image	Measurement capabilities				
		Load transfer efficiency	Depth to steel	Layer thickness	Void detection	Structural assessment
Falling weight deflectometer	 CP Tech Center	Yes	No	Possible, if you know portland cement concrete (PCC) modulus	Generally yes (dependent upon temperature and curling of slab)	Yes
Ground-penetrating radar	 Yu 2012, FHWA	No	Yes	Yes	Generally yes (dependent on moisture content at time of testing; better at larger voids that are air filled)	No
MIRA	 FHWA 2017a	No	Yes	Yes	Yes	Yes, to detect delamination
MIT-SCAN2-BT	 FHWA 2017b	No	Yes	No	No	No
MIT-SCAN-T3	 Yu and Khazanovich 2005, FHWA	No	No	Yes, for new construction only	No	No

For concrete pavements, deflection data can be analyzed to provide a wealth of information about the existing pavement structure, including the following:

- Variability in deflections (and, by extension, the base and subgrade support conditions) over the length of a project and by season
- Backcalculation of key material properties (specifically, the concrete elastic modulus and modulus of subgrade reaction [k-value]) for evaluating their variability along the project and for assessing the structural condition of the pavement
- Load transfer efficiency (LTE) across joints and cracks
- Presence of voids under slab corners and edges

The last two items are most pertinent in the assessment of existing concrete pavements for preservation treatments.

Deflection Testing Equipment

At present, several different deflection testing devices are commercially available. The most common one is the FWD. As shown in Figure 3.6, the FWD releases a known weight from a given height onto a load plate resting on the pavement structure, producing a load on the pavement that is similar in magnitude and duration to that of a moving wheel load.

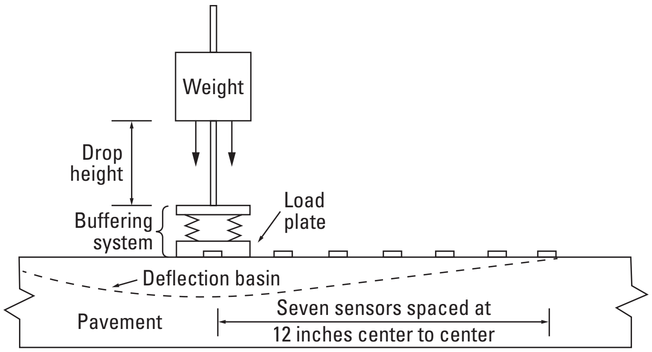

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Figure 3.6. Deflection measurement via the FWD device

A series of sensors is located at predetermined distances from the load plate, so that the deflection basin can be measured. Variations in the force applied to the pavement are obtained by varying the weights and the drop heights; force levels from 3,000 to more than 50,000 lbf can be applied, depending on the equipment. Figure 3.6 shows a typical sensor spacing, but sensor spacing and location can be varied to allow testing on either the approach or leave slab, to measure load transfer across the transverse joint, or to test across longitudinal joints.

Figure 3.7 shows a photo of FWD testing on a concrete pavement with a view of the load plate and sensor bar.

Since 2010, considerable work has been conducted on the development of deflection-measuring equipment capable of collecting *continuous* deflection data along the length of a project. Continuous deflection profiles provide the following advantages over discrete deflection measurements:

- The entire length of the pavement project can be investigated. Thus, there is no danger of missing critical sections and no uncertainty about a test section being representative of the entire pavement system.

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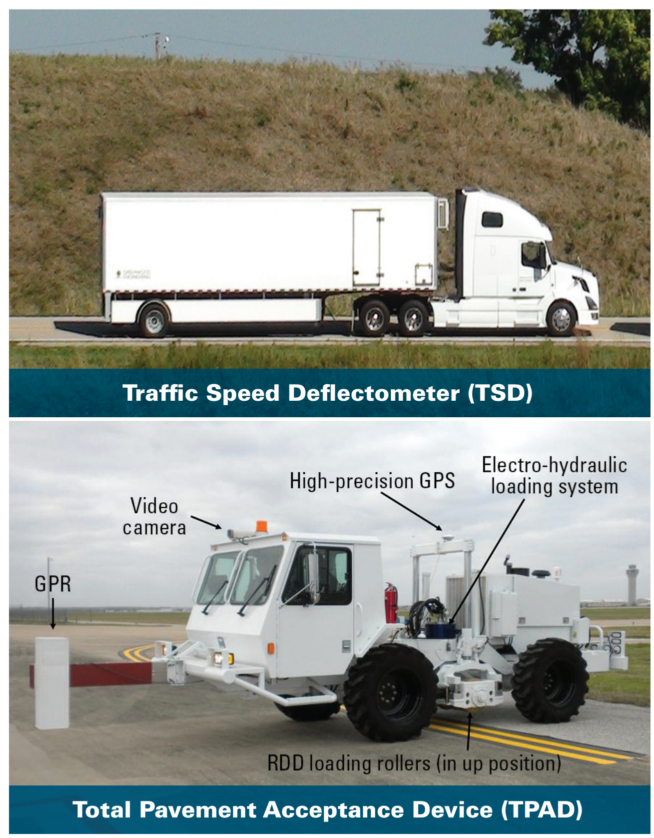
Figure 3.7. FWD showing load plate and sensor bar for joint testing

- The spatial variability in deflections due to pavement features such as joints, cracks, patches, and changing constructed or subgrade conditions are identified.
- More efficient testing and measurement operations are possible since testing equipment does not require stopping and starting.
- Crash risk to testing crew and motorists is reduced.

Currently, two devices (Figure 3.8) are available for the continuous collection of deflection data: the traffic speed deflectometer (TSD) and the total pavement acceptance device (TPAD).

The TSD has generally been applied to flexible pavements and the TPAD to rigid pavements. More detailed information on these devices is available elsewhere (Flintsch et al. 2013, Katicha et al. 2017, Daleiden 2019, Rada et al. 2016, Stokoe et al. 2016, Scullion et al. 2017, and Sivaneswaran et al. 2019).



Recreated from Rada et al. 2016, FHWA (top) and Stokoe et al. 2012, TTI (bottom)

Figure 3.8. Continuous deflection measurement equipment: traffic speed deflectometer and rolling dynamic deflectometer/ total pavement acceptance device

Factors That Influence Measured Deflections

Several factors affect the magnitude of measured pavement deflections, which can complicate the interpretation of the testing results. To the extent possible, direct consideration of these factors should be an integral part of the deflection testing process so that the resultant deflection data are meaningful and representative of actual conditions. The major factors that affect pavement deflections can be grouped into the following categories that should be considered when developing an appropriate testing program for an existing pavement structure:

- **Pavement structural characteristics**—The stiffness of the pavement surface, the layer thicknesses, the slab-base bonding conditions, and the degree and uniformity of support conditions can all influence the magnitude of pavement deflections. Some coring may be needed in conjunction with original construction records to make sure that the pavement cross section and support conditions are well defined.
- **Pavement loading characteristics**—Higher load levels and testing at edges, joints, and cracks will result in larger pavement deflections. Testing at load levels typical of what the pavement will experience is recommended. When performing void analyses, a range of load levels is required to confirm the presence of voids.
- **Climatic factors**—Daily pavement temperatures and moisture content (particularly differences between the top and bottom of the slab) can influence the magnitude of deflections and the measured load transfer efficiencies, as can seasonal variations in temperature and moisture. Testing when the pavement temperature is less than 70°F and, ideally, when the pavement is not curled or warped due to temperature or moisture gradients through the slab is generally recommended. Furthermore, it is recommended that pavement deflections be measured at a time that best represents the effective year-round condition. Another approach is to test at various times during the day to determine if curling and/or warping is present.

Details of the aspects of a concrete pavement deflection testing program are beyond the scope of this document but are available elsewhere (Schmalzer 2006, Pierce et al. 2017). In addition, the following related standards and guides are available from AASHTO and ASTM International:

- AASHTO T 256, Standard Method of Test for Pavement Deflection Measurements
- ASTM D4695, Standard Guide for General Pavement Deflection Measurements
- ASTM D4694, Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device

Interpretation of Deflection Testing Data

Pavement deflection data can be used and interpreted in a number of ways to help characterize the overall pavement condition, as described in the following sections.

Assessment of the Uniformity of the Support Conditions along the Project

The maximum pavement deflection measured at each location can be plotted as shown in Figure 3.9 to illustrate the variation along the project.

The deflections should be referenced directly to project stationing so that they can be related to the distress, drainage, materials, and subgrade surveys. Deflection information is very helpful in identifying subsections within the project where distress, poor moisture conditions, cut/fill, and other conditions may be adversely affecting the pavement.

Backcalculation of Concrete and Subgrade Layer Properties

Backcalculation is the process whereby the fundamental engineering properties of the pavement structure (concrete elastic modulus) and underlying subgrade soil (k-value) are estimated based on measured surface deflections. This information can be used to assess the structural condition of the pavement and to estimate its remaining service life. While the details of the procedures used to compute these parameters are outside the scope of this document, more detailed information on the backcalculation methods for concrete pavements is contained in published reports by AASHTO (1993), Khazanovich et al. (2001), and Pierce et al. (2017).

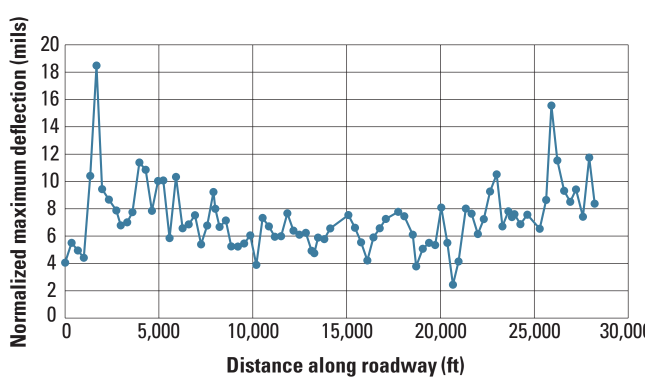


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Figure 3.9. Center slab deflection variation along a project normalized to a 9,000 lbf loading

Evaluation of Joint and Crack Load Transfer

Load transfer is the ability of a joint or crack to transfer the traffic load from one side of the joint or crack to the next. Although load transfer can be defined in several ways, it is commonly expressed in terms of the deflections measured at the joint or crack:

$$LTE = \frac{\delta_U}{\delta_L} \times 100\% \qquad \textbf{(3.1)}$$

Where:

LTE = Load transfer efficiency, percent

δ_U = Deflection on unloaded side of joint or crack, mils

δ_L = Deflection on loaded side of joint or crack, mils

Figure 3.10 illustrates the concept of deflection load transfer.

It should be noted that different LTE values may be obtained depending on which side of the joint is loaded, so it is generally recommended that both sides of the joint be load tested and the lowest value used. Furthermore, temperatures will significantly affect the LTE results, so it is generally recommended that load transfer testing be conducted at temperatures below 70°F. It is important to recognize that accurately establishing the LTE of pavements with very low deflection values can be challenging as error in the measurement may affect the calculated results.

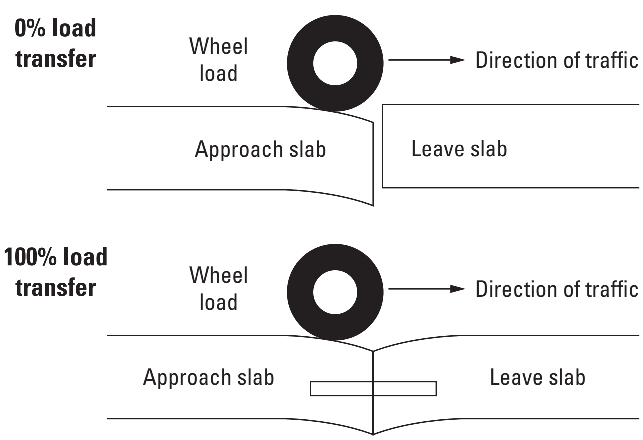

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Figure 3.10. Deflection LTE concept

The following general guidelines may be used to interpret LTE results (ARA, Inc. 2004):

- Excellent: 90% to 100%
- Good: 75% to 89%
- Fair: 50% to 74%
- Poor: 25% to 49%
- Very Poor: 0% to 24%

The magnitude of the corner deflections should also be considered in addition to the LTE. It is possible for slab corners to maintain an acceptable LTE while also exhibiting very high deflections that can still cause pumping, faulting, and corner breaks. Generally speaking, it is desirable that peak corner deflections be less than 25 mils and that the difference in deflections across a loaded joint or crack be limited to 5 mils or less (Odden et al. 2003, Snyder 2011).

Identification of Locations with Loss of Support (Voids)

Loss of support can develop beneath slab corners and edges as the result of high deflections, excess moisture, and an erodible base or subbase. Falling weight deflectometer testing can be performed at suspected void locations to help determine if loss of support exists. In this procedure, a series of loads (typically 6, 9, and 12 kips) are dropped on both the approach and leave sides of a transverse joint, and a load versus deflection plot is generated, as shown in Figure 3.11.

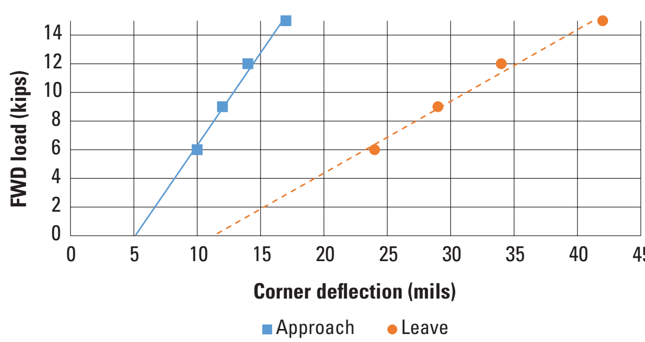

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Figure 3.11. Void detection plot using FWD data

For each testing location (approach side and leave side of the joint), a line is drawn through the points and extrapolated back toward the origin. If no void exists, the line will project very near the origin, typically no farther away than about 2 mils, whereas lines projecting more than that distance from the origin suggest the presence of a void. In Figure 3.11, the results suggest that there is a void beneath the leave side of the joint. (There may also be a void on the approach side, but the leave side is more critical.)

The above method may have difficulties in determining if the void is due to erosion or to curling and/or warping of the slab. Crovetti (2002) used the ratios of foundation support calculated from center slab, edge, and corner FWD testing to determine if the deflections were from erosion or curling. Based on dense-liquid foundation modeling, if the ratio of corner-to-interior foundation support is less than 0.75, nonuniform support is indicated. Rao and Roesler (2005) used FWD testing to estimate the effective built-in temperature difference that represents a significant portion of curling (attributed to the combined effects of nonlinear "built-in" temperature gradients), irreversible shrinkage, moisture gradients, and creep.

Ground-Penetrating Radar

GPR is used primarily to measure pavement layer thicknesses and locate the presence of embedded steel, but it may also be used to identify underlying voids. GPR testing is most effective for identifying layer thicknesses when the dielectric constants or permittivity (a measure of the ability of the material to transmit electrical potential energy) of the individual layers are different. When the dielectric constants of the pavement layers are not significantly different, cores may be required to aid with interpretation of the data.

GPR-estimated layer thicknesses generally are within 3% to 15% of core-measured thicknesses (Maser 1996, 2000). Holzschuher et al. (2007) reported that for nine pavements tested (hot-mix asphalt [HMA] pavements ranging from 2.5 to 13.0 in. and PCC pavements ranging from 6.5 to 8.0 in.), the pavement thicknesses estimated from stationary GPR data resulted in overall average absolute deviations of 0.4 in. for HMA and 0.6 in. for PCC without the aid of calibration cores. These results were further improved to 0.3 in. and 0.4 in. for HMA and PCC, respectively, when cores were used to calibrate the velocities. Measuring at highway speed produced similar results but with a slightly higher standard deviation.

Ground-Penetrating Radar Principles

Ground-penetrating radar technology uses radio wave pulses that are emitted, reflected, and recorded at each testing location. The time and amplitude of the reflected wave pulse can be used to assess the pavement layer thickness, the location of embedded steel, and the presence of underlying voids. As the pavement layer thickness increases, the time duration of the reflected wave pulse also increases; and as the amplitude of the reflected wave pulse increases, the layer moisture content can be interpreted as also increasing (Scullion et al. 1995).

As described above, if the dielectric constants of the paving layers are different, then layer thickness can be readily determined with GPR testing; however, since the dielectric constants of concrete and granular base materials may not be significantly different, the interpretation of layer thicknesses for this type of pavement structure may be difficult (Maser 1996). Because the wave pulses completely reflect metal, however, the location of embedded steel is easy to detect through GPR testing, meaning that GPR can be used to locate dowel bars, horizontally and vertically, in jointed pavements (FHWA 2019), tie bars, and reinforcement depth in CRCP (Durham et al. 2018). The Florida Department of Transportation (FDOT) has utilized 3D radar to identify dowel bars and reinforcing steel in concrete pavements and bridge decks (Chaubane et al. 2018). GPR has also been used in an airport environment to locate utilities and pavement edgedrains.

Ground-Penetrating Radar Equipment

Ground-penetrating radar equipment consists of an antenna, control unit, and data collection computer and software. The antenna can be either air coupled or ground coupled, referring to the location of the antenna relative to the pavement surface. The air-coupled configuration can be used at highway speeds, but it is less able to distinguish between certain materials. The ground-coupled configuration provides a better signal penetration into the ground, but it is limited to slower test speeds because of its contact with the pavement surface.

Applicable AASHTO and ASTM International procedures for GPR testing are the following:

- AASHTO R 37, Standard Practice for Application of GPR to Highways
- ASTM D6432, Standard Guide for Using the Surface GPR Method for Subsurface Investigation
- ASTM D4748, Standard Test Method for Determining the Thickness of Bound Pavement Layers Using Short-Pulse Radar

Ground-Penetrating Radar Interpretation

Analysis and review of GPR signal results require software programs specific to GPR testing. Although the sophistication of the GPR software programs has greatly improved, some interpretation is still required to identify individual pavement layers (Maser 2010). An example of a GPR image illustrating the presence of an underlying void is shown in Figure 3.12.

Scullion (2006) found GPR effective at detecting major defects. In the example shown in Figure 3.12, for instance, there are continuous, strong, multiple reflections. The one gap in the middle of the plot is where a full-depth patch has been placed. This location had earlier been undersealed; however, when this section was cored, it was found that even after the full-depth patching, free water was present beneath the slab. In places, there was a localized 2 to 3 in. thick void beneath the slab.

Magnetic Imaging Tomography

MIT technology can be used to determine concrete pavement layer thicknesses and to identify dowel bar placement and location. Magnetic imaging tomography technology "emits an electromagnetic pulse and detects the induced magnetic field" (Yu and Khazanovich 2005).

The MIT-SCAN2-BT and MIT-DOWEL-SCAN are used for determining dowel bar location only, while the MIT-SCAN-T3 is used for determining the thickness of freshly placed concrete in new construction also.

Although the MIT technology is primarily beneficial for new concrete pavement construction, this technology does have a few preservation treatment applications. For example, the MIT-SCAN2-BT or MIT-DOWEL-SCAN devices can be used for locating existing dowel bars and tie bars prior to conducting full- or partial-depth spall repairs, and they can also be used to determine existing steel location prior to DBR, cross-stitching, or slot-stitching. Each of these devices is further described in the following sections.

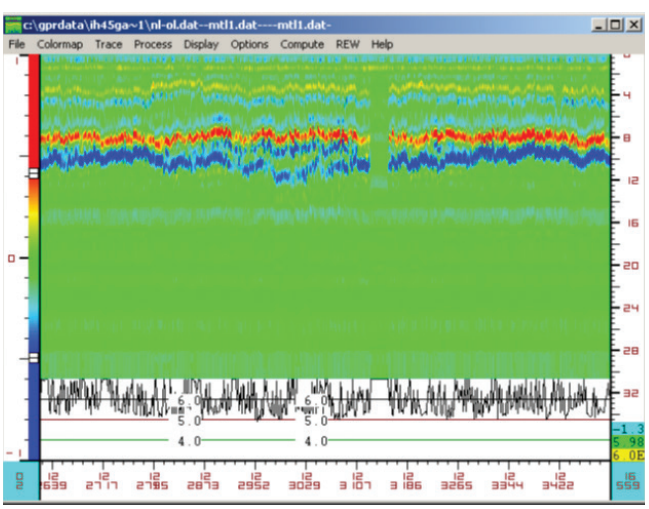
MIT-SCAN2-BT

The MIT-SCAN2-BT device, shown in Figure 3.13, is used in the evaluation of dowel bar placement. It can measure vertical and horizontal misalignments within the following limits (Yu and Khazanovich 2005):

- Dowel bar depth: 3.9 to 7.5 in.
- Dowel bar side shift: +4 in.
- Dowel bar horizontal misalignment: +1.6 in., plus a uniform rotation of +3.1 in.
- Dowel bar vertical misalignment: +1.6 in.

A two-person crew can measure dowel bar placement on 200 or more joints in an 8-hour shift using the MIT-SCAN2-BT device (Yu and Khazanovich 2005). Although the results are not influenced by weather conditions, the operating temperature of the MIT-SCAN2-BT device is 23°F to 122°F. Data analysis can be conducted in real time or stored on a memory card for more detailed analysis later using sophisticated software.

Figure 3.14 illustrates an example of output of the field report from the MIT-SCAN2-BT device.



Scullion 2006, TTI

Figure 3.12. Example of GPR scan indicating underlying void

Yu and Khazanovich 2005, FHWA

Figure 3.13. MIT-SCAN2-BT device

Highway	I20
Station No.	0+31
Bar Spacing	300 mm
Concrete	300 mm
Bar Type	456 X 32.4 mm

Bar No.	Bar Loc.	Bar Spc.	Depth	Side Shift	Alignment	
	mm	mm	mm	mm	mm	mm
1	266	297	130	33	6	0
2	563	304	136	-20	1	-4
3	867	315	139	-15	1	0
4	1182	296	150	1	-4	24
5	1478	303	135	-8	0	9
6	1781	305	140	-19	1	10
7	2086	307	134	-15	2	3
8	2393	297	138	-3	0	4
9	2690	315	143	-42	2	6
10	3005	NA	143	-7	3	1

Yu and Khazanovich 2005, FHWA

Figure 3.14. MIT-SCAN2-BT field report

The input values are shown in the upper portion of Figure 3.14 and include the highway number, the testing (station) location, and the construction specifications for dowel bar spacing, concrete thickness, and dowel bar dimensions. The lower portion of Figure 3.14 provides the results of the MIT scan and includes measurement information related to the dowel bar number (Bar No.), distance from the beginning of the test (Bar Loc.), spacing (Bar Spc.), depth (Depth), side shift, and horizontal (Hor.) and vertical (Vert.) alignment.

The results of the MIT-SCAN2-BT can be used to evaluate the dowel alignment in relation to contract specifications or as part of a forensic investigation. The device does have difficulty evaluating dowel alignment when the basket tie wires are not cut, but many agencies want to leave the tie wires intact to increase basket stability. Additional information on guidelines for specifying dowel bar alignment can be found in Khazanovich et al. (2009).

MIT-DOWEL-SCAN

The MIT-DOWEL-SCAN is an improved version of the MIT-SCAN2-BT. Aicken (2018) has reported that this new equipment does not require that rails be assembled or moved, making this a single-person operation with faster testing. Instead of the rails, a laser is used to guide the device along the joint, thereby greatly simplifying setup and measurement.

MIT-SCAN-T3

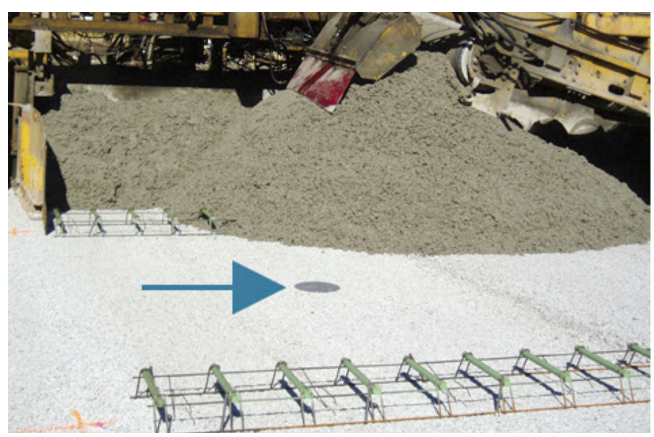
The MIT-SCAN-T3 device, shown in Figure 3.15, is used to determine not only dowel and tie bar location (similarly to the MIT-SCAN2-BT) but also the thickness of a freshly placed concrete pavement in new construction.

The MIT-SCAN-T3 device uses technology similar to the MIT-SCAN2-BT device, but the MIT-SCAN-T3 device requires the placement of a metal reflector prior to paving as part of the testing process to determine concrete pavement thickness (see Figure 3.16).

The MIT-SCAN-T3 device is used to test at the location of the metal reflector to estimate the slab thickness. The MIT-SCAN-T3 device can measure concrete thickness up to 20 in. and is reported to be accurate within a 0.5% tolerance when compared to core measurements (Ye and Tayabji 2009). The results of the MIT-SCAN-T3 are immediately displayed on the device readout screen or can be downloaded to the device software for analysis later.

Yu and Khazanovich 2005, FHWA

Figure 3.15. MIT-SCAN-T3



Ye and Tayabji 2009, FHWA

Figure 3.16. Placement of metal reflector prior to paving

FHWA 2017a

Figure 3.17. MIRA device

Ultrasonic Tomography

There are various stress wave methods used for pavement and subsurface investigations. The MIRA device, shown in Figure 3.17, transmits multiple shear waves through ultrasonic dry point contact sensors in a pitch-catch method and then reconstructs the received wave signals (Germann Instruments 2020).

From the reflected waves and tomographic image processing of MIRA, the operator is able to determine the concrete slab thickness; locate embedded steel; and identify full- or partial-depth cracking, voids, or areas of debonding, joint deterioration, and poor consolidation. Advantages to using MIRA for concrete inspection are that it provides a detailed visualization of the concrete interior, it is nondestructive, it is precise, and no surface preparation is needed (Hoegh et al. 2011, Clayton et al. 2013, CP Tech Center 2013, Popovics et al. 2017, Harrington et al. 2018, Tran and Roesler 2020).

Popovics et al. (2017) found that MIRA used in conjunction with its commercial packages was field ready for the reinforcement localization and thickness measurement of concrete sections. Issues identified during their review, however, included excess dust, occasional signal disruption caused by pavement surface tining/grooving, and the inability to consistently detect shallow delamination (<2 in.). In addition, due to MIRA's limited testing speed, if full coverage of the pavement surface is required, other NDT methods (e.g., GPR or MIT) are better suited. Nevertheless, MIRA can be quickly deployed for initial site investigation to determine concrete thickness and the presence of reinforcement as well as spacing and depth of steel, and to potentially locate concrete voids. (Hoegh et al. [2011] drew similar conclusions on the advisability of applying MIRA to large- versus small-scale testing.)

Another similar technology is the portable seismic property analyzer (PSPA), which integrates two seismic testing methods: impact-echo and ultrasonic surface wave testing. The PSPA has been used to detect common defects in concrete bridge decks and pavements but has not seen extensive use due to several reasons: PSPA data acquisition is slow and labor intensive, its data processing and interpretation are not always straightforward, and its testing protocols have not been well established (Anderson and Li 2014). The PSPA is also not the most appropriate device for determining variations in thickness (which are better provided by GPR or MIRA) but is useful for identifying physically degraded areas in pavements (Anderson et al. 2015).

6. Evaluating Pavement Surface Characteristics

As part of the pavement evaluation process, it is important to assess a pavement's functional performance, which refers to how well the pavement is providing a smooth, quiet, and safe ride to the traveling public. Four easily measurable characteristics that give an indication of a pavement's functional condition are roughness, noise, texture, and surface friction.

Excessive roughness can create user discomfort and irritation and can lead to increased vehicle operating costs, user delay, and crashes (Mamlouk et al. 2018). Excessive noise can be disruptive to the traveling public as well as to adjacent property owners. Texture, inadequate surface friction, and roadway geometry can also contribute to crashes, especially under wet-weather conditions. Higher functional class and operating speed place higher demands on these pavement surface characteristics.

Definitions

This section defines several important roughness-, noise-, texture-, and friction-related terms.

- **Noise levels**—Sound levels are based on a logarithmic scale and are expressed in terms of decibels (dB). For traffic noise measurements, sound levels are adjusted to the human ear and expressed in what is referred to as A-weighted decibels (dBA). The A-weighting scale ranges from a low of 0 dBA to infinity with some key levels being 0 dBA (inaudible), 30 dBA (a whisper), 55 dBA (normal conversation), 90 dBA (a lawnmower), and 140 dBA (the threshold of pain). The human ear, in general, is only able to distinguish a 3 dBA change in similar sounds (Snyder 2006).
- **Pavement roughness**—In its broadest sense, pavement roughness is defined as "the deviations of a surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads, and drainage" (Snyder 2006). Surface irregularities that influence pavement roughness can generally be divided into those that are built into the pavement during construction (e.g., bumps or depressions) and those that develop after construction as the result of developing distresses (e.g., cracking or faulting). Pavement roughness is now commonly expressed in terms of the international roughness index (IRI).
- **Pavement surface friction**—Surface friction is defined as the force that resists the relative motion between a vehicle tire and a pavement surface, and it is influenced by pavement surface characteristics, vehicle operation, tire properties, and environmental factors (Hall et al. 2008). The more critical factors that influence surface friction include the pavement's texture (described in more detail below), vehicle speed, tire tread design and condition, tire pressure, and climate (temperature, water [rainfall or condensation], and snow and ice) (Hall et al. 2008).
- **Pavement texture**—Pavement texture is the feature of the road surface that ultimately determines most of the tire/road interactions, including wet friction, noise, splash and spray, rolling resistance, and tire wear (Henry 2000). Pavement texture is typically divided into categories of microtexture, macrotexture, and megatexture based on wavelength and vertical amplitude characteristics (Gothié 2000, Henry 2000).
    - **Microtexture**—Wavelengths of 0.00004 to 0.02 in. with a vertical amplitude range of 0.00004 to 0.008 in. Microtexture is the surface "roughness" of the individual coarse aggregate particles and of the cement paste, and it contributes to friction through adhesion with vehicle tires. Good microtexture is typically sufficient for adequate friction on dry concrete pavements at normal operating speeds and on wet (but not flooded) concrete pavements at vehicle speeds under 50 mph (Hibbs and Larson 1996).
    - **Macrotexture**—Wavelengths of 0.02 to 2 in. with a vertical amplitude range of 0.004 to 0.8 in. Macrotexture refers to the overall texture of the pavement, which in concrete pavements is controlled by the surface finish (tining). Macrotexture plays a major role in the wet-weather friction characteristics of pavement surfaces, especially at higher vehicle speeds, as it helps to prevent hydroplaning by allowing water to escape from beneath vehicle tires. However, macrotexture is not intended to address surface drainage problems, which are addressed mainly through pavement cross slope (Snyder 2019).
    - **Megatexture**—Wavelengths of 2 to 20 in., with a vertical amplitude range of 0.004 to 2 in. This level of texture is generally a characteristic or a consequence of deterioration of the surface.
- **Powertrain noise**—This refers to noise attributed to the vehicle's engine and exhaust (Cackler et al. 2006). At slower speeds, the powertrain is the predominant source of noise.
- **Present serviceability rating (PSR)**—This is an indicator of pavement roughness based on the subjective ratings of users. The PSR is expressed as a number between 0 and 5, with the smaller values indicating greater pavement roughness (see Table 3.5).
- **Tire-pavement noise**—This is noise attributed to the interaction of the tire-pavement interface as well as vehicle vibration and aerodynamic noise (Cackler et al. 2006). At higher speeds, the tire-pavement noise is the primary source of roadside noise.

Noise Surveys

Tire-pavement noise has emerged as a critical issue on many roadways located throughout the country. Excessive tire-pavement noise levels can be annoying to the traveling public as well as to property and business owners adjacent to roadway facilities. Methods of measuring tire-pavement noise and suggested remedial measures are provided in this section, with more detailed discussion provided elsewhere (Rasmussen et al. 2010).

Table 3.5. Present serviceability rating

PSR	Description
4.0–5.0	Only new (or nearly new) superior pavements are likely to be smooth enough and distress free (sufficiently free of cracks and patches) to qualify for this category. Most pavements constructed or resurfaced during the data year would normally be rated in this category.
3.0–4.0	Pavements in this category, although not quite as smooth as those described above, give a first-class ride and exhibit few, if any, visible signs of surface deterioration. Rigid pavements may be beginning to show evidence of slight surface deterioration, such as minor cracks and spalling.
2.0–3.0	The riding qualities of pavements in this category are noticeably inferior to those of new pavements and may be barely tolerable for high-speed traffic. Rigid pavements in this group may have a few joint failures, faulting and/or cracking, and some pumping.
1.0–2.0	Pavements in this category have deteriorated to such an extent that they affect the speed of free-flow traffic. Rigid pavement distresses that lead to such ratings include joint spalling, patching, cracking, and scaling and may also include pumping and faulting.
0.1–1.0	Pavements in this category are in an extremely deteriorated condition. The facility is passable only at reduced speeds and with considerable ride discomfort. Large potholes and deep cracks exist. Distress occurs over 75% or more of the surface.

Source: After FHWA 2016

Measuring Tire-Pavement Noise—AASHTO T 360

Although several different methods have been used for measuring tire-pavement noise, the primary method used today is AASHTO T 360, Standard Method of Test for Measurement of Tire/Pavement Noise Using the On-Board Sound Intensity (OBSI) Method. The OBSI method utilizes a standard reference tire (ASTM F2493, Standard Specification for P225/60R16 97S Radial Standard Reference Test Tire) and a phase-matched pair of microphones mounted to the outside of a vehicle (see Figure 3.18).

Additional details on measuring and reporting tire-pavement noise using OBSI testing are provided by Rasmussen et al. (2011).

Rasmussen et al. 2010, 2011, CP Tech Center

Figure 3.18. OBSI testing configuration

Reducing Tire-Pavement Noise

While there are a number of features that can be used to construct new, quieter concrete pavements, preservation treatments that may be used, combined or individually, to reduce tire-pavement noise include primarily diamond grinding and thin concrete overlays. A reasonable threshold that defines a quieter concrete pavement, as indicated by OBSI testing measured at 60 mph, is an A-weighted overall sound intensity level between 101 and 102 dB (Rasmussen et al. 2012).

There are several considerations in developing concrete pavement textures that contribute to quieter surfaces; some of the key items, as they apply to diamond grinding and thin concrete overlays, are summarized below (Rasmussen et al. 2012):

- **Surface Texture**
  - Avoid texture patterns with intervals of 1 in. or greater.
  - Impart texture that points down (i.e., grooves) rather than up (i.e., fins).
  - When possible, orient grooves in the longitudinal direction.
  - If used, maintain transverse grooves closely spaced and randomized whenever possible.
- **Concrete Properties**
  - Maintain a consistently strong, durable, and wear-resistant surface mortar.
  - Use a consistent, dense mixture.
  - Use siliceous sands whenever possible.

- Select projects and diamond grinding patterns based on experience and field evaluation so that the final product is both quiet and safe.
- For tined textures, ensure there is an adequate and consistent depth of mortar near the surface to hold the intended geometry.

- **Transverse Joints**
  - Use narrow, single-cut joints.
  - Avoid excess joint sealant, particularly joint sealant that protrudes above the pavement surface.
- **Paving Equipment**
  - Minimize vibrations.
  - Ensure smooth and consistent paver operations.
  - Maintain a constant head of uniform concrete at the proper level.
- **Texturing/Curing Equipment**
  - Minimize vibrations.
  - Minimize the buildup of laitance on the tining equipment.
  - Ensure consistent tracking of texture equipment.
  - Provide multiple passes (or a higher concentration) of the curing application.
- **Grinding Equipment**
  - There does not appear to be an optimum size or spacing of blades and spacers to reduce tire-pavement noise.
  - Larger, heavier grinding equipment is more likely to have the control necessary to consistently impart the texture at the intended depth and lateral coverage.
  - Ensure that the match line between passes of the grinder does not coincide with the wheel path.
  - Ensure that the bogie wheels are true (round).
  - Minimize the variability in the height of the remaining fins of concrete.
  - Avoid excess vibration.

Roughness Surveys

Roughness surveys are an important part of the pavement evaluation process. They can be conducted subjectively (windshield survey—driving over the roadway at the posted speed) or objectively (with roughness-measuring equipment). The primary purpose of a roughness survey is to identify areas of severe roughness on a given project, as well as to provide some insight into its causes. Roughness surveys can also be useful in determining the relative roughness between projects and in gauging the effectiveness of various treatments.

Types of Roughness Surveys

Windshield Surveys

In some cases, a simple windshield survey can be an adequate and valid means of subjectively assessing pavement roughness. This is especially true in an urban setting, although objective roughness testing is easily accessible and economical. A trained surveyor who is familiar with the vehicle they are driving should easily be able to assess pavement roughness, particularly if broad categories of roughness (e.g., not rough, slightly rough, moderately rough, or very rough) are all that is desired from the evaluation. In addition to giving a subjective rating, additional notes should be taken that indicate the estimated sources of the roughness (i.e., roughness due to surface distress [e.g., transverse cracking, corner breaks, faulting, and spalling] versus roughness due to differential elevations [e.g., swells and depressions]). The windshield survey can be augmented by roughness data from cell phones or other handheld devices.

Roughness Testing

Objective roughness testing is conducted using commercially available roughness-measuring equipment. Modern roughness testing is performed using inertial road profiling systems (IRPSs), which measure actual pavement profiles and not a vehicle response to pavement imperfections. Typically, high-speed profilers that are commonly used to monitor the roughness of a pavement network are used for roughness survey testing (see Figure 3.19).

When measuring roughness on concrete pavements with textured surfaces, it is important that the roughness be measured with a line laser instead of a spot laser.

Figure 3.19. High-speed profiler

Karamihas and Gillespie (2002) determined that the conventional spot laser is vulnerable to errors, most notably on longitudinally grooved/tined and diamond-ground textures. The researchers found that the drift of the narrow footprint sensor in and out of the grooves of the pavement surface impacts the profile measurement and is misinterpreted as roughness. Consequently, a line laser (more representative of the tire footprint) is recommended for use when measuring roughness on textured concrete pavement surfaces (ACPA 2013).

To be of most use for the evaluation of a project, it is recommended that the roughness equipment traverse the project in each lane and obtain a representative roughness index for each 0.1 mi increment. Roughness equipment that only measures one wheel path should measure the right wheel path in the direction of traffic for the outer and inner lanes. Special effort should be made to ensure that the equipment is properly calibrated before its use to eliminate potential equipment deviations over time (Sayers and Karamihas 1998). The following AASHTO standards are applicable for quantifying pavement roughness and profile measurements:

- AASHTO M 328, Standard Specification for Inertial Profiler
- AASHTO R 43, Standard Practice for Quantifying Roughness of Pavements
- AASHTO R 57, Standard Practice for Operating Inertial Profiling System and Evaluating Pavement Profiles

AASHTO R 56, Standard Practice for Certification of Inertial Profiling Systems, provides a method of ensuring the IRPS provides repeatable and reproducible results. In addition, the FHWA has developed a *Manual for Profile Measurements and Processing* (Perera et al. 2008) that provides guidance on the calibration of laser profilers for use in its LTPP monitoring program.

One concern when testing on concrete pavements is the effect of daily temperature cycles on the measured roughness (Gillespie et al. 1999). On days when the air temperature changes significantly throughout the day, slab curling effects may be introduced that may cause significant variations in the measured pavement profile over the course of the day. Some advances in IRPS technology, such as laser spot size, have mitigated some of this effect. For project-level profiling, however, several repeat runs of the project at different times during the day may be necessary to quantify the temperature effects.

Types of Roughness Indices

The roughness index to be used on a project is very much dependent on the type of method and type of equipment used to collect the roughness data. One important aspect to remember in selecting an appropriate roughness index is that, ideally, it should be strongly correlated with user response. Provided below are two common indicators used for assessing pavement roughness: the PSR and the IRI.

Present Serviceability Rating

Subjective roughness assessments determined while conducting a windshield survey are typically expressed as ratings of the present serviceability of the pavement. The concept of serviceability was developed as part of the American Association of State Highway Officials (AASHO) Road Test that was conducted in the late 1950s (Carey and Irick 1960, Highway Research Board 1962) and, as previously mentioned, is based on a scale of 0 to 5 (see previous Table 3.5). The PSR was used in the development of the AASHO pavement design procedure and remains an integral part of the 1993 AASHTO procedures for new pavement design and overlay design (AASHTO 1993).

International Roughness Index

The most widely used statistic to describe pavement roughness is the IRI. The IRI is a property of the true pavement profile, and as such it can be computed from the true pavement profile obtained with any valid profiler (Sayers and Karamihas 1998). Furthermore, the IRI provides a common numeric scale for measuring roughness that can be correlated to roughness measurements obtained from both response-type and inertial-based profiler systems (Sayers 1990).

The FHWA has presented guidelines in 23 C.F.R. § 490 (2017) in which good, fair, and poor ride quality for highway pavements are defined by the IRI or by the PSR range as shown in Table 3.6.

Table 3.6. Approximate relationship between IRI and PSR

Ride quality terms	IRI rating (in./mi)	PSR rating
Good	<95	≥4.0
Fair	95–170	>2.0–<4.0
Poor	>170	≤2.0

Source: 23 C.F.R. § 490 (2017)

Surface Friction Testing

The importance of maintaining adequate pavement surface friction is evident as pavement safety continues to be a major concern of most roadway agencies around the world. Based on 2017 data, there were more than 37,000 deaths and 2.7 million injuries in the United States and another 4.5 million crashes that resulted in property damage only (NHTSA 2019). Approximately 21% of these crashes were weather-related.

Nearly 5,000 people are killed and over 418,000 people injured in weather-related crashes each year in the United States. Most weather-related crashes happen on wet pavement and during rainfall: 70% on wet pavement and 46% during rainfall (FHWA 2020). Previous research suggests that 70% of these crashes that occur in wet weather are preventable with improved pavement texture/friction (Larson et al. 2005).

Two primary causes of wet-weather crashes are uncontrolled skidding due to inadequate surface friction in the presence of water (hydroplaning) and poor visibility due to splash and spray (Snyder 2006). Pavement geometry, including longitudinal and transverse profiles, may lead to the ponding of water on the pavement surface and thus contribute to hydroplaning. Moreover, inadequate friction can contribute to accidents in dry weather as well, especially in work zones and intersections where unusual traffic movements and braking action are common.

Historically, pavement friction has been measured directly with different friction-measuring devices and has been expressed as a single number index (i.e., "skid number") (Henry 2000). Recent research, however, has indicated that a single number index for evaluating the friction characteristics of a pavement can be misleading, and it is now recognized that in order to adequately assess pavement friction characteristics, information on the pavement's macrotexture characteristics is also important.

Friction Testing Procedures

FHWA Technical Advisory T 5040.38 (FHWA 2010) recommends that agencies utilize a risk-based approach to determining the frequency and extent of friction testing on the roadway network. The facilities with the highest traffic volumes, the highest likelihood of changes in friction over time, and the highest friction demand (the level of friction needed to safely perform braking, steering, and acceleration maneuvers) justify the most frequent monitoring of friction. Most state transportation departments test in the left wheel path of the driving lane because this is the location where the surface friction is minimum under normal conditions. Test locations should be tied to the milepost markers so that intersections, interchanges, curves, and hills can be identified.

Types of Friction-Measuring Equipment

There are four basic types of highway-speed, full-scale devices used to obtain direct measurements of pavement surface friction. These include locked-wheel, side-force, fixed-slip, and variable-slip testers. Each of these equipment types is described in more detail below.

Locked-Wheel Testers — ASTM E274

Locked-wheel testing devices (see Figure 3.20) simulate emergency braking conditions for vehicles without antilock brakes (i.e., a 100% slip condition).

Most agencies in the United States measure pavement friction with a locked-wheel trailer in accordance with ASTM E274, Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire. In this procedure, the locked-wheel trailer is towed on a pavement that has been wetted with a specified amount of water (0.02 in. film thickness), and then a braking force is applied. Testing can be done with either a ribbed (treaded) or blank (smooth) tire, but results from the blank tire are reportedly better indicators of the pavement's macrotexture (Dahir and Gramling 1990).

Measurements made with the locked-wheel trailer are reported as a skid number—that is, the measured value of the friction coefficient times 100. Skid numbers are reported in the form of SN (test speed [in miles per hour]) followed by an R if a ribbed tire was used or an S if a smooth tread tire was used. If the test speed is expressed in kilometers per hour, it is enclosed in parentheses. For example, if a ribbed tire was used in a locked-wheel trailer test at a test speed of 80 kmph (50 mph), the skid number would be reported as SN(80)R or SN50R (metric and English units, respectively).

Figure 3.20. Locked-wheel skid trailer

Side-Force Testers

Side-force testers are designed to simulate a vehicle's ability to maintain control on horizontal curves. They function by maintaining a test wheel in a plane at an angle (the yaw angle) to the direction of motion, while the wheel is allowed to roll freely (i.e., a 0% slip condition) (Henry 2000). The developed side force (cornering force) is then measured perpendicular to the plane of rotation. An advantage of these devices is that they measure continuously through the test section, whereas locked-wheel devices only sample the friction over the test distance while the wheel is locked (because the wheel is typically locked for only one second before the brake is released) (Henry 2000). Examples of specific side-force testing equipment include the MuMeter and the Sideway-force Coefficient Routine Investigation Machine (SCRIM), both of which originated in the United Kingdom.

The SCRIM averages sideways force measurements from one free-rolling wheel mounted in the wheel path skewed at 20 degrees to the direction of travel (see Figure 3.21).

Some SCRIMs are also fitted with laser macrotexture measurement systems to provide a more complete indication of pavement surface characteristics. The SCRIM can continuously measure friction, macrotexture, and other pavement surface characteristics while being driven up to 50 mph (FHWA 2018).

FHWA from Snyder 2019

Figure 3.21. SCRIM device

Fixed-Slip Testers

The fixed- and variable-slip methods are used to simulate a vehicle's ability to brake while using antilock brakes. Fixed-slip devices operate at a constant slip, usually between 10% and 20% slip (i.e., the test wheel is driven at a lower angular velocity than its free-rolling velocity) (Henry 2000). As with the side-force testers, the largest advantage of using a fixed-slip tester is that these testers can also be operated continuously over the test section without excessive wear of the test tire. Examples of specific fixed-slip testing devices are the GripTester and the SAAB Friction Tester.

Variable-Slip Testers — ASTM E1859

Variable-slip testers are similar to fixed-slip devices, except that instead of using one constant slip ratio during a test, the variable-slip devices sweep through a predetermined set of slip ratios (in accordance with ASTM Standard E1859, Standard Test Method for Friction Coefficient Measurements Between Tire and Pavement Using a Variable Slip Technique) (Henry 2000). An example of a specific variable-slip device is the Norsemeter Road Analyzer and Recorder (ROAR). (This device has not typically been used in the United States for friction testing.)

Pavement Surface Texture

In recent years, it has been recognized that measuring pavement surface texture is necessary to accurately represent a pavement's functional characteristics beyond friction and noise. As described previously, pavement texture is primarily divided into three categories: microtexture, macrotexture, and megatexture. While all three are known to influence a pavement's functional performance, it is the surface macrotexture that is most often assessed with texture-measuring methods.

Traditionally, the sand patch test has been used to assess pavement macrotexture, which produces an indicator of surface texture known as mean texture depth (MTD). To provide adequate surface friction, the average MTD should be 0.03 in. with a minimum of 0.02 in. for any individual test (Hibbs and Larson 1996).

The MTD may be estimated from data collected by vehicle-mounted laser-based measuring devices or by portable devices such as the circular texture meter shown in Figure 3.22. This testing produces measurement data that can be used to compute a mean profile depth (MPD), which is used to estimate the more traditional MTD statistic.

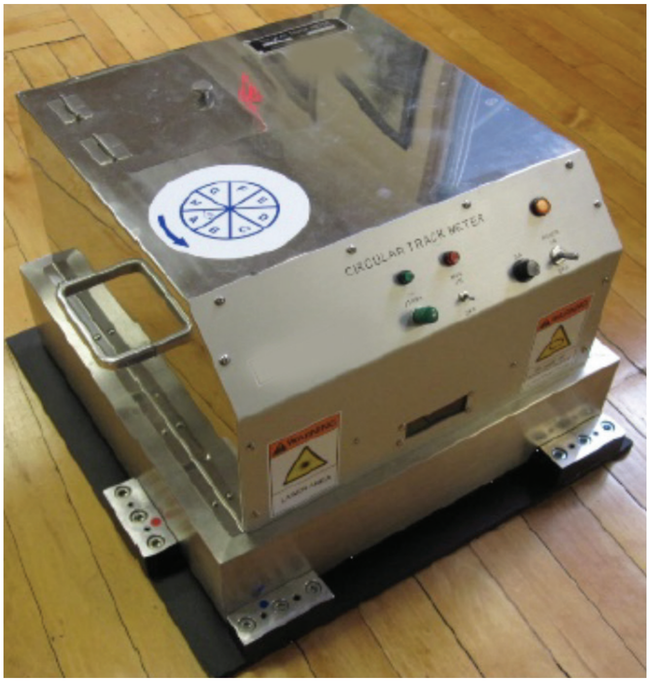


Figure 3.22. Circular texture meter

Additional information relating concrete pavement texture to noise and safety considerations can be found in the FHWA tech brief *Concrete Pavement Texturing* (Snyder 2019).

Evaluation of Noise, Roughness, Friction, and Texture Survey Results

Any collected noise, roughness, friction, and texture data should be evaluated in much the same way as pavement condition survey data. These measured data should be summarized so that a clear picture of the existing functional condition can be obtained by those involved in making design decisions. As with condition survey data, strip charts can be a useful way of showing the various condition deficiencies along a project.

When selecting an appropriate treatment alternative, it is also important to recognize the visible pavement distresses that are indicative of potential roughness or friction problems. For example, common distresses that greatly influence concrete pavement roughness include the following:

- Cracking (i.e., corner breaks, durability cracks, longitudinal cracks, and transverse cracks) and crack deterioration
- Transverse joint faulting
- Transverse joint spalling
- Punchouts
- Patch deterioration

Surface conditions that are indicative of potential surface friction problems include the following:

- Smooth macrotexture that may be the result of inadequate finishing/texturing
- Polishing caused by soft aggregate
- Inadequate pavement cross slopes that result in the slow runoff of water from the pavement surface

It is informative to view these poor friction conditions in conjunction with wet-weather crash data to see if there are any correlations. Overall, the combined results obtained from the roughness and friction assessments can be used to determine if functional improvements are needed.

7. Field Sampling and Testing

Introduction

Most pavement preservation candidate projects will not require field sampling as part of the pavement evaluation process. Some exceptions to this might be projects exhibiting indications of MRD, the presence of unusual or uncharacteristic distresses, or areas suggestive of poor support.

When conducted, the primary purposes of field sampling and testing are to help observe subsurface pavement conditions, to verify pavement layer types and thicknesses, and to retrieve samples for later laboratory testing and analyses. Many different field and laboratory tests are available to determine the subgrade and paving material properties, especially those that are linked to pavement performance. The types and amount of materials sampling and testing are primarily dependent upon the following factors:

- **Observed pavement distress**—The type, severity, extent, and variation of visible distress in a pavement greatly affect the locations and amount of field sampling and testing. If the distress is uniformly spread over the project, sampling is most likely to be conducted in a random (objective) manner. Otherwise, sampling can be targeted in areas of high distress concentrations.
- **Variability**—The variability along the project site will affect the amount of material and sampling required. Projects with greater variability in material properties will require a greater amount of testing in order that this variability can be properly characterized and accounted for.

- **Traffic volume**—The locations and number of allowable samples may be limited on higher speed, higher trafficked roadways because of worker and driver safety concerns. There may also be restrictions related to the rural or urban nature of the roadway with items such as pedestrians, sight distance, utilities, and so on. Such lane closure restrictions and safety-related issues are typically not an issue on roadways with lower traffic volumes.
- **Economics**—Most agencies have a limited budget that determines the types and amount of sampling and testing that can be conducted for a given project. Engineering judgment must be used to determine a sampling and testing plan that minimizes the amount of testing required to adequately assess a pavement's condition while staying within the provided budget constraints. Depending on the apparent variability and the risk associated with the project, requests for the funding of additional fieldwork may be justified in some cases.

The typical field sampling techniques, in situ field testing methods, and standard laboratory testing procedures used in a detailed material investigation are discussed in this section.

Common Field Sampling and Testing Methods

Coring—ASTM C42 and C823

By far, the most common field sampling method is coring, which is the process of cutting cylindrical material samples (cores) from an in-place pavement. Coring is accomplished with the use of a hollow, cylindrical, diamond-tipped core barrel attached to a rotary core drill. The drill is anchored (either to the pavement or to a coring rig) and held perpendicular to the pavement surface while the rotating core barrel is used to slowly cut into the pavement surface. Cores are drilled and retrieved from the pavement and then tested in accordance with ASTM C42, Standard Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete, and ASTM C823, Standard Practice for Examination and Sampling of Hardened Concrete in Construction.

Coring is most often used to determine/verify layer types and thicknesses, as well as to provide samples (concrete slab and stabilized layers only) to test for strength, modulus of elasticity, and coefficient of thermal expansion, as well as possibly for petrographic examination. A visual inspection of retrieved cores can also provide valuable information when trying to assess the causes of visible distress or poor pavement performance. Cores are particularly useful at identifying material consistency problems such as honeycombing in concrete.

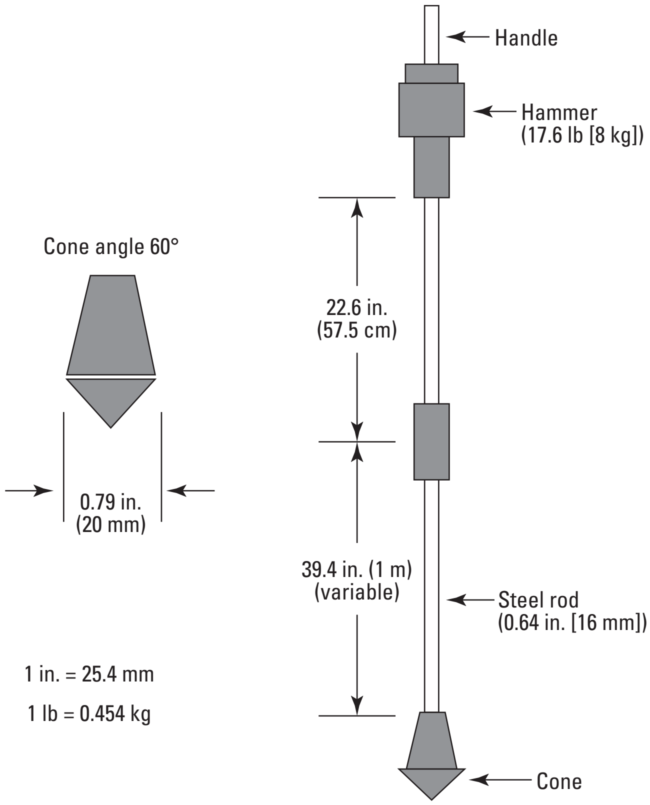
Cores are commonly cut with diameters of 2, 4, or 6 in., the selection of which depends on the purpose. If thickness verification is all that is needed, 2 in. diameter cores are sufficient. Strength testing is most commonly conducted on 4 in. diameter cores; however, a 6 in. diameter core is recommended when the maximum aggregate size is greater than 1.5 in. Although 4 in. diameter cores can be used for petrographic testing, 6 in. diameter cores are often preferred for this purpose.

If desired, material samples of subsurface layers (i.e., subgrade soil, subbase, and base) can be obtained from the core holes. In the event of sampling subgrade materials, the one-call utility notification center should be called prior to coring if any utilities use the right-of-way corridor. Coring should not commence until all utilities have been cleared. Other specialized testing may also be conducted at these locations, such as split-spoon (split-barrel) sampling and Shelby (push) tube sampling.

Dynamic Cone Penetrometer Test—ASTM D6951

The dynamic cone penetrometer (DCP) is a device for measuring the in situ strength of paving materials and subgrade soils. The principle behind the DCP is that a direct correlation exists between the "strength" of a soil and its resistance to penetration by solid objects (Newcomb and Birgisson 1999). The DCP has gained widespread popularity, largely because it is fast, it is easy to use, and it provides reliable estimates of the base or subgrade California bearing ratio (CBR) (Laguros and Miller 1997).

The DCP consists of a cone attached to a rod that is driven into the soil by the means of a drop hammer that slides along the penetrometer shaft (Newcomb and Birgisson 1999) as shown in Figure 3.23.



Adapted from USACE Waterways Experimental Station 1989

Figure 3.23. Dynamic cone penetrometer

The DCP test is performed by driving the cone into the unbound base, subbase, and subgrade through a small core hole in the pavement by raising and dropping the 17.6 lb hammer from a fixed height of 22.6 in. Earlier versions of the DCP used a 30-degree cone angle with a diameter of 0.8 in. (Newcomb and Birgisson 1999). More recent versions of the DCP use a 60-degree cone angle and also have the option of using a 10 lb hammer for weaker soils (Newcomb and Birgisson 1999). In addition, some manufacturers offer a disposable cone that easily slides off the DCP, saving both wear and tear on the device and on the operator while extracting the rod.

During a DCP test, the cone penetration (typically measured in millimeters or inches) associated with each drop is recorded. This procedure is continued until the desired depth is reached. A representative DCP penetration rate (PR) (millimeters or inches of penetration per blow) is determined for each layer by taking the average of the PRs measured at three defined points within a layer: the layer midpoint, midpoint minus 2 in., and midpoint plus 2 in). ASTM D6951, Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications, provides a standard method for DCP testing.

The DCP PRs can be used to identify boundaries between unbound base, subbase, and subgrade strata, as well as to estimate the CBR values of those individual layers. The DCP results are also useful in determining weakening of the aggregate layers due to intermixing with the subgrade soil over time.

Results of the DCP test have been correlated with the CBR for a broad range of material types (including fine-grained soils and gravel). The most commonly used empirical correlations express the CBR as a function of the DCP penetration index (DPI), defined as penetration in millimeters per blow (Newcomb and Birgisson 1999). One of the most widely used correlations between the DPI and CBR is the following developed by Webster et al. (1992) for the manual DCP:

For gravel, sand, and silt: $$CBR = \frac{292}{DPI^{1.12}} \quad \textbf{(3.2)}$$

Where:

CBR = California bearing ratio

DPI = DCP penetration index (measured in mm per blow; 1 in. = 25.4 mm)

Other research has provided variations of this equation that are applicable for heavy and lean clays (Webster et al. 1994).

For highly plastic clays: $$CBR = \frac{1}{(0.002871 * DPI)} \quad \textbf{(3.3)}$$

For low-plasticity clays: $$CBR = \frac{1}{(0.017019 * DPI)^2} \quad \textbf{(3.4)}$$

Another example of an empirical relationship between the CBR and DPI is the following relationship used in Norway (Newcomb and Birgisson 1999):

$$CBR = 2.57 - 1.25 \times \log DPI \quad \textbf{(3.5)}$$

In addition to manual devices, automated DCPs are also available in which the hammer is picked up and dropped automatically. Research results have indicated that CBR values computed using automated DCP results (obtained using the Israeli automated DCP) are about 15% greater than CBR values computed using the DPI computed from the manual DCP (Newcomb and Birgisson 1999).

Plate Load Test—ASTM D1195

When assessing the support conditions of an in situ pavement foundation, plate load testing is used to determine the modulus of subgrade reaction k. The plate load test—outlined in ASTM D1195, Standard Test Method for Repetitive Static Plate Load Tests of Soils and Flexible Pavement Components for Use in Evaluation and Design of Airport and Highway Pavements—consists of a loading device that has support points that are at least 8 ft apart, a hydraulic jack assembly for load application, a bearing plate at least 1 in. thick and 6 to 30 in. in diameter, and a deflection beam that includes three or more dial gauges that record deflection. The testing area should be at least twice the diameter of the chosen bearing plate. Loading is incrementally administered until total deflection has been reached. At each increment, the load and deflection are recorded.

Plate load testing is both expensive and time consuming and it is therefore not commonly conducted, especially for preservation projects. Correlations from FWD testing and other destructive and nondestructive testing are more commonly used to estimate the modulus of subgrade reaction.

Rebound Number of Hardened Concrete—ASTM C805

This test conducted per ASTM C805, Standard Test Method for Rebound Number of Hardened Concrete, determines the rebound number of concrete to evaluate cracked slabs using a spring-driven hammer equipped with a steel plunger. The plunger is held perpendicular to the specimen's surface and the instrument is pressed until the plunger on the hammer impacts the specimen. The rebound number is recorded 10 times for each specimen. By determining the rebound number, areas of inadequate concrete and estimated strength are established.

Ultrasonic Pulse Velocity through Concrete—ASTM C597

Ultrasonic pulse velocity is utilized to determine the presence of voids or cracks within concrete. ASTM C597-16, Standard Test Method for Pulse Velocity Through Concrete, determines the propagation velocity of longitudinal stress wave pulses through concrete. The surface of the concrete in question is put in contact with an electro-acoustical transducer that emits pulses of longitudinal stress. These ultrasonic pulses are converted into electrical energy by a secondary transducer displaced at a certain distance from the transmitter. Based on the transducer distance and time, the pulse velocity is calculated. This information can aid in the determination of suitable repairs.

Common Laboratory Testing Methods

This section presents some of the common laboratory testing methods used in the evaluation of pavement layer materials. The types of tests discussed here are divided into the general categories of material characterization, material strength and strength-related testing, and special concrete materials evaluation.

Material Characterization (for Subsurface Layer Materials)

Collected material samples (e.g., soil samples and granular base samples) are often subjected to a series of standard laboratory tests such as soil classification, gradation, moisture content, and density tests. These tests are primarily run to show whether the properties of the materials have changed since construction. Original construction records containing original test results may be compared with the present condition of each material to determine if any significant changes have occurred that may be suggestive of a problem in the material. The results of these tests should be used in conjunction with other material tests (e.g., strength-related testing) to fully characterize the properties of a material. Some general correlations relating soil classification to traditional measures of subgrade support or strength are provided in AASHTO (1993, 2020), ARA, Inc. (2004), and PCA (1992).

Strength and Strength-Related Testing

The ability of a pavement structure to adequately carry repeated traffic loadings is very much dependent on the strength, stiffness, and deformation-resistance properties of each layer. Strength tests, or tests that are indicative of strength, have long been used to assess the quality of a pavement layer. Measures of elastic or resilient modulus, however, are more relevant because they describe how pavements respond to load. The types of tests used depend on the type of material making up a given layer (stabilized or unstabilized) and the function of the layer (surface, base, subbase, or subgrade soil material).

There are various laboratory testing methods that are used to measure material strength, stiffness, or the ability to resist deformation or bending. Some of the more common tests used in the assessment of paving materials are described in the following sections.

California Bearing Ratio Testing—ASTM D1883

The CBR test measures the resistance of an unbound soil, base, or subbase sample to penetration by a piston with an end area of 3 in²) being pressed into the soil at a standard rate of 0.05 in. per minute. The load resulting from this penetration is measured at given intervals, and the resulting loads at sequential penetrations are compared to the penetration recorded for a standard, well-graded crushed stone. The ratio of the load in the soil to the load in the standard material (at 0.1 in. penetration) multiplied by 100 is the CBR of the soil. CBR values will typically range from 2 to 8 for silts and clays and up to 50 to 70 (or more) for granular bases and high-quality crushed stones (PCA 1992). Additional details on CBR testing can be found in ASTM D1883-16, Standard Test Method for California Bearing Ratio (CBR) of Laboratory-Compacted Soils.

The CBR test is an empirical test that has been used extensively in pavement design. The major advantages of this test are its simple equipment requirements and the database available for correlating its results with field performance. Drawbacks of this test are that it is sensitive to specimen preparation and that it is an empirical test method so does not represent a fundamental soil property.

Hveem Resistance Value (R-Value) Testing—AASHTO T 246/ASTM D1560

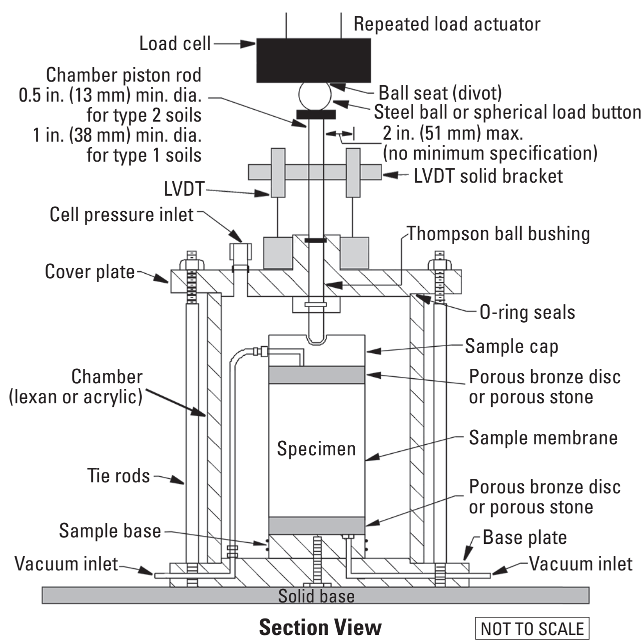
An Hveem Stabilometer measures the transmitted horizontal pressure associated with the application of a vertical load for subgrade and unstabilized aggregate materials (PCA 1992). In accordance with AASHTO T 246, Standard Method of Test for Resistance to Deformation and Cohesion of Hot Mix Asphalt (HMA) by Means of Hveem Apparatus, or ASTM D1560, Standard Test Method for Resistance to Deformation and Cohesion of Bituminous Mixtures by the Hveem Apparatus, the test consists of enclosing a cylindrical sample (4 in. diameter and 2.5 in. tall) in a membrane and loading it vertically over the full face of the sample to a given pressure. The resulting horizontal pressure is measured and used to calculate the resistance value (R-value), which gives an indication of the stiffness of the material. This R-value method has been used by several western state transportation departments, but it is an empirical test method and does not represent a fundamental soil property.

Triaxial Strength Testing—AASHTO T 296/ASTM D2850

The triaxial test is a compressive strength test in which a soil (or unbound material) sample is placed in a triaxial cell and a confining pressure is applied to the sample in the chamber prior to the test. The confining pressure is applied to simulate the confining conditions of the materials in place. A vertical axial load is then applied to the sample until it fails. Several samples are tested under several confining pressure levels to develop a relationship between the vertical load at failure and the associated confining pressure. This test procedure is described in AASHTO T 296, Standard Method of Test for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression, and ASTM D2850, Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils.

Resilient Modulus Testing—AASHTO T 307

The resilient modulus test, as shown in Figure 3.24, provides a material parameter that more closely simulates the behavior of a given material under a moving wheel.



Recreated from Simpson et al. 2007, FHWA

Figure 3.24. Subgrade resilient modulus test apparatus

In the laboratory, the resilient modulus test is conducted by placing in the test apparatus' triaxial cell a compacted material specimen (ideally an undisturbed in situ sample, though it may be necessary to recompact the sample). The specimen is then subjected to an all-around confining pressure, σ_3 or σ_c, and a repeated axial stress (deviator stress), σ_D, is applied to the sample. The number of times the axial load is applied to the sample varies, but it typically ranges from 50 to 200 cycles. During the resilient modulus test, the recoverable axial strain, ε_r, is determined by measuring the recoverable deformations across the known gauge length. The test is run at various combinations of deviator stress and confining pressure, which vary depending on the type of material being tested (i.e., fine grained or coarse grained).

Resilient modulus testing is performed on subgrade soils and on unbound base/subbase materials in accordance with AASHTO T 307-99, Determining the Resilient Modulus of Soils and Aggregate Materials.

Since not all agencies are familiar with the resilient modulus test and the resultant values, it is useful to consider correlations between some of the various material strength indicators. Approximate relationships between the resilient modulus, CBR, and R-value are given below, but these should be taken only as general indicators.

Resilient Modulus versus CBR:

$$M_R = B \times CBR \qquad \textbf{(3.6)}$$

Where:

M_R = Resilient modulus, lbf/in²

CBR = California bearing ratio

B = Coefficient = 750–3,000 (1,500 for CBR < 10)

Resilient modulus versus R-value:

$$M_R = A + B(R) \qquad \textbf{(3.7)}$$

Where:

M_R = Resilient modulus, lbf/in²

R = Resistance value obtained using the Hveem Stabilometer

A = Constant = 772–1,155 (1,000 for R < 20)

B = Constant = 369–555 (555 for R < 20)

Unconfined Compressive Strength Testing—ASTM C39/AASHTO T 22

A very popular test on concrete and other cement- and lime-treated materials is the unconfined compressive strength test. The popularity of this test method is primarily because it is an easy, quick, and inexpensive test to perform and many of the desirable characteristics of concrete are qualitatively related to its strength. The unconfined compression test can also be performed on all stabilized materials used in pavement construction.

For concrete core samples, the test is run in accordance with ASTM C39, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens, or AASHTO T 22, Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens. The test can be performed on cores obtained for slab thickness determination.

Flexural Strength Testing of Concrete Using a Simple Beam—ASTM C78 /AASHTO T 97 or ASTM C293 / AASHTO T 177

There are two potential test methods for determining the flexural strength of concrete using a simple beam. The first method, ASTM C78/AASHTO T 97, Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading), loads the sample beam into an apparatus that utilizes third-point loading and bearing blocks to test the flexural strength of the concrete. The load is applied continuously at a constant, increasing rate until the beam breaks. Upon breaking, the modulus of rupture or flexural strength of the specimen is calculated.

ASTM C293/AASHTO T 177, Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading), is similar to that of ASTM C78 but results in higher values of flexural strength. This test procedure is useful to evaluate cracked or shattered slabs. The load is applied at the center point of the beam and perpendicular to the face of the beam. The specimen is loaded continuously at a constant rate between 125 and 175 lbf/in² per minute until the beam reaches its breaking point. Upon breaking, the modulus of rupture or flexural strength of the beam can be calculated.

This test method is labor- and traffic-control intensive to sawcut the beam from the roadway and repair the void. Care must also be taken in transporting and trimming the beam for testing to ensure that it is not damaged prior to testing.

Elastic Modulus Testing—ASTM C469

Elastic modulus testing is sometimes conducted on concrete core samples to help validate FWD results and as an input into many overlay design procedures. Elastic modulus testing is conducted in accordance with ASTM C469, Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression.

Indirect Tensile Strength Testing—ASTM C496

The indirect tension test, also called the splitting tensile strength test, can be used to determine the tensile strength of concrete cores or any stabilized pavement layer. The procedure is described in ASTM C496, Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens. The test involves applying a vertical load at a constant rate of deformation (0.05 in. per minute) on the diameter of a cylindrical sample. The sample will fail in tension along the vertical diameter of the sample and the indirect tensile strength is calculated from the following equation:

$$\sigma_t = \frac{2P_{ult}}{\pi L D} \qquad \textbf{(3.8)}$$

Where:

σ_t = Indirect tensile strength, lbf/in²

P_{ult} = Vertical compressive force at failure, lbf

L = Length of sample, in.

D = Diameter of sample, in.

This test is particularly valuable for pavement evaluation purposes because it is performed on cores taken from the pavement. As with the compression testing, this test can be performed on cores obtained for slab thickness determination.

Special Concrete Materials Evaluation Tests

In some cases, an existing concrete pavement may be exhibiting MRDs that are compromising the performance of the pavement. MRDs are those distresses that develop due to the concrete's inability to maintain its integrity when subjected to changes in physical (environmental) and chemical mechanisms. MRD is generally visible as cracking or a degradation of the concrete, such as scaling or spalling, often accompanied by some type of staining or exudate. However, some MRD tends to occur at the bottom of joints and may not be visible at the surface.

The occurrence of MRD is a function of many factors, including the constituent materials (e.g., aggregate, cement, and admixtures) and their proportions, the pavement's location (e.g., maritime or inland), the climatic conditions (e.g., temperature and moisture) to which it is subjected, and the presence of external aggressive agents (e.g., roadway deicing chemicals) (Van Dam et al. 2002a). It is not uncommon for combinations of these factors to result in the occurrence of multiple types of MRD in a given pavement section. When multiple MRD types develop together, the process of determining the exact cause(s) of material failure is often complicated. Tables 3.7 and 3.8 summarize details of the most common MRD types, including information regarding their causes, typical time of appearance, and prevention (Van Dam et al. 2002a).

Additional information on the identification, mitigation, and repair of MRD can be found in Weiss et al. (2016) and Harrington et al. (2018).

Table 3.7. Summary of key MRDs due to physical mechanisms

Type of MRD	Surface distress manifestations and locations	Causes/mechanisms	Time of appearance	Method of prevention or reduction
Freeze-thaw deterioration of hardened cement paste	Scaling, spalling, or map cracking, generally initiating near joints or cracks; possible internal disruption of concrete matrix	Deterioration of saturated cement paste due to repeated freeze-thaw cycles	1–5 years	Add air-entraining agent to establish protective air void system
Deicer scaling/ deterioration	Scaling or crazing of the slab surface with possible alteration of the concrete pore system and/ or the hydrated cement paste leading to staining at joints/cracks	Deicing chemicals can amplify freeze-thaw deterioration and may interact chemically with cement hydration products	1–5 years	Limit water-to-cementitious materials (w/cm) ratio to no more than 0.45 and provide a minimum 30-day "drying" period after curing before allowing deicers
Freeze-thaw deterioration of aggregate (D-cracking)	Cracking parallel to joints and cracks and later spalling; may be accompanied by surface staining	Freezing and thawing of susceptible coarse aggregates results in fracturing and/or excessive dilation of aggregate	10–15 years	Use nonsusceptible aggregates or reduce maximum coarse aggregate size

Source: Adapted from Van Dam et al. 2002a

Table 3.8. Summary of key MRDs due to chemical mechanisms

Type of MRD	Surface distress manifestations and locations	Causes/mechanisms	Time of appearance	Method of prevention or reduction
Alkali-silica reactivity	Map cracking over entire slab area and accompanying expansion-related distresses (e.g., joint closure, spalling, and/or blowups)	Reaction between alkalis in the pore solution and reactive silica in aggregate resulting in the formation of expansive gel and the degradation of the aggregate particle	5–15 years	Use nonsusceptible aggregates, add pozzolans to mix, limit total alkalis in concrete, minimize exposure to moisture, and/or add lithium compounds
Alkali-carbonate reactivity	Map cracking over entire slab area and accompanying pressure-related distresses (e.g., spalling and/or blowups)	Expansive reaction between alkalis in the pore solution and certain carbonate/dolomitic aggregates that commonly involves dedolomitization and brucite formation	5–15 years	Avoid susceptible aggregates, significantly limit total alkalis in concrete, blend susceptible aggregate with quality aggregate, and/or reduce size of reactive aggregate
External sulfate attack	Fine cracking near joints and slab edges or map cracking over entire slab area, ultimately resulting in joint or surface deterioration	Expansive formation of ettringite that occurs when external sources of sulfate (e.g., groundwater and/or deicing chemicals) react with calcium sulfoaluminates in the concrete	1–5 years	Use w/cm ratio below 0.45, minimize tricalcium aluminate content in cement, use blended cements, and/or use pozzolans
Internal sulfate attack	Fine cracking near joints and slab edges or map cracking over entire slab area	Formation of ettringite from internal sources of sulfate that results in expansive disruption in the paste phase or fills available air voids, reducing freeze-thaw resistance	1–5 years	Minimize internal sources of slowly soluble sulfates, minimize tricalcium aluminate content in cement, and/or avoid high curing temperatures
Corrosion of embedded steel	Spalling, cracking, and deterioration at areas above or surrounding embedded steel	Chloride ions penetrating concrete, resulting in steel corrosion, which in turn results in expansion	3–10 years	Reduce the permeability of the concrete, provide adequate concrete cover, protect steel, and/or use corrosion inhibitor

Source: Adapted from Van Dam et al. 2002a

When MRD is suspected of playing a role in the premature deterioration of concrete, laboratory tests are essential to help understand the underlying mechanisms at work (Van Dam et al. 2002b). Typical laboratory methods used to characterize a concrete's microstructure include optical microscopy (OM), staining tests, scanning electron microscopy (SEM), analytical chemistry, and x-ray diffraction (XRD).

Optical microscopy using the stereo microscope and the petrographic microscope are recognized as the most versatile and widely applied tools for diagnosing causes of MRD. Specifically, ASTM C457, Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete, can be used to quantify air void size and spacing. Electron microscopy is also becoming more prevalent, especially for chemical identification of reaction products and other secondary phases using energy dispersive spectroscopy (Van Dam et al. 2002b). Analytical chemistry is an effective method of determining some of the key parameters of the concrete (e.g., w/cm ratio and chloride content).

Research has identified joint deterioration in some concrete pavements, generally located in the Midwest. This deterioration is the result of a combination of saturated freezing and thawing along with calcium oxychloride formation as a result of using certain deicer chemicals. It can be exacerbated by inadequate air, improper joint detailing, and poor construction practices. Guide documents for the identification, mitigation, and prevention of this distress are available (Weiss et al. 2016).

ASR is one particularly troublesome MRD that can produce severe performance problems in concrete pavements. As previously described in Table 3.8, ASR can lead to slab cracking, pressure-related distresses such as spalling and blowups, and damage to adjacent structures (e.g., bridges, abutments, and utilities). Some efforts have looked at ways of identifying and managing ASR in existing pavements. For example, a field book for the identification of ASR was produced in 2011 (Thomas et al. 2011), a procedure for evaluating and managing ASR in existing pavements was released in 2010 (Fournier et al. 2010), and a synthesis on ASR mitigation strategies for airfield pavements was also recently published (Smith and Van Dam 2019).

8. Summary

This chapter presented guidelines and procedures for conducting an overall pavement evaluation, which is essential to the identification of appropriate and cost-effective preservation solutions for specific projects. In setting up a pavement evaluation plan, a number of items will need to be considered, including the following:

- Evaluation budget
- Project schedule
- Functional class and traffic level of the roadway
- Urban or rural environment of the roadway
- Available data
- Risk

Pavement evaluation begins with the collection and review of all available historical data associated with a given project. This includes reviewing original design data, construction information, subgrade data, pavement management performance data, and so on. A collective review of these data often provides an engineer with valuable insight into why the pavement is performing the way it is.

A pavement distress survey is the first and most fundamental pavement evaluation procedure. As part of the survey, pavement distress is defined in terms of type, severity, and extent to fully characterize the condition of the existing pavement. By knowing the type of distress, insight as to whether the distress is primarily load related or primarily materials/climate related can be gained, which in turn will assist in the selection of an appropriate treatment alternative.

Drainage surveys are performed as part of a pavement distress survey to assess the overall drainage conditions of the existing pavement. This is because poor drainage conditions have long been recognized as a major cause of distress in pavement structures, and unless moisture-related problems are identified and corrected where possible, the effectiveness of any treatment will be reduced.

Several other field testing procedures are available for evaluating an existing pavement, although some of these procedures may not commonly be needed for an agency's candidate pavement preservation projects. These procedures include deflection testing; noise, texture, roughness, and friction testing; and field sampling and testing.

Nondestructive testing procedures (such as the use of deflection testing devices, GPR equipment, ultrasonic tomography, or MIT) may be conducted as part of a pavement evaluation program to assess the uniformity and structural adequacy of existing pavements. For concrete pavements, deflection data can be analyzed to provide a wealth of information about the existing pavement structure, including the concrete elastic modulus and modulus of subgrade reaction (k-value), seasonal variations in these values, load transfer efficiencies, and the presence of voids under slab corners and edges. Over the years, a variety of deflection testing equipment has been used, with the FWD established as the current worldwide standard.

In addition to determining a pavement's *structural* condition, it is also important to assess a pavement's *functional* characteristics. Functional considerations are those pavement characteristics that identify how well the pavement is providing a quiet, smooth, safe ride to the traveling public. Measurable characteristics that give an indication of a pavement's functional condition include noise, roughness, surface friction, and surface texture. Common methods and equipment used to assess these functional characteristics were also included in this chapter.

Finally, it may be necessary to conduct a more detailed investigation of the in-place materials within a pavement structure. These additional material property data are commonly used to calibrate/verify distress and deflection data, provide material information where NDT data are not available, and help determine the causes of any observed pavement deficiencies. Many of the more commonly used in situ field tests and laboratory test methods were described in this chapter.

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Chapter 4

Slab Stabilization and Slab Jacking

1. Introduction

Pumping and loss of support occurs beneath concrete pavements due to the presence of three factors: an erodible base or subbase, excessive moisture, and significant slab deflections. Poor support conditions can lead to faulting and corner breaks and can be a major contributor to the accelerated deterioration of the concrete pavement.

Slab stabilization, also referred to as undersealing, is used to restore support to slabs by filling voids, thereby reducing deflections and retarding the development of additional pavement deterioration.

Settlements sometimes occur on concrete pavements in areas of poor foundation support. Such settlements not only cause riding discomfort; they also can create large stresses in the slab that can lead to cracking. In some cases, these slabs can be raised back to their original elevation by using pressure to insert a material beneath the settled slabs and raising them back to the desired elevation. This process of raising slabs is referred to as slab jacking.

Slab stabilization should not be confused with slab jacking. The goal of slab stabilization is to restore support but not change the elevation of the slab. Slab jacking involves restoring both the support and elevation of the slab as closely as possible to their original condition.

2. Slab Stabilization

Purpose and Project Selection

Slab stabilization consists of pressure insertion of a flowable material (commonly polyurethane or a cement grout but occasionally asphalt cement) beneath a concrete slab to fill voids and restore full support. Slab stabilization should be performed only at joints and working cracks where loss of support is known to exist. Attempting to stabilize slabs where loss of support does not exist is not only wasteful—it may even be detrimental to pavement performance (Crovetti and Darter 1985, Darter 2017). To be most effective, it is important to perform slab stabilization prior to the onset of pavement damage that may occur due to loss of support (ACPA 1994).

Because loss of support and slab settlement may be caused by a number of different factors (including excessive moisture, poor load transfer at joints, and poor consolidation), slab stabilization performed by itself may not be sufficient to eliminate the problems. If the underlying mechanisms that led to the development of the support or settlement issues are not addressed as part of the treatment process, the same distress conditions will once again resurface (ACPA 1994, Hoerner et al. 2001, Onyango et al. 2018). Thus, candidate pavements should be thoroughly evaluated and the need for additional preservation treatments (e.g., DBR, diamond grinding, and joint sealing) carefully considered.

Limitations and Effectiveness

Over the years, state and local highway agencies have experienced mixed results with slab stabilization. One of the biggest issues has been the ability to accurately identify the presence of voids beneath the slab. When slab stabilization has been conducted where no voids exist, the pumping of the material beneath the slab can induce stress points and increase the rate of pavement deterioration. On the other hand, some agencies have shown that slab stabilization can be an effective technique when performed under the right conditions. For example, a study conducted by the Missouri Department of Transportation (MoDOT) concluded the following (Donahue et al. 2000):

- Slab stabilization and diamond grinding can be an effective concrete pavement rehabilitation technique under the right conditions.
- Evidence of pumping and highly plastic fine-grained subgrade soils with high in situ water contents over an extensive length of the project should eliminate a concrete pavement from being a candidate for slab stabilization/diamond grinding. Slab stabilization in isolated areas may still be effective.
- Retrofitting edgedrains provide little, if any, additional performance benefit to the combination of slab stabilization/diamond grinding.
- Slab stabilization/diamond grinding should not be expected to provide more than 5 years of reasonable service to a concrete pavement with high cumulative traffic loadings.
- Slab stabilization/diamond grinding may provide 10 years or more of service to a concrete pavement with low cumulative traffic loadings.

Overall, the effectiveness of slab stabilization is greatly dependent on the selection of an appropriate project and careful quality control of the construction process.

Good candidate projects are those that may be exhibiting pumping and faulting but are not showing significant structural deterioration, such as corner breaks and cracking. Darter (2017) identified four key aspects of a successful slab stabilization project:

- Limit usage to the slab locations where deflection testing indicates loss of support.
- Provide detailed and effective specifications, special provisions, and standard drawings.
- Use appropriate materials and ensure their effectiveness in reducing deflections at the slab corners and in maintaining full support without pumping and eroding.
- Implement effective inspection/acceptance procedures, including post-stabilization deflection testing to confirm effectiveness.

Darter (2017) noted a typical service life of slab-stabilized JRCP (mostly at working transverse cracks) in Missouri was estimated at 5 to 10 years. In Georgia, the support to JPCP nondoweled joints was restored for 5+ years with no DBR at the transverse joints.

Materials and Design Considerations

Determining the Repair Area

For slab stabilization, the first step in the process is locating the areas of voids beneath the slab caused by the base or subbase deterioration. The following techniques have been used to determine whether loss of support has occurred beneath a concrete pavement slab:

- **Deflection testing**—This is the most common method for identifying loss of support and one that is very effective. As described in Chapter 3, deflection testing is typically performed using an FWD, but it is important that deflection testing be conducted when the ambient temperature is below 70°F in order to minimize the impact of slab curling (which could erroneously indicate a void) and joint lockup. Ways of using collected deflection data for void detection include the following:
  - Measure and plot the profile of both the approach and leave corner deflections. As voids first form under the leave corner, it is normal to find that the approach corner deflection is less than the leave corner deflection. If this difference is great, then the presence of a void is likely (Darter et al. 1985, AASHTO 1993a). This procedure recommends the identification of a corner deflection value above which slab stabilization is warranted. For example, in Figure 4.1, a reasonable value might be 0.03 in.
  - Measure the magnitude of the corner deflection of the pavement using three different load levels. Three load levels are required so that a load versus deflection plot can be generated. For most highway pavements, load levels of 6,000, 9,000, and 14,000 lbf have typically been used for each test location to develop load versus deflection plots (Crovetti and Darter 1985). Using these deflection data, a line is plotted and extrapolated back toward the origin; lines passing through or very near the origin on these charts suggest that full support exists under the slab corner. Figure 4.2 shows the results before and after slab stabilization (i.e., undersealing) for a project in Missouri (Darter 2017). In these examples, the deflection plots before slab stabilization intercept the x-axis at 13 mils for the third approach slab and 7 mils for the second approach slab. After slab stabilization, both plots intercept the x-axis near zero.

This method, however, may have difficulties in determining if the void is due to erosion or to curling and/or warping of the slab. Crovetti (2002) used the calculated corner-to-interior and edge-to-interior slab support ratios from FWD testing to determine if the deflections were from erosion or curling. Based on dense-liquid foundation modeling, if the ratio of corner-to-interior foundation support is less than 0.75, nonuniform support is indicated. Rao and Roesler (2005) used FWD testing to estimate the effective built-in temperature difference that represents a significant portion of curling (attributed to the combined effects of nonlinear "built-in" temperature gradients), irreversible shrinkage, moisture gradients, and creep.

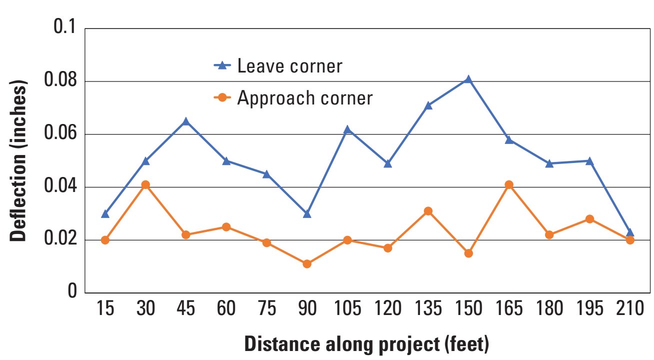

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Figure 4.1. Approach and leave corner deflection profiles

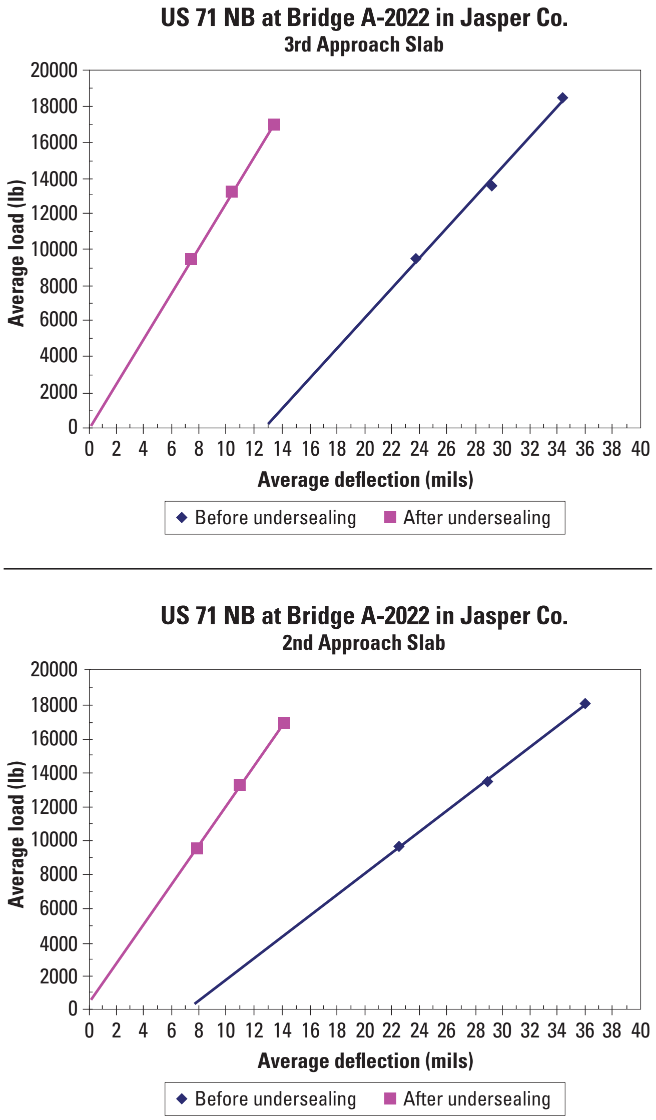

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Figure 4.2. Slab corners in a Missouri project tested before and after slab stabilization that show impact of stabilization on restoring support

- **Ground-penetrating radar**—GPR equipment and data interpretation techniques have enabled the detection of air-filled voids as small as 0.25 in. thick, although the detection of water-filled voids is more difficult (Morey 1998, Maser 2000). An example of a GPR image illustrating the presence of an underlying void is shown in Figure 4.3. Scullion (2006) found GPR effective at detecting major defects. In the example shown in Figure 4.3, there are continuous, strong, multiple reflections. The one gap in the middle of the plot is where a full-depth patch has been placed. This location had already been undersealed; however, when this section was cored, it was found that free water was present beneath the slab. In places, there was a localized 2 to 3 in. thick void beneath the slabs.

Clearly, the rehabilitation options for this highway are limited because of the presence of the water. Chapter 3 provides additional information on GPR testing.

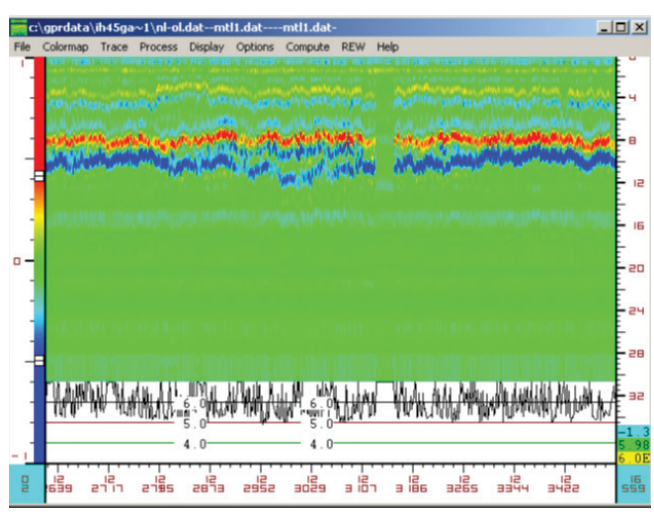
- **Visual observations**—Faulting of transverse joints and cracks, pumping, corner breaks, and shoulder drop-off all indicate that loss of support has occurred (ACPA 1994). Ideally, slab stabilization should be conducted before the onset of significant void development; at later stages, more substantial preservation treatments (e.g., FDRs) are required.

Selecting an Appropriate Injection Hole Pattern

After identifying any voids that would benefit from slab stabilization, the next step is to determine the optimal locations of grout insertion holes (i.e., the hole pattern). The hole pattern is dependent on several factors, including the following (ACPA 2003):

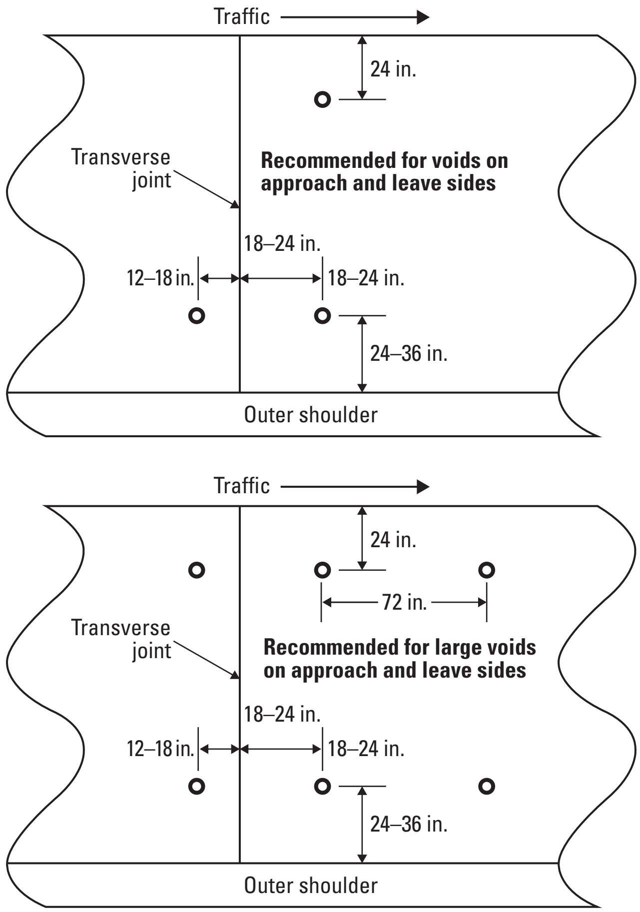
- Pavement type (i.e., JPCP, JRCP, or CRCP)
- Transverse joint spacing (for jointed pavements)
- Estimated size and shape of the detected void
- Flowability of the material being used
- Location of cracks and joints near the void
- Slab condition

Holes should be placed as far away from nearby cracks and joints as possible, but they should still be within the area of the identified void. Moreover, the holes should be placed sufficiently close to one another to achieve a flow of slab stabilization material from one insertion hole to another when a multiple-hole pattern is used (see Figures 4.4 and 4.5).



Scullion 2006, TTI

Figure 4.3. Example of GPR image of underlying void



Recreated from ©2022 Applied Pavement Technology, Inc., after ACPA 1994 line drawings, used with permission

Figure 4.4. Typical grout insertion hole patterns for slab stabilization of jointed concrete pavements

John Donahue, MoDOT, used with permission

Figure 4.5. Polyurethane stabilization hole pattern

Additional details on hole patterns are available from Darter et al. (1985) and ACPA (1994). It is noted that, in some cases, the slab stabilization may be needed only on the leave (downstream) side of the joint, whereas in other cases slab stabilization may be needed on both the approach (upstream) and leave sides. Holes should be drilled to deliver slab stabilization material into the void. When the pavement is bonded to a cement-treated or other stabilized base material, the grout holes should be drilled completely through the base material (MnDOT 2006).

Selecting an Appropriate Material

The material chosen for slab stabilization must be able to penetrate very thin voids while having the strength and durability to withstand pressures caused by traffic, moisture, and temperature. Cement grout has been used commonly in the past but today expansive, high-density, two-component, water-resistant polyurethane material is the primary slab stabilization material being used (Brown and Reed 2014). Other slab stabilization materials that have been used include asphalt cement, limestone dust–cement grouts, and silicone rubber foam, but these are seeing far less use today.

Polyurethane

Polyurethane materials have become the most common materials for use in slab stabilization and slab jacking. Polyurethane materials are made of two liquid chemicals that combine under heat to form a strong, lightweight, foam-like substance. After being injected beneath the pavement, a reaction between the two chemicals causes the material to expand and fill any existing voids (ACPA 1994). For slab stabilization purposes, the polyurethane density is about 3 to 4 lb/ft^3 and the compressive strength ranges from about 60 to 145 lbf/in^2 (ACPA 1994). One laboratory study indicated that the injected polyurethane will consistently penetrate openings as small as 0.25 in. and will penetrate some openings as small as 0.125 in. (Soltesz 2002). Hydrophobic polyurethane materials are preferred for filling voids since they repel water, and the cured structure is unaffected by the presence or absence of water.

Polyurethane materials offer several advantages for use in slab stabilization and slab jacking, including the following (ACPA 1994, Soltesz 2002, Gaspard and Morvant 2004, White et al. 2015, Vennapusa et al. 2016):

- Light weight (so does not contribute to additional settlement)
- High compressive and tensile strengths
- Expansive (able to fill surrounding voids)
- Insensitive to moisture (if hydrophobic polyurethane materials are used)
- Rapidly curing (opening times of 15 to 30 minutes)
- Improved load transfer at cracks

Several highway agencies, including Oregon (Soltesz 2002), Missouri (Donahue et al. 2000, Darter 2017), and Kansas (Barron 2004), have had success in using polyurethane foam for slab stabilization. Vermont has achieved similar success in stabilizing slabs but has noted difficulties in estimating quantities, resulting in a substantial overrun (Ellis 2015). For slab jacking, the Wisconsin Department of Transportation (WisDOT) has reported that the lifting process has been successful but has also indicated shortcomings in the ability to estimate material quantities (Abu al-Eis and LaBarca 2007).

Other agencies have reported varying degrees of success. In Louisiana, for example, a study was conducted in which polyurethane was used to stabilize CRCP, JPCP, and bridge approach slabs (Gaspard and Morvant 2004). The initial results of this study found the material to be an effective method of leveling CRCP and bridge approach slabs, but the JPCP results were not as positive. Although it was determined that the polyurethane did fill the voids, the material did not appear to provide much support to the joints because the joints were observed to still be deflecting under traffic loadings; however, it was also reported that the load transfer devices in this pavement were not functioning for the project in question (Gaspard and Morvant 2004). The Tennessee Department of Transportation (TDOT) applied high-density polyurethane foam to five sections of Interstate highways and concluded that the application neither improved nor degraded the existing pavement (Fortunatus 2018).

Cement Grout Mixtures

Cement-based grout mixtures are typically a pozzolanic cement material. The flowability of these mixtures is assessed through a flow test in which the time it takes for a fixed amount of material to flow through a standard flow cone is measured; the flow cone time typical for fly ash grouts is in the 10- to 16-second range (for comparison, water has a flow cone time of 8 seconds) (ACPA 2003).

The following is a typical mix design for a pozzolanic cement grout for use in slab stabilization (ACAA 2003, ACPA 2003):

- One part by volume cement Type I or Type II (Type III may be specified if there is a need for early strength)
- Three parts by volume pozzolan (Class F fly ash; it may be possible to reduce the cement component if Class C fly ash is used); pozzolans shall conform to the requirements of ASTM C 618, if used
- Water (usually about 1.5 to 3.0 parts by volume) to achieve required fluidity
- If ambient temperatures are below 50°F, an accelerator may be used (if approved).
- A minimum compressive strength (600 lbf/in² at 7 days) is normally required to ensure the durability of the grout; the ultimate strength of the grout will typically be much higher (on the order of 1,500 to 4,000 lbf/in²).
- Additives, superplasticizers, water reducers, and fluidifiers as needed

Overall, a thorough testing regimen should be instituted to ensure the suitability of the grout prior to the start of any slab stabilization project. The contractor should be able to verify the chemical and physical properties of the pozzolan; 1-, 3-, and 7-day compressive strength tests; flow cone results; time of initial set; and shrinkage/expansion results.

Asphalt

Asphalt for slab stabilization requires a material that will flow into voids at a high temperature and pressure. ASTM D3141, Standard Specification for Asphalt for Undersealing Portland-Cement Concrete Pavements, describes typical asphalt material requirements.

Construction Considerations

Step 1: Drilling of Injection Holes

Any handheld or mechanical drill that produces clean holes with no surface spalling or breakouts on the underside of the slab is acceptable for creating the injection holes (ACPA 1994).

For polyurethane slab stabilization, handheld electric-pneumatic rock drills are typically used to drill the injection holes to help reduce slab breakout on the underside of the slabs (ACPA 1994). For these procedures, the maximum hole diameter should not exceed 0.625 in. (ACPA 1994). The use of smaller holes grouped together every 4 to 5 ft provides more uniform coverage (Darter 2017). Figure 4.6 shows holes being drilled for polyurethane injection.

For cement-based grout projects, any pneumatic or hydraulic rotary percussion drill that can cut 1.25 to 2.0 in. diameter holes through the slab is suitable. A general specification recommends limiting the downward force on any drill to 200 lbf to avoid conical spalling at the bottom of the slab (ACPA 1994). When large pieces spall on the underside of the slab, those pieces can potentially block the void and make it impossible to fill.

A quick check of whether or not the hole should be grouted may be made by first pouring water into the drill hole (note that the water does not create a problem as it is displaced when the undersealing material is pumped into the hole). If the hole does not take water, there is no void and therefore no need for undersealing. When it is determined that there is no void, the hole can be filled with an acceptable patching material and the operation can proceed to the next hole.

While the typical injection hole pattern is determined during the design process, the location of the injection holes may need to be adjusted in the field to effectively fill each void. If flow is easily achieved, the hole spacing may be increased. Conversely, if good flow is not achieved before maximum back pressure is reached, the hole spacing should be reduced.

Step 2: Material Preparation

Most slab stabilization contractors use mobile, self-contained equipment that carries all the tools and materials needed for slab stabilization (ACPA 1994). The differences between preparing polyurethane and cement-based materials are discussed in this section.

WSDOT

Figure 4.6. Drilling polyurethane injection holes

Polyurethane

When using polyurethane, all material is stored, proportioned, and blended within a self-contained pumping unit. It is pressurized through a mixing nozzle where the two components combine and begin to set in 30 seconds. The handling and usage of these materials should be in accordance with the material manufacturer's instructions and specifications.

Cement Grout Mixtures

For cement grout mixtures, a grout plant that is capable of accurately measuring, proportioning, and mixing the material by volume or weight is used. When working with pozzolanic cement grouts, it is recommended that contractors use colloidal mixing equipment. Whenever possible, contractors should avoid using paddle-type drum mixers with pozzolanic cement grouts (ACPA 1994). This is because the low agitation of these mixers makes it very difficult to thoroughly mix the grout. Conveyors, mortar mixers, or ready mix trucks should not be used to mix any type of stabilization material because these mixers require adding too much water for fluidity, and the solids in those mixers tend to agglomerate and clump in the mix (ACPA 1994).

Step 3: Material Injection

Because the injection procedures differ slightly by material type, specific procedures associated with each material type are described separately below.

Injection of Polyurethane

The injection process for polyurethane uses pumping equipment specific to the use of the polyurethane material. The pressure and temperature control devices found on this equipment can maintain proper temperature and proportionate mixing of the polyurethane component materials. In addition, the polyurethane undersealing operations use injection equipment consisting of plastic nozzles that screw onto hoses and deliver the material into the holes (ACPA 1994). Also, as previously described, the injection of polyurethane materials uses a smaller injection hole, typically 0.625 in.

After the injection has been completed, the excess polyurethane material is cleaned from the area and the hole can be left unpatched because of its small size (and since it will be already filled with the polyurethane material). Traffic can be opened on the roadway in as little as 15 to 30 minutes. Figure 4.7 shows a photo of the polyurethane injection process.

Injection of Cement Grout Mixtures

It is recommended that positive-displacement injection pumps or nonpulsing progressive-cavity pumps be used for cement grout slab stabilization. It is important that the pump be capable of maintaining low pumping rates and injection pressures. Specifically, a pump should work well if it maintains pressures between 25 and 200 lbf/in^2 during grout injection (ACPA 2003). Maintaining a low pumping rate (ideally about 1.5 gallons per minute) and low pumping pressure ensure better placement control and lateral coverage, and this also usually keeps the slab from rising (AASHTO 1993b). Typical pumping pressures are in the 40 to 60 lbf/in^2 range (ACPA 2003).

Cement-based grouts are typically injected using a grout packer to prevent material extrusion or backup during injection. Two types of grout packers are used, depending on the size of the hole. Drive packers are pipes that taper and fit snugly into the injection hole by tapping with a small hammer (ACPA 2003). Drive packers are generally used with 1 in. diameter holes. Expandable packers consist of a threaded inner pipe, a thin-walled steel outer sleeve, and a short rubber sleeve at the bottom (near the nozzle) that expands to fill the hole during injection (ACPA 2003). Expanding rubber packers require 1.5 in. or larger diameter holes (ACPA 2003).

The injection equipment should include either a return hose from the injection device (packer or tapered nozzle) to the material storage tank or a fast-control reverse switch to stop grout injection quickly when slab movement is detected on the uplift gauge (ACPA 2003). A grout-recirculation system also helps eliminate the problem of grout setting in the injection hoses because the grout circulates back to the pump after pumping ceases (Darter et al. 1985). It is generally recommended that the cement grout not be held in the mixer or pump hopper for more than 1 hour after initial mixing.

After grouting has been completed, the packer is withdrawn and the hole is plugged immediately with a temporary wooden plug. When sufficient time has elapsed to permit the grout to set, the temporary plug is removed, and the hole is sealed flush with an acceptable patching material (see Figure 4.8).

It should be noted that some highway agencies do not require the holes to be plugged as a means of allowing the pressure to dissipate and the slab to settle.

WSDOT

Figure 4.7. Injecting polyurethane for slab stabilization

WSDOT

Figure 4.8. Patching drill holes

Slab stabilization should not be performed when the ambient temperature is below 40°F. Unless a fast-setting material is used, traffic should be kept off a stabilized slab for at least 3 hours after grouting to allow adequate curing of the grout (Darter et al. 1985).

Injection of Asphalt

Asphalt undersealing consists of pumping 375°F to 450°F liquid asphalt under pressure beneath the concrete pavement on both sides of the joint. Safety equipment is required to protect traffic and crew from the hot liquid asphalt that may squirt out of the pavement. The roadway cannot be opened to traffic until the asphalt cools, which can be a minimum of 30 minutes after pumping. Figure 4.9 shows a project receiving asphalt undersealing.

Quality Assurance

The purpose of slab stabilization is to fill existing voids, not to raise the slab. Close inspection is required by the contractor and the inspector during the stabilization operation because lifting of the slabs can create additional voids that may lead to slab cracking. The success of the slab stabilization operation is highly dependent upon the skill of the contractor.

For cement grouts, the injection process should start with a low pumping rate and pressure. The cement grout should be pumped until one of the following conditions occurs (Darter et al. 1985, White et al. 2015):

- A maximum allowable pressure of 100 lbf/in^2 is obtained. (Note that, if necessary, a short surge of up to 200 lbf/in^2 can be allowed when starting to pump in order for the cement grout to penetrate the void structure.)
- The slab lift exceeds 0.05 in. or movement is detected.
- Injection material is observed flowing from adjacent holes, cracks, or joints.
- Injection material is being pumped unnecessarily under the shoulder, as indicated by lifting.
- More than about 1 minute has elapsed (any longer than this indicates the grout is flowing into a cavity).

During the slab stabilization process, the slab height should be monitored to ensure that raising of the slab does not occur. As described previously, if the slab is allowed to rise, additional voids may be created or excessive stresses may be induced in the slab. The uplift for any given slab corner should be monitored using a device that is capable of detecting 0.001 in. of uplift movement. Several methods of monitoring slab uplift are shown in Figure 4.10.

As indicated in these images, the reference point for monitoring movement must be far enough away from the injection area so that it will not be unduly affected by the flow of the stabilizing material.

The effectiveness of slab stabilization can be determined only by monitoring the subsequent performance of the pavement. An early indication of the effectiveness can be obtained, however, by measuring slab deflections before and after stabilization. For example, Figure 4.11 presents the change in deflections at slab corners after slab stabilization for a MoDOT project. The slab corner deflections decreased by more than 30% for 16 of the 22 testing locations, with 9 of those locations showing more than a 50% reduction in corner deflection.

John Donahue, MoDOT, used with permission

Figure 4.9. Injection of asphalt undersealing material

Wouter Gulden, retired GDOT (left) and John Donahue, MoDOT, used with permission (center and right)

Figure 4.10. Methods of monitoring slab uplift: mechanical method for cement grout injection (left), mechanical method for asphalt injection (center), and water level testing for polyurethane injection (right)

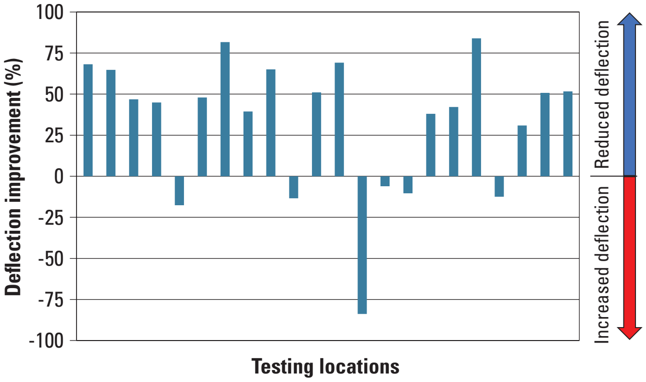


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Figure 4.11. Corner slab deflection before and after slab stabilization

If the follow-up deflection testing still indicates a loss of support, the slabs should be regrouted using new drilled holes. Guidelines from ACPA recommend that if voids are still present after three attempts to stabilize the slab, other methods of repair should be considered (e.g., FDR) (ACPA 2003).

Troubleshooting

Some of the more common problems that a contractor or inspector may encounter in the field during a slab stabilization project are shown in Table 4.1. Typical causes and recommended solutions associated with these problems are also provided.

3. Slab Jacking

Purpose and Project Selection

Slab jacking consists of the pressure insertion of a polyurethane material or cement grout mixture beneath a slab or series of slabs to slowly return the pavement to a smooth profile. This requires a series of insertion holes over the settled slabs and the application of the slab jacking material in alternating holes to minimize stress points. Ideal projects for slab jacking are pavements that exhibit localized areas of settlement but are generally free of cracking. Settlements can occur anywhere along a pavement profile, but they most usually are associated with fill areas, over culverts, and at bridge approaches.

Slab jacking is not recommended for repairing faulted joints along a project because these are more effectively addressed through DBR and/or diamond grinding. Louisiana attempted to reduce existing faulting by sawcutting through the joints and injecting polyurethane foam under the slabs. This method was successful at reducing faulting to less than 0.25 in. but with an accompanying loss of load transfer and increase in joint deflections (Gaspard and Zhang 2015).

Because loss of support and slab settlement may be caused by a number of different factors (including excessive moisture, poor load transfer at joints, and poor consolidation), slab jacking performed by itself may not be sufficient to eliminate the problems. If the underlying mechanisms that led to the development of the support or settlement issues are not addressed as part of the treatment process, the same distress conditions will once again resurface (ACPA 1994, Hoerner et al. 2001, Onyango et al. 2018). Thus, candidate pavements should be thoroughly evaluated and the need for additional preservation treatments (e.g., DBR, diamond grinding, and joint sealing) carefully considered.

Limitations and Effectiveness

The effectiveness of slab jacking is highly dependent upon closely monitoring the amount of lift being performed at any one location. It is very important that the slab not be lifted more than 0.25 in. at a time at any one location to prevent the development of excessive stresses in the slab. Where careful monitoring has been conducted, slab jacking has been effective at leveling out isolated depressed areas (such as over culverts) and at bridge approach slabs.

Table 4.1. Potential slab-stabilization-related problems and associated solutions

Problem	Typical cause(s)	Typical solution(s)
There is a combination of (1) no evidence of grout in any adjacent hole, joint, or crack after 1 minute, and (2) no registered slab movement on the uplift gauge.	Grout is flowing into a large washout cavity.	Stop the injection process. The cavity will have to be corrected by another repair procedure.
High initial pumping pressure does not drop after 2 to 3 seconds.	Spalled material at the bottom of the hole may be blocking entrance to the void.	Material blockages may sometimes be cleared by pumping a small quantity of water or air into the hole to create a passage that will allow grout to flow into the void. If this activity does not solve the problem, it is possible that the hole was drilled outside of the boundaries of the void.
Testing after one properly performed grouting still indicates a loss of support.	The void was not adequately filled. The first assumption should be that the selected hole pattern did not provide complete access to the void.	Regrout the void using different holes from those that were initially used.
Testing after two properly performed groutings (i.e., after regrouting) still indicates a loss of support.	The void is still not adequately filled. After regrouting has been attempted, the assumed typical causes are the following: • The second selected hole pattern still did not provide complete access to the void. • The void may be deeper in the pavement layer system.	One of the following may apply: • If it is suspected that the selected hole pattern did not adequately locate the boundaries of the void, the contractor may choose to drill holes at additional locations. • If the contractor is confident that the boundaries of the void have been established, the injection holes may have to be extended into the subgrade.
Uplift gauge exceeds the maximum specified slab lift (typically 0.05 in.).	Overgrouting occurred.	Overgrouting a void can cause immediate cracking or, at a minimum, increase the potential for long-term slab cracking. The solution to this problem is determined by the governing agency specification. If slab damage is immediately observed, the contractor will most likely be responsible for replacing the slab at no cost to the agency.
Grout extrudes into a working transverse joint or crack.	This typically indicates that the void is filled or that the hole has been drilled too closely to a joint or crack.	The presence of incompressible material in a joint or crack can increase the probability of spalling or blowups. For a joint, the solution is to restore the joint reservoir and joint sealant. For a crack, the solution is to rout or saw and seal the crack.

Onyango et al. (2018) surveyed state departments of transportation (DOTs) and noted that while slab replacement was the most common method of slab leveling, almost as many respondents were using a slab jacking technique and most of those were using a polyurethane compound. Asphalt injection was listed as the least common method employed by state DOT respondents. The respondents rated polyurethane injection as the most cost-effective method of slab leveling because of its lower material cost and time of construction. The study concluded that using polyurethane injection did not statistically improve or degrade the roadway profile. The study also recommended the following:

- Prior to injection of the material, a detailed ground investigation of the damaged pavement section must be carried out to establish the causes and thus whether polyurethane foam injection will be the appropriate remedial measure.
- Polyurethane foams should be injected under structurally sound slabs resting on a granular subbase, and sophisticated leveling equipment (e.g., a laser level) should be used to avoid overcorrection of the treated slabs. For stabilized bases, the polyurethane foam should be injected under the stabilized layers.
- For structurally damaged slabs, slab replacement is more cost-effective and a very appropriate option.
- If polyurethane injection is conducted on Interstates where heavy trucks are expected, the density of the material is recommended to be very high.

Materials and Design Considerations

Determining the Repair Area

Slab jacking should be considered for any condition that is the result of nonuniform support. These conditions often result in localized dips or depressions that adversely affect the rideability of the pavement. Common areas include slabs over culverts or bridge approach slabs, both typically the result of poor and inadequate compaction of the underlying fill. Localized settlements may also occur over embankment areas. Subsurface testing (such as using a dynamic cone penetrometer) may be performed to identify soil and base properties and the potential extent of the settlement area. Figure 4.12 shows before and after photos of the raising of a settled slab.

IGGA, used with permission

Figure 4.12. Settled slab before (top) and after (bottom) slab jacking

Selecting an Appropriate Injection Hole Pattern

The appropriate location of holes for a given site can only be determined by experienced personnel. This is important because the slab must be lifted in such a way so as not to create stresses that could cause cracking. Holes should be spaced not less than 12 in. but not more than 18 in. from a transverse joint or slab edge to help minimize stress points (MnDOT 2006). In addition, holes should be spaced 6 ft or less center to center, so that less than 25 to 30 ft^2 of the slab is raised by grouting a single hole (MnDOT 2006). Figure 4.13 illustrates an example of pattern in which the holes are placed in a triangular fashion to correct a settlement over two lanes.

The holes are spaced, as nearly as possible, equidistant from one another, because the grout tends to flow in a circular pattern from each hole. Holes in adjacent slabs should follow the same arrangement. MoDOT uses the same triangular pattern with a 6 ft spacing, but the holes are kept 3 ft from centerline joints and the edge of the pavement (MoDOT 2018). The Alabama Department of Transportation (ALDOT) uses a rectangular pattern as shown in Figure 4.14 (ALDOT 2017). Onyango et al. (2018) recommend five to nine holes per slab to be lifted.

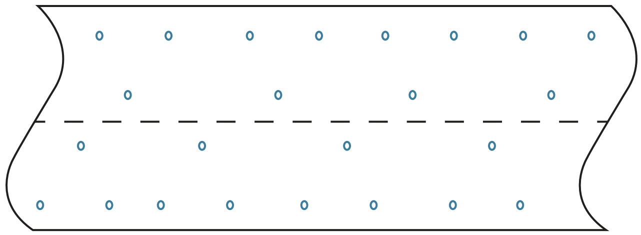

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Figure 4.13. Pattern of grout pumping holes used to correct a settlement

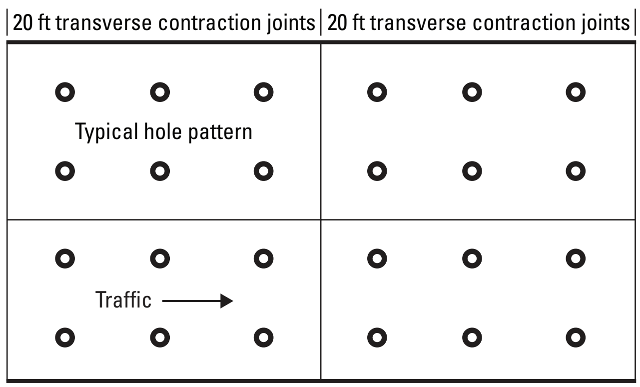

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Figure 4.14. ALDOT grout hole pattern

Selecting an Appropriate Material

Much of the material discussion in the slab stabilization section is also applicable to slab jacking. Onyango et al. (2018) found that in a survey of state DOTs, polyurethane compounds were most commonly used, followed by cement grouts and then asphalt. Cement grouts used for slab jacking are typically slightly stiffer than those used for slab stabilization procedures, generally having flow cone times of 16 to 30 seconds. Pozzolan- and fly-ash-based grouts generally consist of three to seven parts fine aggregate (or a mixture of aggregate and pozzolans or fly ash) to one part cement, with enough water to produce the desired consistency (MnDOT 2006).

Construction Considerations

The slab jacking process for grout or polyurethane injection is similar to that of slab stabilization; however, procedures are required for monitoring that the final slab profile meets the desired grade. The taut string line method (illustrated in Figure 4.15) is the traditional way not only to control the pumping sequence but also to achieve the proper grade.

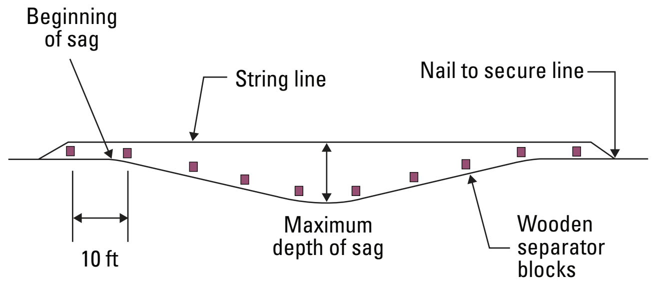

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Figure 4.15. String line method of slab jacking

MnDOT (2006) notes that, for correcting isolated faulted slabs, a straightedge may instead be used. For dips more than 50 ft long, a survey rod and level were recommended to check the profile well beyond the dip. Laser technology may also be used to simulate a string line.

In the string line method, small wooden blocks, 0.75 in. high, are set on the pavement surface along the outer and inner edges and a string line is secured at least 10 ft from each end of the depression. As material pumping proceeds, the exact amount of rise at each point within the sag can be observed, allowing the pumping at specific holes to be carefully controlled. This method can consistently achieve profiles within tolerances of 0.25 to 0.38 in.

Although the string line method has worked well, laser technology is now being used by many contractors for monitoring pavement elevations because of its increased speed and accuracy. With a laser level, profiles are established (typical maximum spacing of 5 ft) in the wheel paths prior to pumping. An ideal profile can then be established and these same profile points monitored during the pumping operation.

State and local highway agencies often have their own unique techniques for the raising of a slab, but a typical procedure is described below:

1. After all preliminary work has been completed (i.e., holes drilled and relief opening also cut if needed), the pavement is ready to be raised. The slab must be raised only a very small amount at each hole at a time. A good rule is not to raise a slab more than 0.25 in. while pumping in any one hole. No portion of the slab should be more than 0.25 in. higher than any other part of the slab (or an adjacent slab) at any time. The entire working slab and all those adjacent to it must be kept in the same plane, within 0.25 in., throughout the entire operation to avoid cracking.

2. Pumping should be done over the entire section so that no great strain is developed at any one place. If, for example, pumping is started at either end of a dip, the tension on the top surface will be increased and the slab will undoubtedly crack. If pumping is started at the middle where the tension is on the lower surface, however, lifting will tend to reduce the tension and the slab can be raised an appreciable amount without any damage. As the section is brought back to its original profile, the pumping is extended farther and farther in either direction until the entire dip is at the desired elevation.
3. Care must be taken not to prematurely flatten the middle out completely. This will cause a sharp bend and cracking. The middle section naturally must be raised faster than the ends of the dip, but lifting should be conducted in such a manner as to avoid sharp bends.

An example of a suggested slab jacking pumping sequence that provides a general guideline for obtaining satisfactory results is presented in the following text. It must be remembered that this sequence should be modified to meet the specific needs of a given project.

1. Figure 4.16 shows a plan view of a dip. Pumping should begin in the middle of the dip, with the hole at the top of the figure labelled “1” in the pumping sequence. Pumping should be stopped when movement is noted in the slab and before it reaches 0.25 in. The hole where the material is initially pumped will take more material than those at either side because of the shape of the dip. Pumping should always begin at the outside row of holes, followed by the inside row of holes.
2. Pumping at hole 2 relieves the strain that may have resulted from lifting the slab at hole 1. The third hole to be grouted will be back at the first hole grouted, and then material is pumped at holes 4 through 8 as shown in Figure 4.16. This results in material being pumped four times into the initial hole and twice at the holes on either side. If the same amount of material were pumped each time and traveled the same distance away from its respective hole, the slab would be raised twice as much at the middle hole as at the other two. Pumping should never be performed back and forth across the entirety of a slab from the beginning to the end of a consecutive series of holes; instead, to avoid cracking, work always proceeds starting from the middle of the slab and then going from one side back to the middle and then to the other side and again back to the middle, systematically adding one adjacent new hole at a time on each side. A concrete slab can withstand more twisting than transverse bending.
3. The line of holes in the middle of the pavement is pumped after the outer row, using the same sequence described above. If both sides of the slab are at about the same elevation, the next pumping is at the outer side of the adjoining slab (bottom of Figure 4.16), following the same sequence, with additional pumping conducted farther from the center of the dip (i.e., grout applications in holes 9 through 13). Pumping is continued in this order until the slab has been brought to the desired elevation.
4. The last hole at each end of the dip, noted with triangles in Figure 4.16, should not be used until the slab is at the desired grade. A very thin grout, similar to that used for slab stabilization, may be used to ensure complete filling of the thin wedge-shaped opening that was created at this part of the dip.

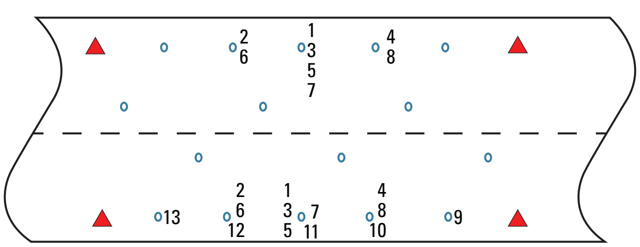

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Figure 4.16. Order of grout pumping used to correct a settlement

For cement grout materials, the injection holes should be plugged with tapered wooden plugs immediately after pumping in the hole has been completed to retain the pressure of the grout and to prevent any return flow of the mixture (MnDOT 2006). When the slab jacking operation is complete, the temporary plugs are removed, any excess material should be removed flush with the pavement surface, and the hole should be filled with an approved patching material. Holes for polyurethane materials may also be filled with an approved patching material, but they are often left unpatched because they are so small (and are now filled with the polyurethane material).

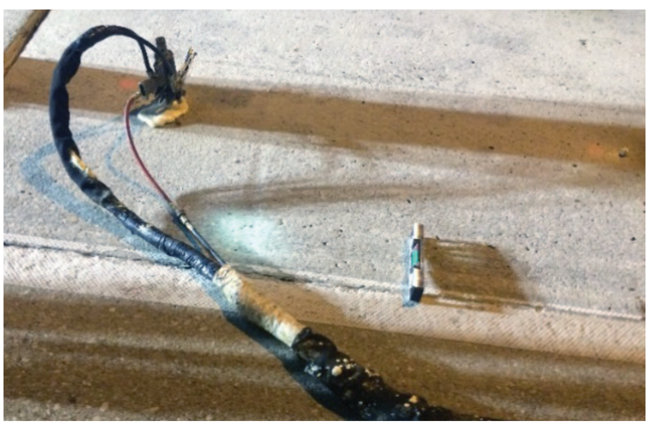
Figures 4.17 through 4.19 show the sequence of drilling to raise slabs as performed in a Tennessee installation (Onyango et al. 2018).

Onyango et al. 2018, University of Tennessee at Chattanooga, used with permission

Figure 4.17. Drilling holes for polyurethane foam installation on I-24 West in Tennessee

Onyango et al. 2018, University of Tennessee at Chattanooga, used with permission

Figure 4.18. Injection holes in concrete slab



Onyango et al. 2018, University of Tennessee at Chattanooga, used with permission

Figure 4.19. Installation of polyurethane foam on I-24 West in Tennessee

Quality Assurance

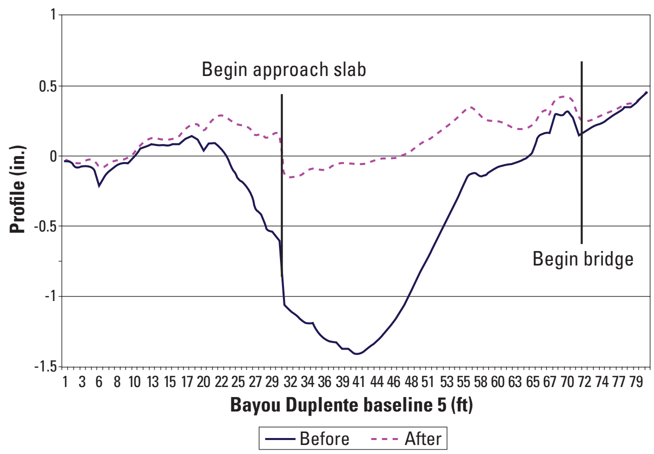
The primary concern of slab jacking is excessively raising the slab, which can induce stresses in the slab that can lead to cracking. Therefore, it is critical that the slab be raised no more than 0.25 in. at a time when pumping at each hole. In addition, no portion of the slab should be more than 0.25 in. higher than any other part of the slab (or an adjacent slab) at any time during the lifting process to avoid cracking. These elevations can be monitored using a string line or other leveling system.

It is generally recommended that pumping start at the middle of the depressed slab. This will help to reduce the tension that has developed at the top of the slab. As the section is brought back to its original profile, the pumping is extended farther and farther in either direction.

The effectiveness of the slab jacking process can be assessed both visually and from an examination of the pavement profile. Figure 4.20 shows the profile of a bridge approach slab, both before and after the slab jacking operation.

Onyango et al. (2018) found mixed results in the improvement of the mean roughness index (MRI) on five Tennessee Interstate locations as shown in Figure 4.21.

Quality assurance staff also need to be aware of blowouts that may occur, especially on elevated embankments and bridge approaches. If blowouts do occur, the contractor can attempt to plug that area or wait for the slab jacking material to set up before resuming slab jacking.



Recreated from Gaspard and Morvant 2004, Louisiana Transportation Research Center

Figure 4.20. Bridge approach slab profile before and after slab jacking

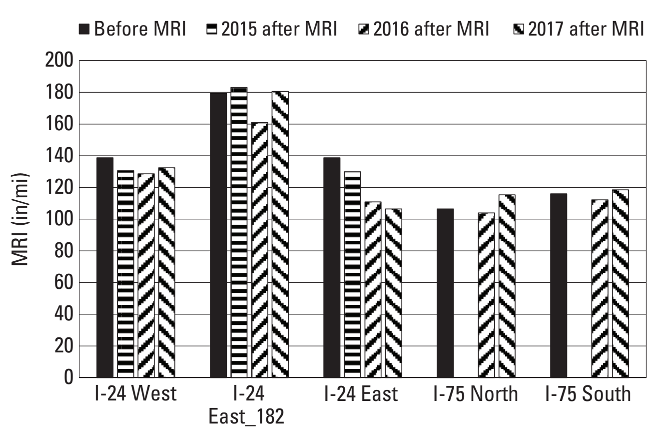


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Figure 4.21. Tennessee Interstate MRI values before and after slab jacking

4. Summary

Slab Stabilization

Loss of support from beneath concrete pavement slabs is a major factor contributing to pavement deterioration. Slab stabilization is defined as the insertion of a material beneath the slab or subbase to fill voids, thereby reducing deflections and associated distresses. Because loss of support can be caused by multiple factors, slab stabilization is often done in conjunction with other rehabilitation activities (e.g., patching, diamond grinding, and DBR) in order to address the causes of the voids (ACPA 1994).

Commonly used slab stabilization materials include cement-based mixtures (limestone dust–cement grouts and pozzolanic cement grouts) and polyurethane. Since slab stabilization is not intended to lift the slab, it is very important to monitor slab lift during the material injection process to avoid overgrouting the slab, which could result in slab damage. An experienced contractor and proper inspection are essential to a successful slab stabilization project.

Slab Jacking

In areas of localized settlements or depressions, slab jacking can be used to lift the slab and reestablish a smooth profile. This is accomplished through the pressure injection of a material beneath the slab or subbase and careful monitoring of the lift at different insertion holes until the desired profile is obtained. Typically, polyurethane or slightly stiffer cement grouts than those used for slab stabilization are required for slab jacking. During slab jacking, a laser level or the string line method can be used effectively to monitor slab lifting, which is essential to minimize the development of slab stresses.

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Chapter 5
Partial-Depth Repairs

1. Introduction

Partial-depth repairs are the removal of small, shallow (less than half the slab thickness) areas of deteriorated concrete that are then replaced with a suitable repair material. These repairs restore the overall integrity of the pavement and improve its ride quality, thereby extending its service life. Partial-depth repairs of spalled joint areas also restore a well-defined uniform joint reservoir prior to joint resealing.

Partial-depth repairs are an alternative to full-depth repairs in areas where slab deterioration is located primarily in the upper half of the slab and where the existing load transfer devices (if present) are still functional. The success of PDRs is heavily dependent on the quality of the installation, requiring strict adherence to specified removal, preparation, placement, and curing procedures. Historically, PDRs have exhibited variable performance, but when applied to the appropriate distresses and with durable materials and proper construction practices, service lives of 10 to 20 years or more can be achieved (Darter 2017). Moreover, since they are smaller and target very specific distresses, PDRs can be more cost-effective than FDRs. This chapter presents effective techniques that can be used in the design and construction of well-performing concrete PDRs.

2. Purpose and Project Selection

Partial-depth repairs are appropriate for concrete pavement distresses that are confined within the upper half of the concrete slab. Distresses or conditions that have been successfully addressed with PDRs include the following:

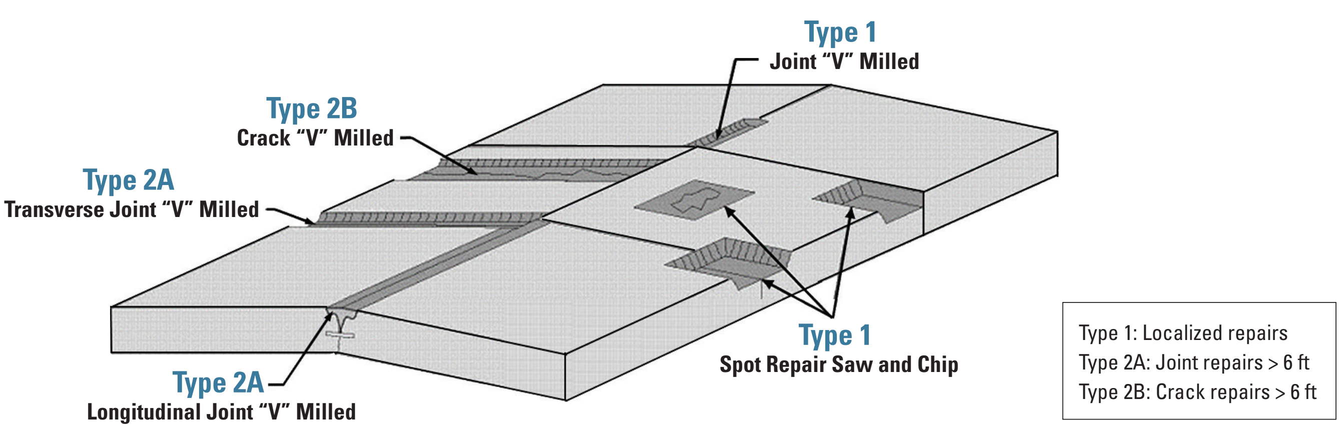
- Spalling caused by the intrusion of incompressible materials into the joints or cracks
- Spalling caused by poor consolidation, inadequate curing, or improper finishing practices
- Spalling or delamination caused by weak concrete, clay balls, or mesh reinforcing steel located too close to the surface
- Spalling caused by an inadequate air void system
- Other localized areas of deterioration, delamination, or scaling that are limited to the upper portion of the slab and are of sufficient size and depth to warrant repair
- Filling of rumble strips in a shoulder that is being converted to a driving lane

Conversely, there are a number of concrete pavement distresses that are not good candidates for PDRs, as in these cases there are often other mechanisms at work that will detract from the performance of the PDR. Examples of these types of distresses include the following:

- Spalling caused by dowel bar misalignment or lockup
- Spalling that extends more than half of the slab thickness
- Spalling caused by MRD, such as D-cracking or reactive aggregate. PDR may be used as a stopgap measure in these cases, but the deterioration will continue to develop and create performance issues. Coring will be required to determine the depth and extent of deterioration on the bottom of the slab. Weiss et al. (2016) provide guidance on the applicability of PDRs for concrete pavements exhibiting premature joint deterioration.

3. Types of Partial-Depth Repairs

Frentress and Harrington (2012) define the general types of PDRs for cracks, joints, and spalls as shown in Figure 5.1.



Adapted from Frentress and Harrington 2012, CP Tech Center

Figure 5.1. Types of PDRs

Details on each of these repair types are provided in the following sections.

Type 1: Spot Repairs of Cracks, Joints, and Spalls

Type 1 repairs address localized areas of deterioration and are not recommended for long, continuous repairs (say, longer than 6 ft). Common distresses addressed by Type 1 repairs include the following (Frentress and Harrington 2012):

- Joint or crack spalling
- Midpanel surface spalling, scaling, or deterioration
- Deterioration of joint reservoir

On joints or cracks, some agencies allow longer longitudinal runs that combine a series of smaller repairs into a single, continuous repair. This can help improve the overall efficiency of construction operations and can be quickly and effectively performed using milling procedures.

The deteriorated concrete can be removed by either sawing around the perimeter of the repair and breaking out with light jackhammers or using small milling machines (these methods are described in more detail in the construction section). For repairs performed using conventional cementitious concrete mixtures, the repair area should be angled out slightly (approximately 30 to 60 degrees) at the edges to help facilitate bonding, but proprietary materials should follow the manufacturer's recommendations for repair areas, dimensions, and geometries. Typical details for Type 1 repairs using these two removal methods are shown in Figure 5.2.

Type 2: Joint and Crack Repairs

These types of repairs are performed on longitudinal or transverse joints (Type 2A) or on cracks (Type 2B) and are more than 6 ft long (Frentress and Harrington 2012). Figure 5.3 shows candidate distresses for Type 2 repairs.

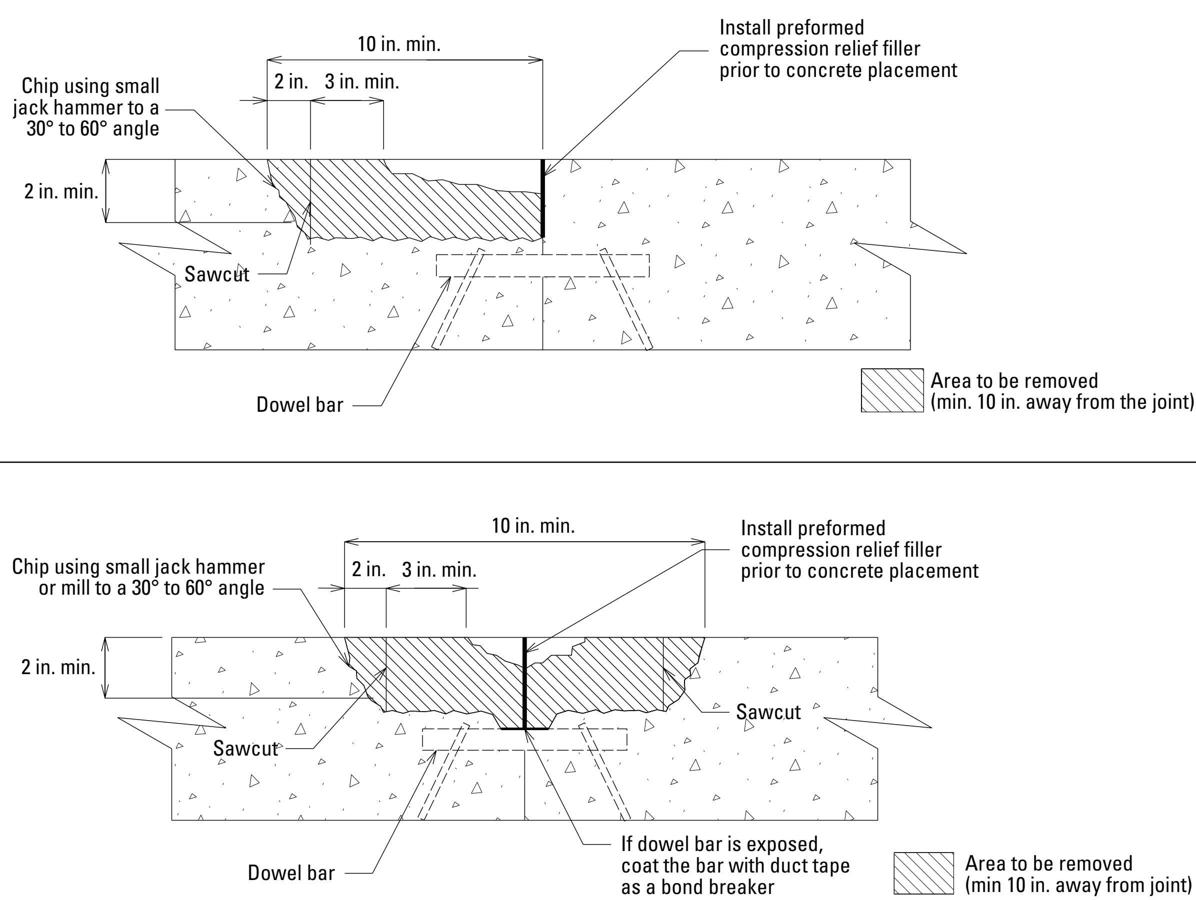


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Figure 5.2. Typical details for Type 1 PDRs: saw and chip (top) and saw and chip or milled (bottom)

Kevin McMullen, Wisconsin Concrete Pavement Association, from Frentress and Harrington 2012, CP Tech Center

Figure 5.3. Candidate distresses for Type 2 PDRs

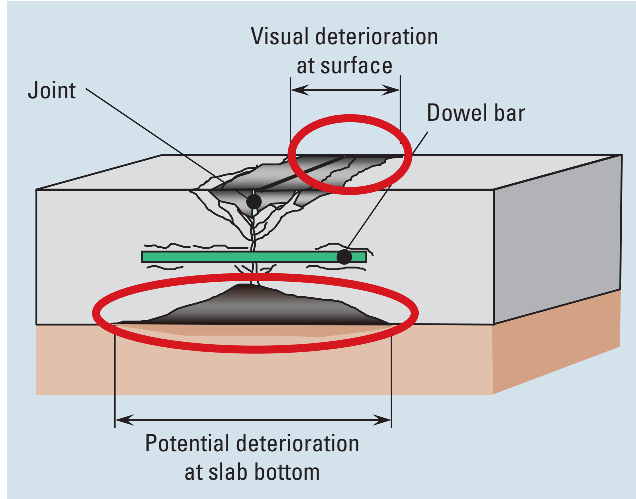

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Figure 5.5. Potential extent of deterioration beneath a joint

Compression relief for repairs at joints or cracks can be performed either by sawing at the defined joint location or by installing a preformed compression material. When performing these repairs at joints, the sawing to reestablish the joint and provide compression relief must be administered for the full thickness of the repair plus an additional 0.25 to 1 in. Additionally, the width of the sawcut must be sufficient to help prevent compression failures. Typical details for Type 2 repairs are shown in Figure 5.4.

4. Limitations and Effectiveness

Partial-depth repairs are an effective treatment for joint or crack spalling that is isolated within the upper portion of the slab or for surface scaling, spalling, or delamination. Partial-depth repairs are not recommended as a long-term solution for pavements with MRD, joint spalling caused by dowel bar lockup, or deep spalling that goes beyond half of the slab thickness. (These recommendations are based on minimizing the potential risk of failure when PDR has been used as a preservation treatment.)

Some concrete pavements may be afflicted with deterioration on the bottom of the slab that is not visible on the surface (see Figure 5.5); this deterioration can be the result of a number of factors, such as the presence of MRD, the use of nondurable concrete mixtures, saturated subsurface conditions, or the use of aggressive deicing materials.

To determine the suitability of PDR, therefore, a limited coring program should be conducted to determine the severity and extent of any subsurface deterioration, as well as the depth of deterioration at the surface. Significant levels of subsurface deterioration and surface deterioration at or below the dowel bar suggest that FDR may be the more appropriate treatment.

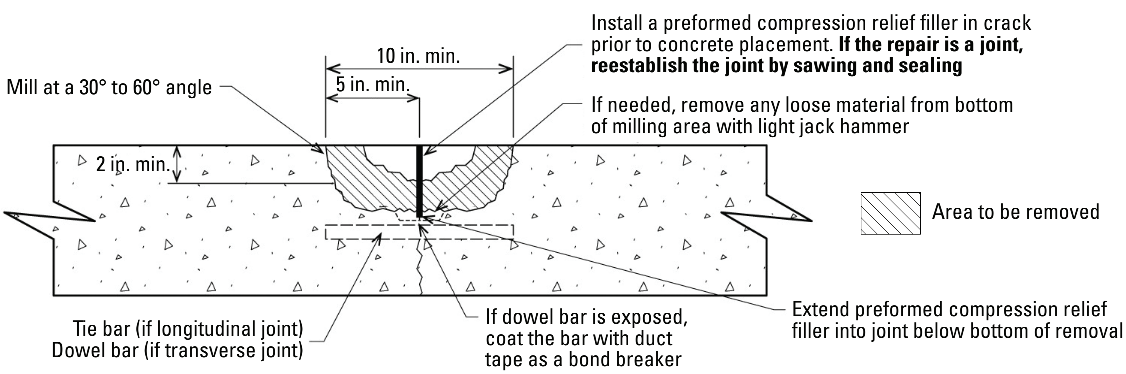

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Figure 5.4. Typical details for a Type 2 joint PDR

For effective performance, PDRs must meet the following requirements (Darter 2017, Frentress and Harrington 2012):

- Sound concrete must be present both surrounding and beneath the PDR. If sound concrete is not present, FDR should be considered (see Chapter 6).
- Deteriorated material should be completely removed from the existing concrete pavement through the use of appropriate removal techniques (milling and removal or sawing/chipping and removal).
- The repair area must be thoroughly cleaned (through media blasting or other appropriate methods) to promote bonding of the repair material; some agencies also specify the application of a bonding agent to the repair area immediately prior to the placement of the repair material. For cement-based repair materials, saturated surface-dry conditions may be adequate for effective bonding as long as the substrate concrete is properly cleaned and textured, whereas proprietary repair products should follow the manufacturer's installation recommendations.
- Existing joints or cracks beneath or along the sides/edges of the PDR must be sealed.
- When appropriate, any joint/crack through or at the edge of the PDR must be formed properly.
- The PDR should be adequately cured to ensure a durable repair.

The performance of PDRs depends on the general condition of the existing pavement, the type of materials used, and the proper construction and placement techniques. In general, when sound construction practices and a durable material are used, PDRs can last 10 to 20 years (Darter 2017), but when poor materials or workmanship are encountered, PDRs may fail in as little as 2 to 3 years (ACPA 2006).

5. Design and Materials Considerations

The first part of this section describes the steps and techniques used to determine and mark individual repair boundaries. The second part of the section focuses on repair materials, including specific discussions of different available materials commonly used in PDRs, considerations when selecting the material for a given project, and the use of bonding agents.

Sizing Repairs

When a project is first triggered as a candidate for PDRs, the first step in the process is to conduct a field survey of the project to confirm overall conditions and to estimate repair quantities; this also could be done working off video surveys of the pavement. The quantities gained from the initial evaluation serve as a starting point, and many agencies use a simple multiplier (often 25% to 30% or more) to estimate quantities for bidding purposes.

However, as discussed previously, coring is typically performed to determine the depth and extent of surface and subsurface deterioration in order to help confirm the feasibility of PDRs. Some agencies also plan for a certain portion of the PDRs to be converted to FDRs due to the expectation that some unsound material will be encountered during the actual construction process.

It is important that all weak and deteriorated concrete is located and removed if the repair operation is to be effective. It is generally recommended that the repair boundaries extend 3 in. beyond the detected delaminated or spalled area to ensure removal of all unsound concrete, but some judgment is still required based on the severity of the deteriorated conditions. A minimum repair length (along the joint or crack) of 10 in. and a minimum repair width (away from the joint or crack) of 4 in. are recommended for cementitious materials (Wilson et al. 1999), but proprietary materials should follow the manufacturer's recommendations for repair dimensions. For cement-based materials, the repair should be at least 2 in. deep in order to provide sufficient mass to bond to the underlying substrate; other products (e.g., some polymers and epoxies) allow a thinner application (Darter 2017).

The repair area should also be kept nominally square or rectangular in shape and in line with the existing joint pattern to avoid irregular shapes that could cause cracks to develop in the repair material (ACPA 2006). If separate repair areas are closer than 24 in. apart, they can be combined to help reduce costs and eliminate numerous small repairs (ACPA 2006).

Repair Material Types

A variety of materials may be used in PDRs, from conventional cementitious materials to proprietary, early-strength cementitious and polymeric products; in addition, various bituminous mixtures (both conventional and proprietary) are also available.

It should be noted that most of the conventional cold-mixed bituminous materials are intended for short-term, emergency-type repairs, but there are a number of proprietary, modified bituminous mixtures that offer longer performance lives. In addition, a number of new repair materials are being introduced, and several recent studies have been performed evaluating the laboratory and field performance of those materials (Burnham et al. 2016, Ram et al. 2019, Falls 2019, Ramsey et al. 2020). This section describes some of the characteristics and properties of common materials used for PDRs.

Conventional Concrete Mixtures

High-quality concrete is generally accepted as the most appropriate material for PDRs. Typical mixtures combine concrete with coarse aggregate not larger than half the minimum repair thickness, with a 0.375 in. maximum size often used. The material should be a low-slump, low-shrinkage mixture having a w/cm ratio not exceeding 0.44. Adequate air entrainment is important for repairs performed in cold-weather states.

For repairs that must be opened to traffic quickly, a mixture featuring either a Type I cement with a set-accelerating admixture or a Type III cement have been used successfully. Type I concrete, with or without admixtures, is more widely used than most other materials because of its relatively low cost, availability, and ease of use. Rich mixtures (say 750 lb/yd^3 or more) gain strength rapidly in warm weather, although their rate of strength gain may be too slow to permit quick opening to traffic in cool weather. In cool weather conditions, insulating layers can be used during installation to help retain the heat of hydration and reduce curing time. Concrete mixtures produced using Type III cement should be used with caution as they can be more difficult to work and may develop shrinkage cracks.

Many state and local highway agencies have developed standard mixtures for use in their PDRs. As an example, the Minnesota Department of Transportation (MnDOT) has had good success with a cementitious mixture that provides an 18-hour opening strength of 3,000 lbf/in^2; the composition of this mixture is as follows (Frentress and Harrington 2012):

- 850 lb Type I cement
- 295 lb water
- 1,328 lb coarse aggregate
- 1,328 lb sand
- Target 0.35 w/cm ratio
- Type E water-reducing and accelerating admixture
- 6.5% air

As dictated by opening requirements, there are also several proprietary concrete-based repair materials available to achieve high-early strength for critical PDRs; most agencies have a list of approved products for use in these applications.

Modified Hydraulic Cements

A number of modified hydraulic cements are available for use in PDRs, including gypsum-based cements, calcium aluminate cements, and other hydraulic-cement-based mixtures. Commentary on three of these products is provided below:

- Gypsum-based cements contain calcium sulfates that provide setting times of 20 to 40 minutes, enabling opening to traffic in as little as 1 hour (depending on conditions). Gypsum-based cements are recommended for use in temperatures above freezing and are not affected by deicing chemicals, but they generally require dry conditions during placement. Furthermore, gypsum-based cements are not recommended for use in reinforced pavements because the presence of free sulfates in the typical gypsum mixture may promote steel corrosion (Good Mojab et al. 1993).
- Calcium aluminate cements gain strength rapidly, have good bonding properties, demonstrate good resistance to freeze-thaw cycles and deicing chemicals, and exhibit low shrinkage. However, concrete made from calcium aluminate cements undergoes a phenomenon called "conversion," during which a portion of the concrete strength is lost. To address this, the proposed concrete mixture design should be evaluated with an accelerated conversion test to ensure the converted strength is in excess of the specified strength required for the application, as described by Ideker et al. (2013).
- Calcium sulfoaluminate (CSA) cements are a modified derivative of portland cement clinker and are being used by a number of state and local highway agencies. CSA cements offer rapid strength gain, good durability, and high sulfate resistance while also exhibiting very low shrinkage (as low as 200 macrostrains at 28 days) without the use of shrinkage-reducing admixtures (Ross 2019). CSA cements also do not undergo the conversion process associated with calcium aluminate cements and have exhibited good performance in a number of repair applications (Ramseyer and Perez 2009, Guan et al. 2017, Ross 2019).

Polymer-Based and Resinous Concretes

Polymer-based concretes are formed by combining polymer resin (molecules of a single family or several similar families linked into molecular chains), aggregate, and an initiator. Aggregate is added to the resin to make the polymer concrete more thermally compatible with the concrete (large differences in the coefficient of thermal expansion [CTE] can cause debonding), to provide a wearing surface, and to make the mixture more economical. The main advantage of polymers is that they set much more quickly than most of the cementitious materials, but they are also expensive and some can be sensitive to temperature and moisture conditions. Polymers used for pavement repairs include urethane resins and epoxies, among others.

Some polymer-based repair materials are very sensitive to temperature changes, and consequently their elastic modulus can vary significantly depending on ambient temperature conditions. However, these variations are not expected to adversely impact the bond between the repair material and the substrate concrete provided that all unsound concrete is completely removed before the placement of the repair material (Ram et al. 2019).

Descriptions of some of the polymer-based repair materials are provided below:

- Polyester concretes are a mixture of polyester-styrene resin binder and aggregate that bonds very well to the underlying substrate and can be opened to traffic in just a few hours. Although polyester concretes are more expensive than traditional cementitious materials or commercially available rapid-setting materials, they exhibit better performance over a wider range of conditions. These materials can also be used over a wide range of surface temperature conditions between 40°F and 130°F (Caltrans 2015).
- Epoxy polymer concretes are also two-component systems consisting of a liquid epoxy resin that is mixed with a curing agent. They are impermeable and generally have excellent adhesive properties, but they also exhibit a wide range of setting times, application temperatures, strengths, and bonding conditions. Wherever they are used, the epoxy concrete mixture (which typically has a higher CTE) must be compatible with the concrete in the pavement. Differences in the CTE values between the repair material and the existing concrete may lead to failures of the repair due to stresses resulting from the thermal expansion of the epoxy material. In addition, deep repairs must frequently be placed in multiple lifts to control heat development.
- Polyurethane-based repair materials generally consist of a two-part polyurethane resin mixed with aggregate. Polyurethanes are generally very quick setting and are very flexible. They also often exhibit, however, a high CTE and significant initial shrinkage, and many types are intolerant of moisture. These types of materials have been used for several years.
- Some of the polymer-based repair materials are particularly flexible, which allows them to be placed across joints and cracks without having to reestablish the joint. These more flexible repair materials generally do not crack. Minimizing the size of repairs (particularly when the material is used to repair severely distressed areas) can help ensure better performance when using these materials (Ram et al. 2019). However, as with all proprietary products, the manufacturer's recommendations should be followed for the depth of placement and the repair size/configuration.

It should be noted that the placement of thin overlays (with either asphalt or concrete) over areas repaired using polymer-based repair materials may cause delamination issues. For areas that are likely to be overlaid in the near future, polymer-based repair materials are not recommended for PDRs. Any existing PDRs using polymer-based repair materials may need to be removed and replaced with conventional concrete or bituminous repair materials prior to overlay placement (Ram et al. 2019).

Magnesium Phosphate Concrete

Magnesium phosphate concretes set very rapidly and produce a high-early-strength, impermeable material that will bond to clean dry surfaces. These materials may be packaged as one- or two-component systems, with the one-component system consisting of magnesium and phosphate mixed together in powdered form to which a specified amount of water is added, whereas the two-component system consists of powdered magnesium and aggregate that is mixed with a liquid solution of phosphate.

Magnesium phosphate materials are extremely sensitive to moisture, either on the substrate or in the mixture (even very small amounts of excess water cause severe strength reduction). Furthermore, magnesium phosphate concretes are very sensitive to aggregate type (e.g., limestones are not acceptable). In hot weather (greater than 90°F), many magnesium phosphate concrete mixtures experience short setting times (less than 15 minutes) and can therefore be difficult to work with; as a result, some manufacturers produce special magnesium phosphate concrete formulations for hot weather conditions.

Conventional Bituminous Materials

Conventional bituminous materials are often considered temporary repair materials on concrete pavements that are used until more rigorous patching can be performed. They have the advantage of being relatively low in cost, widely available, and easy to handle and place, and they generally need little, if any, cure time. In some cases, they have even demonstrated longer-term performance (on the order of 3 to 5 years). In addition, they may be suitable in some cases for patching concrete pavements prior to the placement of an overlay, particularly when the existing concrete pavement is too deteriorated to permit FDRs. It is again emphasized, however, that the use of conventional bituminous materials should largely be considered a stopgap, temporary repair measure.

Proprietary Modified Bituminous Materials

Several proprietary modified bituminous materials are available for use in PDR applications on street and highway pavements. Although these products are more expensive, they have demonstrated much better performance than conventional bituminous materials. In addition, the flexibility of some hot-applied, polymer-modified asphalt paving materials is such that they can be placed across transverse and longitudinal joints without the need for maintaining or reforming the joint, which helps reduce installation time.

Some proprietary polymer-modified bituminous repair materials, however, exhibit very low elastic-modulus values, and these materials can be very sensitive to changes in temperature and loading rate. Importantly, if exposed to prolonged high temperatures (>95°F), especially in hot weather areas during summer months, these materials may be prone to rutting/permanent deformation issues that could create safety concerns (Ram et al. 2019).

Selecting Repair Materials

The selection of a repair material is based on a number of factors often specific to a particular project, but it should be recognized that repair materials are but one aspect of the PDR "system." Thus, the performance of the PDR depends not only upon the repair material itself but also upon the suitability of the repair for the project and the proper installation of the repair. Transportation agencies often maintain a list of approved materials and repair approaches to meet the needs of their specific repair applications.

Among the factors to be considered in the selection of a PDR material for a specific project are the following:

- Available curing time
- Placement conditions (ambient temperatures and moisture levels)
- Material properties (particularly shrinkage, CTE, and bond strength)
- Material and placement costs
- Material handling and workability
- Compatibilities between the repair material and existing pavement
- Size and depth of the repair
- Performance capabilities and performance requirements of the project
- Project size

The available curing time (i.e., how quickly the repair must be opened to traffic) is often the primary factor driving the selection of the repair material. Table 5.1 presents the opening requirements used by several highway agencies (Frentress and Harrington 2012). Since PDRs are confined and supported by the existing concrete, the minimum compressive strength requirements to support traffic loading without experiencing any deterioration can be lower than those for FDRs.

Table 5.1. Examples of opening strength requirements for PDRs

State	Compressive strength (lbf/in^2)
Colorado	2,500
Georgia	2,500
Kansas	1,800
Michigan	1,800
Minnesota	3,000
Missouri	1,600
Nebraska	3,624
New York	1,527

Sources: After Frentress and Harrington 2012 and Darter 2017

In certain environmental regions, freeze-thaw durability is an important property of the repair material. A study of the properties of repair materials found that the freeze-thaw durability of many materials is unacceptable, especially under severe exposure conditions (ACI 2006). Moreover, materials with rapid strength gain characteristics may sometimes be particularly susceptible to durability problems. The composition of modern cements is such that they gain higher strengths earlier, but they have a lower long-term strength gain; this may affect the long-term durability of the concrete (Van Dam et al. 2005). Also, depending on the application, early opening times may be desired, which can significantly reduce the available curing time. The early strength criterion and enhanced durability may be most effectively achieved by using high-quality materials, by reducing the w/cm ratio, and by increasing the aggregate volume as long as workability is maintained (Van Dam et al. 2005). ASTM C666 is commonly used to assess the freeze-thaw durability of cementitious repair materials.

In addition to strength, other factors also play a role in the short- and long-term performance of the repair. For example, shrinkage characteristics and the CTE of the material should be considered. The drying shrinkage of many repair materials is greater than normal concrete, and when the material is restrained, it can increase tensile stresses. Differential expansion due to differences in the CTE between the repair material and the surrounding concrete can also be detrimental. Both of these factors can lead to poor or weak bond development, resulting in a delaminated repair that breaks up under loading. Also, minor edge deflections can sometimes be detected by traffic traveling across bituminous and some polymer-based repairs.

ASTM C928, Standard Specification for Packaged, Dry, Rapid-Hardening Cementitious Materials for Concrete Repairs, recommends the use of the slant-shear bond strength test method (ASTM C882, Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete by Slant Shear) to determine the bond strength between the repair material and the substrate concrete, with recommended 1- and 7-day performance requirements of 1,000 lbf/in² and 1,500 lbf/in², respectively. The slant shear test method is used to evaluate the bond strength when the interface between the repair material and the substrate concrete is subjected to the simultaneous action of compressive and shear stresses. To evaluate the tensile bond strength, ASTM C1583, Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-Off Method), can be used. Table 5.2 summarizes the typical laboratory test methods used to evaluate the mechanical, durability, and dimensional stability properties of cementitious repair materials along with some performance criteria for cementitious repair material selection.

Table 5.2. Laboratory test methods to evaluate properties of cementitious repair materials

Property	Test method	Example of performance criteria for the selection of rapid-setting cementitious repair material *
Set time	ASTM C403 (AASHTO T 197)	• Initial set: ≥15 min • Final set: 15 to 90 min
Compressive strength	ASTM C39 (AASHTO T 22)	• 2-hour: ≥2,500 lbf/in² • 3-hour: ≥3,000 lbf/in² • 1-day: ≥4,000 lbf/in² • 7- and 28-day: ≥5,000 lbf/in²
Flexural strength	ASTM C78 (AASHTO T 97)	• 2-hour: ≥350 lbf/in² • 7- and 28-day: ≥600 lbf/in²
Free/drying shrinkage	ASTM C157 (AASHTO T 160)	Length change after 28 days: -0.04% to +0.03%
Restrained shrinkage	ASTM C1581 (AASHTO PP 34)	No cracking after 28 days
Slant-shear bond strength	ASTM C882 (as specified by ASTM C928)	• 1-day: ≥1,000 lbf/in² (between repair material and concrete pavement) • 7-day: ≥1,250 lbf/in² (between repair material and concrete pavement) • 7-day: ≥1,500 lbf/in² (between repair material and repair material)
Tensile bond strength	ASTM C1583	Not specified
Modulus of elasticity	ASTM C469	$2 \text{ to } 6 \times 10^6$ lb/in²
Coefficient of thermal expansion	ASTM C531	≤7 in./in./°F × 10^{-6}
Freeze-thaw resistance	ASTM C666 (AASHTO T 161)	Not specified

*Based on Ramsey et al. 2020 (for airfield concrete pavement partial-depth spall repair applications)

Bonding Agents

The purpose of a bonding agent is to enhance the bond between the repair material and the existing pavement. Not all repair materials require a bonding agent, and in most cases simply having a clean, textured, and saturated surface-dry condition for the existing concrete is sufficient to ensure a good bond. One study evaluated the bond strength of 13 different cementitious repair materials using three different bonding test methods, and the results showed that a cement bonding agent did not necessarily improve the bond between the substrate concrete and the repair material, suggesting that saturated surface-dry conditions may be adequate for effective bonding for conventional cement-based repair materials (Dave et al. 2014). However, bonding agents are still specified by some agencies and may be required for some proprietary products.

If used, it is critical that the bonding agent not be allowed to dry out before the placement of the repair material, as this would inhibit bonding and lead to premature failure of the PDR. Sand-cement grouts are commonly used as bonding agents with concrete repair materials, but epoxy bonding agents have been used with both concrete and proprietary repair materials as a means of reducing repair closure times. One common cement grout formulation is as follows (Frentress and Harrington 2012):

- 2 parts Type I cement
- 1 part water (as needed to develop a creamy consistency)
- 1 part sand

This sand-cement-water grout mixture produces a mortar with a thick, creamy consistency, which helps to fill any small spalls or gouges that may be left by the removal process.

For its polyester concrete PDRs, Caltrans (2015) requires the use of a high-molecular-weight methacrylate (HMWM) bonding agent to penetrate microcracks in the substrate surface and to increase shear strength at the bond interface.

6. Construction Considerations

The construction and installation of PDRs involves the following steps:

1. Marking of repair boundaries
2. Concrete removal
3. Repair area preparation
4. Joint preparation
5. Bonding agent application
6. Repair material placement
7. Curing
8. Diamond grinding (as dictated by project conditions)
9. Joint resealing

A simplified overview of the PDR process is illustrated in Figure 5.6, with more details provided in the following sections.

A number of other resource documents describing the construction procedures for PDRs are also available (Wilson et al. 1999, ACPA 1998, Hoerner et al. 2001, ACPA 2004, ACPA 2006, Frentress and Harrington 2012).

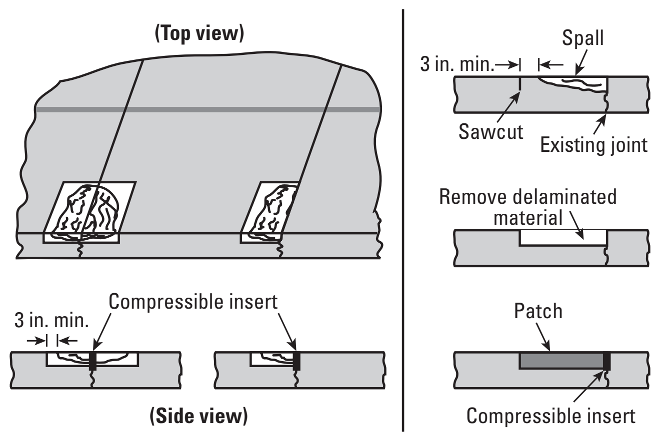

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Figure 5.6. Partial-depth repair details

Step 1: Repair Boundaries

Determining the boundaries for PDRs is often accomplished by "sounding" the concrete with a solid steel rod, a heavy chain, or a heavy hammer to determine unsound areas. Figure 5.7 shows two of these methods of sounding.

Areas yielding a sharp metallic ringing sound are judged to be acceptable, while those emitting a dull or hollow thud are delaminated or unsound (see Figure 5.8).

The repair boundaries should then be clearly marked to encompass all of the deterioration while keeping in mind the minimum repair dimension requirements for the repair material to be used. It is generally recommended that the repair boundaries extend at least 3 in. beyond the visible deterioration and any unsound areas. If there is a significant amount of time between the field marking and the construction process, the repair boundaries should be verified by the construction crew to ensure that deterioration has not increased. Figure 5.9 shows improperly and properly marked repair areas.

Andy Gisi, KDOT (bottom), from Frentress and Harrington 2012, CP Tech Center

Figure 5.7. Two methods of sounding a concrete pavement: hammer (top) and steel chain (bottom)

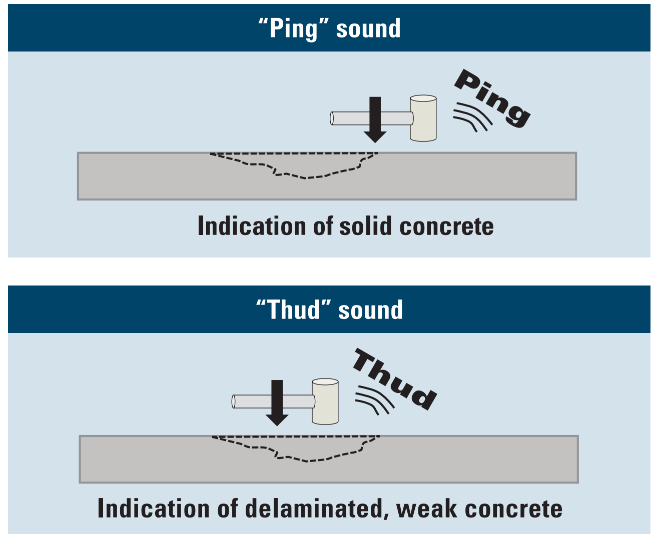

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Figure 5.8. Sounding a concrete pavement

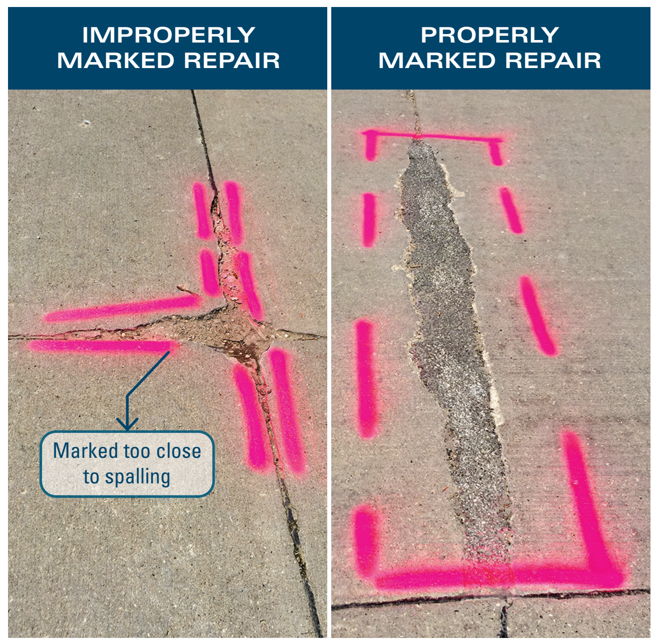

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Figure 5.9 Improperly and properly marked PDR areas

Step 2: Concrete Removal

The second step of the construction process is the removal of the unsound material. During this step, it is important to remember that PDRs should always be limited to no more than half of the slab thickness. In addition, most cementitious repairs should be at least 2 in. deep for the sake of weight and volume stability, but proprietary materials should follow the manufacturer's recommendations. Finally, the removal procedure should never expose any embedded dowel bars, but if it does and significant deterioration exists at that depth, then an FDR will be required.

The removal of the deteriorated concrete may be accomplished using one of four methods, which are described in the following sections:

- Saw-and-patch (Type 1 repairs)
- Chip-and-patch (Type 1 repairs)
- Mill-and-patch (Type 1 and 2 repairs)
- Clean-and-patch (emergency Type 1 repairs)

Saw-and-Patch Procedure (Type 1 Repairs)

This method employs a diamond-bladed saw to outline the repair boundaries. Typically, a single sawcut is made around the perimeter of the repair to a depth of 2 in. Smaller diameter blades (12 to 14 in.) are helpful in minimizing the amount of runout at repair corners (see Figure 5.10).

For larger repairs, additional sawing in the interior of the repair area in a crisscross pattern may be needed to help facilitate removal of the unsound concrete.

After sawing, removal of the unsound concrete is usually accomplished using a light jackhammer with a maximum weight of 15 lb; a jackhammer with a maximum weight of 30 lb may be allowed if damage to sound pavement is avoided (Wilson et al. 1999). The purpose of the weight restrictions is to reduce the possibility of breaking through the slab, but smaller-sized hammers can significantly affect productivity. If acceptable to the agency, and typically for larger repairs, some contractors elect to use heavier jackhammers or a hydraulic hammer or breaker attachment affixed to a backhoe or skid loader to help speed up production.

The jackhammer is also used to remove the polished vertical sawcut edge by chipping out concrete 2 in. beyond the sawcut to produce an angle between 30 and 60 degrees; this creates a roughened surface that promotes bonding of the repair material to the existing concrete, as shown in Figures 5.2 and 5.11. Care must be taken, however, to avoid fracturing the sound concrete below or causing shallow chips adjacent to the repair area.

The main advantages of the saw-and-patch procedure are that most crews are familiar with the method and it is cost-effective for small projects; drawbacks include its relatively slow productivity on larger projects and the potential for spalling to occur outside the sawcut boundaries during the jackhammering operation (Frentress and Harrington 2012).

Chip-and-Patch Procedure (Type 1 Repairs)

The chip-and-patch procedure shown in Figure 5.12 differs slightly from the saw-and-patch procedure in that the repair boundaries are not sawed. The deteriorated concrete in the center of the repair is removed using a lightweight jackhammer with a maximum weight of 15 lb; however, a jackhammer up to 30 lb or more may be allowed if damage to the sound pavement is avoided and if approved by the agency (Wilson et al. 1999). The material near the repair edge is then removed using either the light jackhammer or hand tools. Work should again progress from the inside of the repair toward the edges.

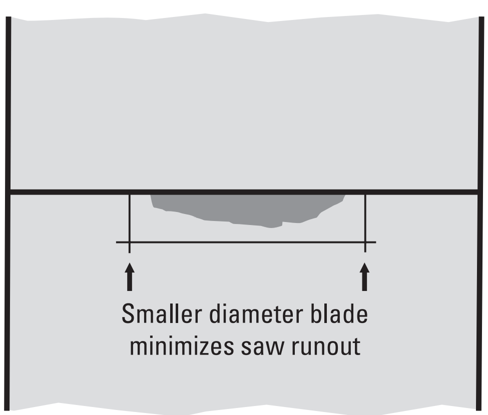

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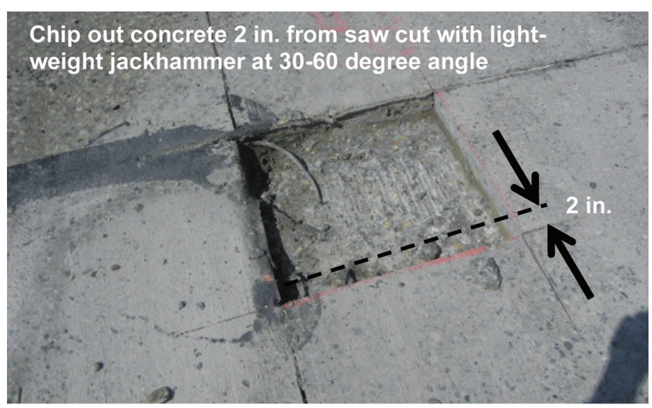
Figure 5.10. Minimizing sawcut runouts



Kevin McMullen, Wisconsin Concrete Pavement Association, from Frentress and Harrington 2012, CP Tech Center

Figure 5.11. Repair area prepared using the saw-and-patch (Type 1) procedure

John Donahue, MoDOT, used with permission

Figure 5.12. Repair area prepared using the chip-and-patch (Type 1) procedure

Kevin McMullen, Wisconsin Concrete Pavement Association, from Frentress and Harrington 2012, CP Tech Center

Figure 5.13. Milling options: milling along a joint (top) and dish-shaped milling perpendicular to a joint (bottom)

Mill-and-Patch Procedure (Type 1 and Type 2 Repairs)

Cold milling is a quick and efficient method for the removal of deteriorated concrete. Milling machines with cutting heads of 12 to 18 in. are commonly used, but they must be affixed with a mechanism that will stop penetration of the milling head at a preset depth. As depicted in Figure 5.13, the milling operation can proceed either along a joint or perpendicularly across a joint.

Milling along a joint is effective for spalling that occurs along the entire joint, whereas milling across a joint is effective for smaller, individual spalls. A small amount of chipping may be needed after the milling, along the edge of the repair or at joint intersections.

Milling produces a very rough, irregular surface that promotes a high degree of mechanical interlock between the repair material and the existing concrete. In a study for the Air Force, the cold milling machine was found to be the most efficient method of removal for PDRs (Hammons and Saeed 2010). Petrographic examinations of the milled repair area indicated no significant damage to the existing concrete, and post-traffic bond strength testing showed that the cold milling produced the highest degree of bonding as compared to the other concrete removal methods tested.

Cold milling has been used as part of the PDR procedure since at least the early 1980s, and it is now the standard practice for a number of Midwestern states.

Benefits of milling include the following (Frentress and Harrington 2012):

- The repair size is uniform for long-term success.
- The rough, irregular surface promotes bonding.
- Milling is efficient and economical when repairing large areas.
- Debris is easy to remove with a shovel and broom or a skid loader pickup broom.
- Milling is less labor intensive than jackhammer removal.

Drawbacks of the milling method of concrete removal are as follows (Frentress and Harrington 2012):

- Extra milling may be required to widen the original milled channel, especially when milling long cracks (e.g., longitudinal) to create a minimum distance of 3 in. to an outside milled face.
- Milling equipment and mobilization may be costly for small projects.

The common milling heads used in the industry today are the V head, rounded head, and the vertical edge. These are described in the following sections.

V- and Round-Shaped Concrete Milling

Milling heads manufactured to produce a V shape (as shown in Figure 5.14) or a round shape (as shown in Figure 5.15) can be used on longitudinal and transverse joints and cracks.

Concrete removal producing a tapered edge with a taper angle between 30 and 60 degrees to the bottom of the joint is the preferred shape to prepare for PDR. Milling with the V-head or rounded head has been used in PDR on transverse joints without any additional sawing and with only minor chipping at the edge of the repair required (Frentress and Harrington 2012).

Vertical Edge Milling

Vertical edge mill heads produce vertical edges along longitudinal and transverse joints and cracks (see Figure 5.16).

Since milling a vertical face has the potential for increased spalling along the top edge, a few highway agencies (such as the Kansas Department of Transportation [KDOT]) require sawcuts for all transverse joints repaired with partial-depth vertical edge milling. KDOT, however, does not require sawcuts for longitudinal joints unless excessive spalling occurs. Although potentially a risk, debonding issues have not been reported in PDRs installed on longitudinal or transverse joints when using the vertical edge milling technique (Frentress and Harrington 2012).

Kevin McMullen, Wisconsin Concrete Pavement Association, from Frentress and Harrington 2012, CP Tech Center

Figure 5.15. Rock saw capable of producing rounded milling (top) and milling pattern (bottom)

Daniel P. Frentress, Frentress Enterprises LLC, from Frentress and Harrington 2012, CP Tech Center

Figure 5.14. V-shaped milling head (top) and milling pattern (bottom)

Daniel P. Frentress, Frentress Enterprises LLC (top), and Kevin McMullen, Wisconsin Concrete Pavement Association (bottom), from Frentress and Harrington 2012, CP Tech Center

Figure 5.16. Vertical edge mill head (top) and milling pattern (bottom)

Clean-and-Patch Procedure (Emergency Type 1 Repairs)

The clean-and-patch procedure is used to perform emergency repairs under adverse conditions (Wilson et al. 1999). The procedure consists of removing deteriorated or loose concrete with hand tools or a light jackhammer (only used if the area is large and the cracked concrete is held tightly in place). The loosened material is then swept away with stiff brooms immediately before the placement of the patching material. Such a procedure should only be used if a spall presents a significant safety hazard and the conditions are so adverse that no other procedure can be used (Wilson et al. 1999).

Step 3: Repair Area Preparation

Following removal of the concrete, the surface of the repair area must be prepared to provide a clean, roughened surface for the development of a good bond between the repair material and the existing slab. Dry sweeping, light media blasting, and compressed air blasting are normally sufficient for obtaining an adequately clean surface. Media blasting, as shown in the top photo of Figure 5.17, is very effective at removing dirt, oil, thin layers of unsound concrete, and laitance, but care must be exercised not to spall the edges of the repair area.

John Donahue, MoDOT, used with permission

Figure 5.17. Media blasting (top) and air blasting (bottom) of PDR area

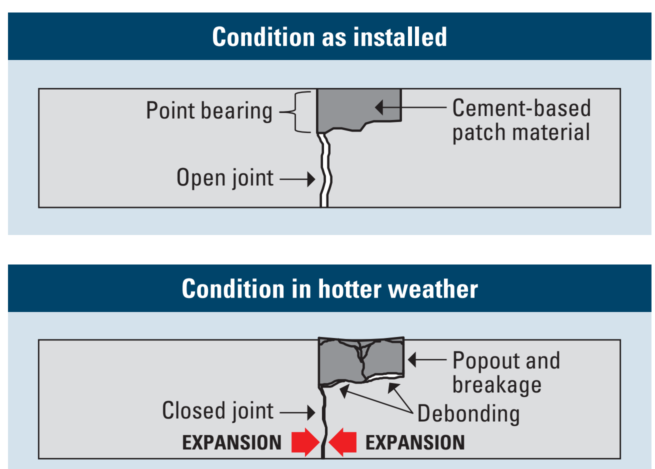
During the media blasting operation, steps must also be taken to limit worker exposure to respirable crystalline silica. The Occupational and Safety Health Administration (OSHA) Standard 1926.1153, Respirable Crystalline Silica, provides guidance on this topic in 29 C.F.R. § 1926.1153 (2016).

High-pressure water blasting is also used by a few agencies, although it is important that all slurry be removed prior to the placement of the repair material. These activities are then followed by air blasting for the final cleaning (see bottom photo of Figure 5.17), and it is important that the compressed air be free of oil that would otherwise inhibit bonding. This can be checked by placing a cloth over the air compressor nozzle and looking for any signs of oil discharge. Note that hot-poured polymer-modified resin materials generally do not require media blasting.

With any cleaning method, the prepared surface must be checked prior to placing the repair material. If a finger rubbed along the prepared surface picks up any loose material (e.g., dust, asphalt, or slurry), the surface should be cleaned again. Also, if there is a delay between the preparation/cleaning operation and the placement of the repair material, the surface of the repair should be cleaned again.

Step 4: Joint Preparation

A common cause for failure of PDRs at joints is excessive compressive stresses on the repair material. Partial-depth repairs placed directly against transverse joints and cracks will be crushed by the point-bearing compressive forces created when the slabs expand unless sufficient space is provided to accommodate thermal expansion (see Figure 5.18).



Adapted from ACPA, used with permission

Figure 5.18. PDR failure when a joint is prepared without a bond breaker

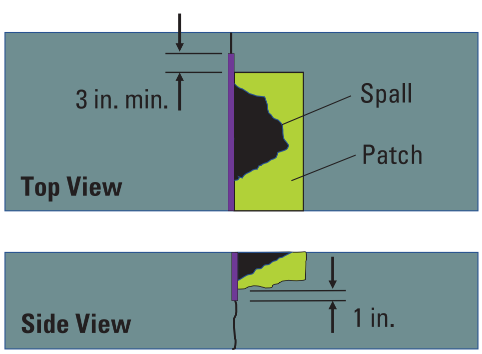
Failure may also occur when the repair material is allowed to infiltrate the joint or crack opening below the bottom of the repair, which would resist slab movement and thereby prevent the joint or crack from functioning.

To reduce the risk of such failures, a bond breaker or joint reformer (e.g., a strip of polystyrene, polyethylene, asphalt-impregnated fiberboard, waxed cardboard, or other compressible material) is placed between the new concrete and the adjoining slab, as illustrated in Figure 5.19.

This insert must be compatible with the repair material and must be placed so that it prevents intrusion of the repair material into the joint or crack opening. Failure to do so can result in the development of compressive stresses at lower depths that will damage the repair. It is recommended that the compressible insert extend 0.25 to 1 in. below the deepest removal depth for stability and extend 3 in. beyond the repair boundaries. Figure 5.20 illustrates the placement of the bond breaker on a PDR project.

To avoid cracking, most PDRs placed across joints or cracks require that the joint or crack be reestablished by using compression-absorbing materials or by sawing. If sawed, the sawcut must go through the entire thickness of the repair and as soon as the repair material has gained sufficient strength to permit sawing without significantly raveling the concrete. Timing is absolutely critical in the sawing operation, because any closing of the joint before sawing will fracture the repair.

As mentioned previously, certain proprietary “flexible” or “elastic” repair materials may have sufficient compressibility to accommodate joint movements without the need for a compressible insert. The manufacturers of these products should be consulted for appropriate joint treatment. Figure 5.21 shows a PDR being performed using proprietary polymer-based repair material without any aggregates, and Figure 5.22 shows a PDR featuring the use of a proprietary polymer-based repair material with premixed aggregates.



Recreated from Frentress and Harrington 2012, CP Tech Center

Figure 5.19. Bond breaker placement in PDR

John Donahue, MoDOT, used with permission

Figure 5.20. Placement of bond breaker on a PDR project

John Donahue, MoDOT, used with permission

Figure 5.21. Placement of proprietary polymer-based repair material without aggregates

John Donahue, MoDOT, used with permission

Figure 5.22. Placement of proprietary polymer-based repair material with premixed aggregates

Step 5: Bonding Agent Application

Concrete Repair Materials

If used, a bonding agent is applied after the surface of the existing concrete has been cleaned and just prior to placement of the repair material. Cement grouts are commonly used but epoxy grouts may be used when early opening times are required.

The existing surface should be in a saturated surface-dry condition prior to the application of cement grouts. When using epoxies or other manufactured grouts, the manufacturer's directions should be followed closely. Thorough coating of the bottom and sides of the repair area is essential. Commonly, epoxy grouts are applied with a soft brush while cement grouts use a stiff bristle broom (Harrington and Frentress 2012). Excess grout or epoxy should not be permitted to collect in pockets. The grout should be placed immediately before the repair material so that the grout does not set; should the grout material set, it will need to be removed by media blasting and fresh material reapplied before continuing. Figure 5.23 shows the application of a cement grout material to the existing concrete of a PDR area.

Proprietary Repair Materials

Bonding agents should only be used for proprietary repair materials as recommended by the manufacturer and applied in accordance with the proprietary repair material manufacturer's installation guidelines.

Jim Fox, MnDOT, from Frentress and Harrington 2012, CP Tech Center

Figure 5.23. Application of cement grout as bonding agent

Step 6: Repair Material Placement

Repair Material Mixing

The volume of material required for a PDR is usually small (0.5 to 2.0 ft^3). Small drum or paddle-type mixers or mobile mixers are often used to produce these mixtures. Based on trial batches, repair materials may be weighed and bagged in advance to facilitate the batching process. For long joint/crack repairs (such as that shown in Figure 5.24), ready mix or mobile mixers can also be used to produce the required amount of material in a more efficient manner (Frentress and Harrington 2012).

Careful observation of mixing times and water content for prepackaged rapid-setting materials is important because of the quick-setting nature of the materials. Mixing longer than needed for good blending reduces the already short time available for placing and finishing the material (Frentress and Harrington 2012). For polymer-based repair materials (especially the hot-applied materials), specialized mixing equipment is required to produce consistent mixtures (Ram et al. 2019).

Placement and Consolidation of Material

Concrete and most of the rapid-setting proprietary repair materials should not be placed when the air temperature or pavement temperature is below 40°F. Additional precautions, such as the use of warm water, insulating covers, and longer cure times, may be required at temperatures below 55°F.

Kevin McMullen, Wisconsin Concrete Pavement Association, from Frentress and Harrington 2012, CP Tech Center

Figure 5.24. Placement of PDR material for long joints using mobile concrete truck

Some epoxy concretes may require that the material be placed in lifts not exceeding 2 in. due to their high heat of hydration. The time interval between placing additional layers should be such that the temperature of the epoxy concrete does not exceed 140°F at any time during hardening.

Effective consolidation of the repair materials is critical and will help to avoid premature failures. Common methods of achieving consolidation include the following:

- Use of internal vibrators with small heads (diameter less than 1 in.)
- Use of vibrating screeds
- Rodding or tamping and cutting with a trowel or other hand tool (for very small repairs)

The internal vibrator (shown in Figure 5.25) and the vibrating screed or roller screed give the most consistent results. The internal vibrator is often more readily available and is commonly used.

The placement and consolidation procedure begins by slightly overfilling the area with repair material to allow for a reduction in volume during consolidation. The vibrator is held at a slight angle (15 to 30 degrees) from the vertical and is moved through the repair in such a way as to vibrate the entire repair area. The vibrator should not be used to move material from one place to another within the repair because this may result in segregation. Adequate consolidation is achieved when the mixture stops settling, air bubbles no longer emerge, and a smooth layer of mortar appears at the surface. On very small repairs, the mixture can be consolidated using hand tools. Cutting with a trowel seems to give better results than rodding or tamping. The tools used should be small enough to easily work in the area being repaired.

Kevin McMullen, Wisconsin Concrete Pavement Association, from Frentress and Harrington 2012, CP Tech Center

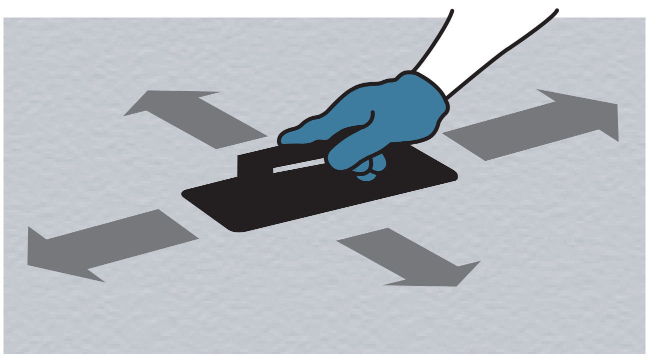
Figure 5.25. Consolidation of PDR material using internal vibrator

Screeding and Finishing

Partial-depth repairs are usually small enough that a stiff board can be used to screed the repair surface and make it flush with the existing pavement. The materials should be worked toward the perimeter of the repair to establish contact and enhance bonding to the existing slab (see Figure 5.26).

Partial-depth repairs cover a small area and have little effect on surface friction but are commonly textured to match the surrounding slab as much as possible. Figure 5.27 shows completed partial-depth joint repairs.

If the joint did not incorporate a compressible insert, a relief cut should be made to reestablish the joint before cracking of the repair material occurs. This should be made through the full thickness of the PDR plus 0.25 in. (Harrington and Frentress 2012).



CP Tech Center

Figure 5.26. Finishing PDR to the outside perimeter

Concrete Paving Association of Minnesota, used with permission

Figure 5.27. Completed PDRs

As soon as feasible after the PDR has been placed, the edges of the repair should be sealed with a one-to-one cement grout in order to form a moisture barrier over the interface and to impede delamination of the repair (ACPA 2006). Delamination of the repair can also occur if water at the interface freezes in cold weather (ACPA 2006). Sawcut runouts extending beyond the repair perimeter at repair corners can therefore also be filled with grout to help prevent moisture penetration that may negatively affect the bond (ACPA 2006). Alternatively, in lieu of grout, the runouts can be sealed with the material used to seal the adjacent joint or crack.

Step 7: Curing

Because PDRs have a large surface area in relation to their volume, moisture in PDRs can be lost quickly. Thus, curing is an important component of the construction process and must be effectively conducted in order to prevent the development of shrinkage cracks, which may lead to premature failure of the repair.

Curing Methods

For concrete materials, the most common curing method is to apply a white-pigmented curing compound (see Figure 5.28) as soon as the bleed water has evaporated from the repair surface.

Kevin McMullen, Wisconsin Concrete Pavement Association, Frentress and Harrington 2012, CP Tech Center

Figure 5.28. PDR material curing operations

Such curing compounds will reflect radiant heat while allowing the heat of hydration to escape and will provide protection for several days. Most curing compounds adhere to the requirements of ASTM C309 or AASHTO M 148. MnDOT has had poor experience with standard water-based curing compounds and now recommends the use of linseed-oil-based curing compounds for PDR (Darter 2017). Also, some agencies are specifying the use of poly-alpha-methylstyrene (PAM) curing compounds, which are white-pigmented materials with strong moisture retention characteristics.

Because of the greater potential for shrinkage cracking to occur with the relatively "thin" PDRs, some agencies require that curing compound be applied at 1.5 to 2 times the normal application rate. Moist burlap and polyethylene may also be used, and in cold weather the use of insulating blankets or tarps (see Figure 5.29) may be required to help retain heat and thereby ensure strength development.

Curing of proprietary repair materials should be conducted in accordance with the manufacturer's recommendations.

Opening to Traffic

It is important that the PDR attain sufficient strength before it is opened to traffic. As previously indicated in Table 5.1, a range of compressive strengths are specified by highway agencies for PDRs, but values on the order of about 2,000 lbf/in² may be appropriate given the confinement of this type of repair. Cylinders or beams cast in the field using material samples from the mixtures used for performing PDRs can be tested for strength to determine appropriate opening-to-traffic times. A recent review of state specifications and rehabilitation policies has recommended a compressive strength of 2,000 lbf/in² for both FDRs and PDRs (Collier et al. 2018).

John Donahue, MoDOT, used with permission

Figure 5.29. Insulating blanket being placed on finished PDR in cold weather conditions

Step 8: Optional Diamond Grinding

Rehabilitation techniques such as PDR may result in increased roughness if not finished properly. This is typically due to differences in elevation between the repair areas and the existing pavement. It is therefore often desirable to blend PDRs into a concrete pavement with diamond grinding, which leaves a smooth surface that matches the surrounding pavement.

When flexible polymer-based repair materials are used to repair areas in a concrete pavement that is scheduled to be diamond ground, the repairs must be performed at least 24 hours prior to the diamond grinding operation. The top layer of the repair area that is expected to be diamond ground should be fortified with moisture-free structural surfacing aggregate (per material manufacturer specifications). Key considerations for diamond grinding of a concrete pavement having large areas repaired with flexible polymer-based repair materials are summarized below (Ram et al. 2019):

- Loading and time of the grinding operations should be reduced to the extent possible. Heavy downward load applied by the grinding machine may remove too much material and this should be avoided. If the diamond-head blades sink too deep into the repair material, it will "gum up" the blades and can potentially cause material to be sucked into the vacuum pumps. Proper care must be exercised to avoid this situation.
- Grinding operations should be avoided when the ambient temperatures are high (temperatures when the flexible polymer-based repair material can become excessively soft).
- The grinding head must be kept as cool as possible.
- Repair areas need to be relatively small, as large repairs with these materials will gum up diamond-bladed grinding heads.
- To avoid these issues altogether, the installation of flexible repair materials alternatively could be performed after the grinding operation.

Step 9: Joint Resealing

The final step in the PDR procedure is the restoration of joints. This is accomplished by resawing the joint to a new shape factor, media blasting and air blasting both faces of the joint, inserting a closed-cell backer rod, and applying the sealer. More detailed information on joint resealing is provided in Chapter 10.

Construction of Partial-Depth Repairs in Continuously Reinforced Concrete Pavement

In some cases, localized crack, surface spalling, and horizontal delamination at steel depths in CRCP may be candidates for PDR. Effective PDR techniques performed in these situations can provide significant cost and time savings as compared to performing an FDR. A brief summary of the recommended TxDOT PDR process for CRCP is provided below (Yeon et al. 2012):

- **Evaluate the CRCP distress.** Evaluate whether PDR is an appropriate repair option for the distress observed by estimating the depth of the distress using sounding devices. The following features based on a visual examination of the distressed area may also be used to determine whether the condition can be suitably addressed using a PDR:
    - Multiple longitudinal cracks are present with small spacing occurring between transverse cracks that are also closely spaced.
    - The distress is located near a longitudinal construction joint (see Figure 5.30, left photo) or longitudinal contraction joint or in the wheel path (see Figure 5.30, right photo).
    - There is no faulting at the longitudinal joints.
- **Determine repair boundaries.** The general process is the same as described earlier (see Section 6, Step 1), except that in CRCP partial-depth distresses are generally confined longitudinally by transverse cracks. Repair boundaries should include the longitudinal and transverse cracks and should generally extend 3 to 4 in. beyond the cracks since the full extent of the distress is not always apparent. Partial-depth repair area dimensions in CRCP should be at least 2 ft by 2 ft.
- **Remove deteriorated concrete.** It is important that the sawcut depth not reach the longitudinal steel. The sawcut concrete is then removed using jackhammers with a maximum weight of 37.5 lb or using a carbide-tipped milling machine. Once all the distressed concrete has been removed, the condition of the remaining substrate concrete should be examined for any damage that may have occurred due to the concrete removal activity. If the longitudinal steel is damaged or ruptured during the concrete removal process, FDRs should instead be performed.
- **Clean repair area.** The repair area should be cleaned first with compressed air to remove the fine material. Next, media blasting should be applied to remove the remaining fine material and any other debris on the repair surface. Cement paste on any exposed reinforcing steel should also be removed by media blasting.
- **Mix, place, and cure repair material.** Similarly to the PDR processes described earlier (see Section 6, steps 6 and 7), once the repair area has been prepared, the next steps are to mix, place, and adequately cure the repair material that has been selected to meet the opening requirements of the project. Depending on the type of material used, the surface may need to be wetted prior to the placement of the repair material. For conventional cementitious mixtures, a saturated surface-dry condition may be adequate for effective bond with the underlying concrete. Again, the curing compound is applied at a much higher rate for PDRs than for conventional FDRs because of the greater surface-area-to-volume ratio of thin repairs.
- **Open to traffic.** The partial-depth repair of CRCP is opened to traffic once it reaches the specified design strength; the use of maturity in monitoring concrete strength gain is recommended.

Yeon et al. 2012, Center for Multidisciplinary Research in Transportation, Texas Tech

Figure 5.30. Partial-depth distress in CRCP near longitudinal joint (left) and in the wheel path (right)

7. Quality Assurance

The combination of proper design procedures and sufficient construction quality control is extremely important to achieving well-performing PDRs. On many projects where construction inspections have been known to be less stringent, the performance of PDRs has often been less than satisfactory. This section summarizes key portions of the *Partial-Depth Repair of Portland Cement Concrete Pavements* checklist that was created by the CP Tech Center for the FHWA to guide state and local highway agencies on the design and construction of well-performing PDRs (FHWA 2019). These checklist items are divided into the general categories of preliminary responsibilities, project inspection responsibilities, and cleanup responsibilities.

Preliminary Responsibilities

Agency and contractor personnel should collectively conduct a review of the project documentation, project scope, intended construction procedures, materials usage, and associated specifications. Such a collective review is a continuous process throughout the entire duration of the job and is intended to minimize any misunderstandings in the field between agency designers, construction personnel, and inspectors. Specific items for this review are summarized below.

Project Review

An updated review of the current project's condition is warranted to ensure that the project is still a viable candidate for PDR. Specifically, the following items should be verified as part of the project review process:

- Verify that pavement conditions have not significantly changed since the project was designed and that a PDR is still appropriate for the pavement.
- Document the potential reasons behind the conditions observed and note if the distresses are primarily due to joint-related distress or more traditional spalls resulting from construction defects and/or incompressibles in the joints.
- Verify that the estimated number of PDRs agrees with the number specified in the contract.
- Agree on quantities to be placed but allow flexibility if additional deterioration is found below the surface.
- Some PDRs may become FDRs if deterioration extends below the top half of the slab thickness. Make sure that the criteria for identifying this change are understood.

Document Review

The key project documents should be reviewed prior to the start of any construction activities. Some of the critical project documents include the following:

- Bid/project specifications and design
- Applicable special provisions
- Traffic control plan
- Manufacturer's specific installation instructions for the selected repair material(s)
- Manufacturer's material safety data sheets (MSDSs)

Materials Checks

A number of materials-related checks are recommended prior to the start of a PDR project. Specifically, agency and contractor personnel should collectively verify the following:

- The selected repair material is of the correct type and meets specifications.
- The repair material is obtained from an approved source or is listed on the agency's qualified products list as required by the contract documents.
- The repair material has been sampled and tested prior to installation as required by the contract documents.
- Additional or extender aggregates have been properly produced and meet requirements of the contract documents.
- The material packaging is not damaged so as to prevent proper use (for example, packages are not leaking, torn, or pierced).
- The material age is within the manufacturer-recommended shelf life.
- The bonding agent (if required) meets specifications.
- The curing compound (if required) meets specifications.
- The joint/crack reformer material (compressible insert) meets specifications (typically polystyrene foam board, 12 mm [0.5 in.] thick).
- The joint sealant material meets specification requirements.
- Sufficient quantities of materials are on hand for completion of the project.

Equipment Inspections

All equipment that will be utilized in the construction of PDRs should be inspected prior to construction. Ensuring that construction equipment is in good working order will help avoid construction-related problems during the construction process. The following items should be verified as part of the equipment inspection process prior to the start of a PDR project.

Concrete Removal Equipment

- Verify that concrete saws are of sufficient weight and horsepower to adequately cut the existing concrete pavement to the depth along the repair boundaries required by the contract documents.
- Verify that the concrete saws and blades are in good working order.
- Verify that pavement milling machines are power-operated, self-propelled, cold-milling machines capable of removing concrete as required by the contract documents.
- Verify that milling machines used for concrete removal are equipped with a device that allows them to stop at preset depths to prevent removal of more than the top third of the slab and to prevent damage to embedded steel.
- Verify that the maximum rated weight of removal jackhammers is 30 lb, unless a heavier hammer is otherwise approved by the agency.

Repair Area Preparation Equipment

- Verify that the media-blasting unit is adjusted for the correct rate and that it is equipped with and using properly functioning oil/moisture traps.
- Verify that air compressors have sufficient pressure and volume capabilities to clean the repair area adequately in accordance with contract specifications.
- Verify that air compressors are equipped with and using properly functioning oil and moisture filters/traps. (This can be accomplished by placing a cloth over the air compressor nozzle and visually inspecting for oil.)
- Verify that the volume and pressure of water-blasting equipment (if used) meets the contract specifications.

Mixing and Testing Equipment

- Verify that auger flights and paddles within auger-type mixing equipment are kept free of material buildup that can result in inefficient mixing operations.
- Ensure that volumetric mixing equipment such as mobile mixers are kept in good condition and are calibrated on a regular basis to properly proportion mixes.
- Verify that the concrete testing technician meets the requirements of the contract documents for training/certification.
- Ensure that material test equipment required by the specifications is all available on site and in proper working condition. (Equipment typically includes a slump cone, pressure-type air meter, cylinder molds and lids, as well as a rod, mallet, ruler, and 10 ft straightedge.)

Placing and Finishing Equipment

- Verify that a sufficient number of concrete vibrators (1 in. diameter or smaller) are available on site and are in proper working condition.
- Verify that all floats and screeds are straight, free of defects, and capable of producing the desired finish.

Other Equipment

- Ensure that a steel chain, rod, or hammer is available to check for unsound concrete around the repair area.
- Verify that grout application brushes (if necessary) are available.

Weather Requirements

Immediately prior to the start of the construction project, the following weather-related concerns should be checked:

- Review manufacturer installation instructions for requirements specific to the repair material being used.
- Verify that air and surface temperatures meet manufacturer and contract requirements for the placement of the repair material (commonly 40°F and rising but no more than 90°F).
- Ensure that the repair activity does not proceed if rain is imminent.

Traffic Control

The developed traffic control plan should be reviewed by field personnel prior to construction. Specifically, the following pre- and postconstruction traffic-related activities should be performed:

- Verify that the signs and devices used match the traffic control plan stipulated in the contract documents.
- Verify that the traffic control setup complies with the Federal *Manual on Uniform Traffic Control Devices (MUTCD)* or local agency traffic control procedures.
- Verify that traffic control personnel are trained/qualified according to contract documents and agency requirements.
- Verify that unsafe conditions, if any, are reported to a supervisor.
- Verify all workers are wearing the required personal protective equipment (PPE).
- Ensure that the repaired pavement is not opened to traffic until the repair material meets the strength requirements stated in the contract documents.
- Verify that signs are removed or covered when they are no longer needed.

Project Inspection Responsibilities

During the construction process, careful project inspection by construction inspectors helps ensure well-performing PDR installations. Specifically, the following checklist items (organized by construction activity) summarize the recommended project inspection items.

Repair Area Removal and Cleaning

- Ensure that the area surrounding the repair is checked for delamination and unsound concrete using a steel chain, rod, or hammer.
- Ensure that the boundaries of unsound concrete area(s) are marked at least 3 in. beyond the area of deterioration.
- Verify that concrete is removed by either (1) sawcutting the boundaries and jackhammering interior concrete, (2) using just chipping hammers, or (3) using a cold milling machine.
- Verify that vacuum equipment used with sawing operations to remove slurry or collect dust is functioning properly.
- Verify that concrete removal extends at least 2 in. deep and does not extend below half of the slab thickness and that load transfer devices are not exposed. If dowels are exposed, verify that the repairs are conducted in accordance with the applicable specification. Verify that, after concrete removal, the repair area is prepared by light media blasting or water blasting.
- Verify that the repair area is cleaned by air blasting. A second air blasting may be required immediately before placement of the repair material if the repair areas are left exposed for a period of time longer than that specified in the contract documents.

Repair Area Preparation

- Ensure that the repair area is effectively media blasted to remove any dirt, debris, or laitance.
- Ensure that compressible joint inserts (joint/crack reformers) are inserted into existing cracks/joints in accordance with contract documents. Compressible joint inserts are typically required to extend 0.25 to 1 in. below the deepest removal depth and 3 in. beyond the repair boundaries. Tooling and sawing may be used in lieu of compressible inserts to reestablish the joint.
- When a repair abuts a bituminous shoulder, ensure that a wooden form is used to prevent the repair material from entering the shoulder joint.
- Prewetting of patch areas with water mist prior to grout application may help prevent moisture loss from grout.
- Ensure that the bonding agent (epoxy- or cement-based) is placed on the clean, prepared surface of the existing concrete immediately prior to the placement of the repair material as required by the contract documents. If the bonding agent shows any sign of drying before the repair material is placed, it must be removed by media blasting, cleaned with compressed air, and reapplied.
- Verify that cement-based bonding agents are applied using a wire brush and epoxy bonding agents are applied using a soft brush.

Placing, Finishing, and Curing Repair Material

- Verify that manufacturer recommendations are followed when mixing and placing the repair material.
- Verify that quantities of repair material being mixed are relatively small to prevent material from setting prematurely.
- Verify that the fresh concrete is properly consolidated using several vertical penetrations of the surface with a handheld vibrator.
- Verify that the surface of the concrete repair is level with the adjacent slab using a straightedge in accordance with contract documents. The material should be worked from the center of the repair outward toward the boundary to prevent pulling material away from the repair boundaries.
- Verify that the surface of the fresh repair material is finished and textured to match the adjacent surface.
- Verify that the perimeter of the repair and sawcut runouts (if saws are used) are sealed using grout material. Alternatively, sawcut runouts can be sealed using joint sealant material.
- Verify that adequate curing compound is applied to the surface of the finished and textured fresh repair material in accordance with contract documents.
- Ensure that insulation blankets are used when ambient temperatures are expected to fall below 40°F. Maintain blanket cover until the concrete attains the strength required in the contract documents.

Resealing Joints and Cracks

- Verify that the compressible inserts are sawed out to the dimensions specified in the contract documents when the repair material has attained sufficient strength to support concrete saws.
- Verify that joints are cleaned and resealed according to contract documents.

Cleanup Responsibilities

- Verify that all concrete pieces and loose debris are removed from the pavement surface and disposed of in accordance with contract documents. It is recommended that debris removal be performed daily using power brooms.
- Verify that all equipment used for mixing, placement, and finishing is properly cleaned for the next use.

Inspection/Acceptance

Several agencies check the bonding condition of PDRs by sounding using chains or rods as part of the inspection process (Darter 2017). The identification of any debonding or the presence of early cracking will trigger the requirement for PDR removal. At least one agency specifies a 30-day warranty period on PDR projects (Darter 2017).

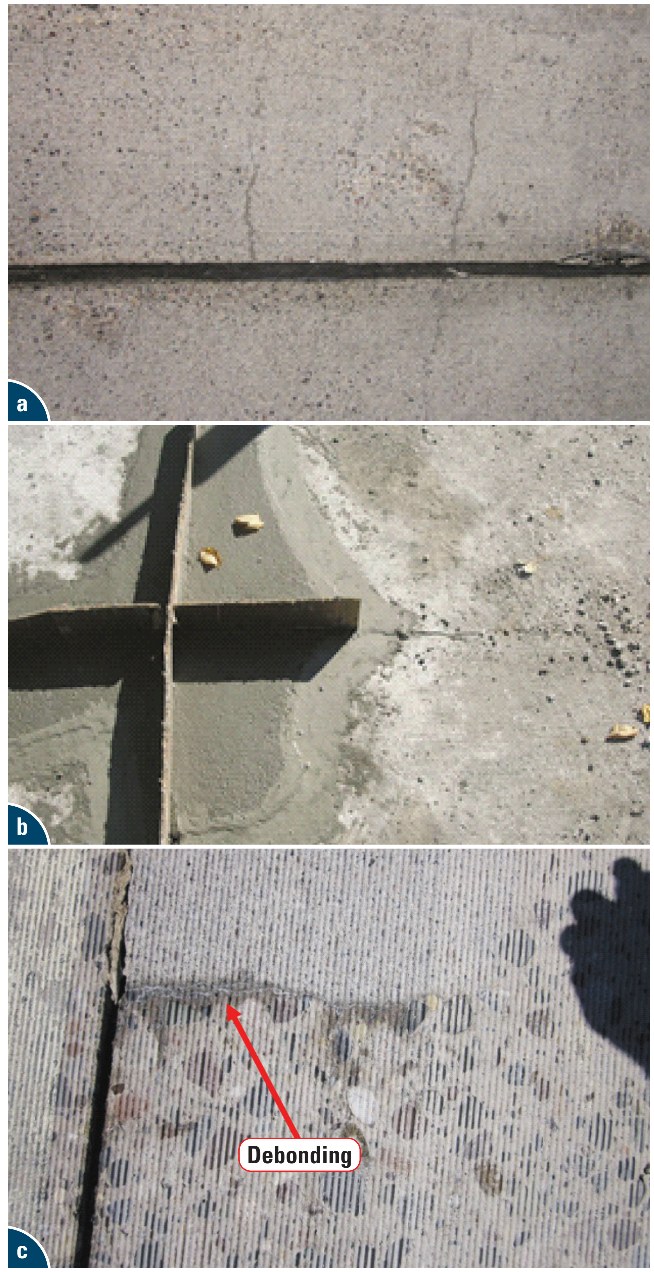
8. Troubleshooting

As mentioned previously, poor performance of a PDR is typically attributed to inappropriate use, improper design, or improper construction and placement techniques. Although paying close attention to the checklist items in the previous section minimizes design- and construction-related problems, construction problems do sometimes develop in the field. Some of the more typical problems that are encountered either during or after construction are summarized in Table 5.3. Typical causes and recommended solutions accompany each of the identified potential problems.

Table 5.3. Potential PDR-related construction problems and associated solutions

Problem	Typical solution(s)
Deterioration found to extend beyond the originally planned repair boundaries	The first solution is to extend the limits of the repair area to encompass all of the deterioration. If the deterioration is found to extend significantly deeper than expected (i.e., deeper than one-half of the slab thickness), an FDR should be placed instead of the planned PDR.
Repair failures associated with inadequate provision of compression relief (Figure 5.31a)	The typical solution is to replace the repair, being sure to provide adequate compression relief.
Dowel bar exposed during concrete removal	If only a small portion of the dowel bar is exposed and no further deterioration around the dowel is evident, place duct tape over the exposed area of the dowel bar and proceed with the PDR. If deterioration is present around and beneath the dowel bar, an FDR should be used instead of the planned PDR.
Reinforcing mesh in JRCP is exposed during concrete removal	If the steel is in the upper half of the slab, the steel should be cut back to the edges of the repair area and the placement of the repair should continue as planned. If the exposed steel is below the upper half of the slab, however, an FDR should be used instead of the planned PDR.
Repair material flows into joint or crack	When this problem is observed, there are two solutions: either remove and replace the repair, or mark the joint for sawing as soon as it can support a saw without raveling the mixture. If repair material is allowed to infiltrate a crack, it should be removed and replaced.
Shrinkage cracking and/or surface scaling due to improper finishing and/or curing (Figure 5.31b)	Minor surface scaling and/or shrinkage cracking is typically not a major issue; however, the repair must be monitored for signs of additional deterioration. If excessive scaling and/or cracking is observed, the repair must be replaced.
Repair cracking or debonding of repair material (Figure 5.31c)	If the repair fails prematurely due to one of these causes, the only practical solution is to replace the distressed repair. It is important to try to determine the cause of the premature failure, however, in order to avoid repeating the same mistake on future repairs.

Sources: Adapted from ACPA 2006, Frentress and Harrington 2012, CP Tech Center 2019



Kevin McMullen, Wisconsin Concrete Pavement Association (top), Daniel P. Frentress, Frentress Enterprises LLC (middle), and Gordy Bruhn, MnDOT (bottom), from Frentress and Harrington 2012, CP Tech Center

Figure 5.31. Repair failures associated with (a) poor compression relief, (b) improper curing/finishing, and (c) improper grout placement resulting in debonding

9. Summary

Partial-depth repairs are an excellent tool for restoring rideability and the overall integrity of a concrete pavement. Various products are available for these types of repairs, and the selection of the proper material is dependent upon the specific project requirements. The selection of repair materials should not be based solely on the strength and other engineering properties of the repair material itself. Instead, careful consideration must be given to the compatibility of the properties of the repair material with the existing substrate concrete. Each material will call for different handling and mixing steps. All PDR products, however, require the same surface preparation steps. Taking the time to properly prepare the repair area, following the manufacturers' recommendations when placing the materials, and paying attention to weather concerns during placement and curing will all contribute to the long-term performance of the PDR.

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Chapter 6
Full-Depth Repairs

1. Introduction

Full-depth repairs are a primary preservation treatment that can be used to restore the rideability and structural integrity of concrete pavements. Full-depth repairs are used to address intermittent distresses (such as transverse cracking, corner breaks, deteriorated joints, blowups, and punchouts) that can develop over the life of the pavement and are also an effective pre-overlay treatment to prepare distressed concrete pavements to receive a structural overlay (either asphalt or concrete). However, concrete pavements with an extensive amount of cracking and other structural distresses may not be good candidates for FDRs.

The performance expectations for FDRs will vary depending on the condition of the existing pavement and type of repair application, but service lives of 20 years or more have been achieved on many FDR projects over a range of climates, traffic loadings, materials, and designs (Darter 2017). Long-lasting FDRs are dependent upon many items, including appropriate project selection, effective load transfer design, and effective construction procedures.

This chapter focuses on proper techniques that can be used to design and install cast-in-place FDRs on JPCP, JRCP, and CRCP designs. In addition, the use of precast slabs for repair of concrete pavements is presented, along with guidelines on performing utility cut restoration for concrete pavements.

2. Purpose and Project Selection

Full-depth repairs consist of either cast-in-place concrete or precast slabs that replace the entire thickness of an existing slab to address cracking and other significant deterioration. The repairs may include the complete removal and replacement of an individual slab (or several consecutive slabs) or may be confined to the removal and replacement of portions of a slab or portions of adjacent slabs. FDRs can be applied to all concrete-surfaced pavements, including concrete overlays.

In a preservation mode, the expectation is the FDRs will be used to address intermittent distresses that have developed on an existing concrete pavement. For newer pavements, this may be to address a few isolated distresses that have developed prematurely, but for older pavements, this may include more frequent repairs in order to maintain the serviceability of the pavement and extend its service life.

Jointed Plain Concrete Pavement/ Jointed Reinforced Concrete Pavement

Table 6.1 provides a summary of the JPCP and JRCP distresses and severity levels that can be successfully remedied using FDRs, although other treatments (such as PDRs) may still be appropriate in some cases. The distress severity levels are based on the criteria used in the FHWA Long-Term Pavement Performance Program (Miller and Bellinger 2014).

State and local highway agencies employ different practices in identifying the need for FDR of jointed pavements, some of which focus more on rehabilitation instead of preservation. Some examples of state transportation department practices in selecting FDR for jointed concrete pavements include the following (Darter 2017):

- California (JPCP):
  - For projects with <10% stage 3 cracking (i.e., slabs in three or more pieces), individual slab replacement
  - For projects with 10% to 20% stage 3 cracking, life-cycle cost analysis required to determine whether to replace slabs or lanes
  - For projects with >20% stage 3 cracking, lane replacement of all slabs
- Missouri (JPCP and JRCP): Varies based on the extent of longitudinal cracking, transverse cracking, and JRCP spalling present in the pavement
- Utah (JPCP): Presence of more than two cracks per slab
- Washington (JPCP): Presence of corner breaks, transverse cracks within 4 to 5 ft of transverse joints, settled slabs, highly distressed DBR slots, panels in three or more pieces, and a single unrepaired panel between two repaired panels
- Minnesota: Panels with deteriorated transverse joints and cracks, particularly in the wheel path
- Georgia: Presence of transverse, longitudinal, and corner cracking

Many of these recommendations largely look at the type and severity of the distress, but consideration must also be given to the extent of distress within a project in determining the appropriateness for FDR. Good candidates for the application of FDRs are concrete pavements in which deterioration is limited to the joints or cracks, provided that the deterioration is not widespread over the entire length of the project.

Table 6.1. Candidate JPCP/JRCP distresses addressed by FDRs

Distress type	Distress severity levels that could trigger FDR
Transverse cracking	Medium, high
Longitudinal cracking	Medium, high
Corner break	Low, medium, high
Spalling of joints	Medium,[1] high
Blowup	Low, medium, high
D-cracking (at joints or cracks)[2]	Medium,[1] high
Reactive aggregate spalling[2]	Medium,[1] high
Deterioration adjacent to existing repair	Medium,[1] high
Deterioration of existing repairs	Medium,[1] high

[1] Partial-depth repairs can be used if the deterioration is limited to the upper half of the pavement slab.

[2] If the pavement has a severe material problem (such as D-cracking or reactive aggregate), FDRs may be used as a stopgap measure, with continued deterioration of the original pavement likely to occur.

Note: Roadways with low traffic volumes may not require repair at the recommended severity level.

Source: Hoerner et al. 2001

Table 6.2. Candidate CRCP distresses addressed by FDRs

Distress type	Distress severity levels that could trigger FDR
Punchout	Low, medium, high
Deteriorated transverse cracks[1]	Medium, high
Longitudinal cracking	Medium, high
Blowup	Low, medium, high
Construction joint distress	Medium, high
Localized distress	Medium,[2] high
D-cracking (at cracks)[3]	High
Deterioration adjacent to existing repair	Medium,[2] high
Deterioration of existing repair	Medium,[2] high

[1] Typically associated with ruptured steel.

[2] Partial-depth repairs can be used if the deterioration is limited to the upper half of the pavement slab.

[3] If the pavement has a severe material problem (such as D-cracking or reactive aggregate), FDRs may be used as a stopgap measure, with continued deterioration of the original pavement likely to occur.

Note: Roadways with low traffic volumes may not require repair at the recommended severity level.

Source: Hoerner et al. 2001

Concrete pavements exhibiting severe structural distresses over an entire project are more suited for a structural overlay or reconstruction.

Continuously Reinforced Concrete Pavement

Table 6.2 provides a summary of the CRCP distresses and severity levels that can be successfully remedied using FDRs.

Punchouts are the major structural distress in CRCP and are commonly addressed with FDRs. Some examples of state transportation department practices in selecting FDR for CRCP include the following:

- TxDOT has the most mileage of CRCP in the US and has a program of successful FDR practices that provide good performance. The two primary distresses that TxDOT targets with FDR are punchouts and deep spalling at cracks, the latter of which is often the result of coarse aggregates with a high coefficient of thermal expansion (TxDOT 2019).
- The Illinois Department of Transportation (IDOT) addresses a range of CRCP distresses with FDR, including punchouts, deep spalling, blowups, and terminal joint failures (IDOT 2010).
- The California Department of Transportation (Caltrans) considers punchout severity and frequency in determining general strategy selection, as indicated below (Caltrans 2015):
    - Low severity: None
    - Medium severity (2 to 5 punchouts per mile): FDR
    - Medium severity (6 to 9 punchouts per mile): FDR or asphalt overlay
    - High severity (more than 10 punchouts per mile): lane replacement, asphalt overlay, or unbonded concrete overlay

3. Limitations and Effectiveness

Although FDRs can be designed and constructed to provide good long-term performance, the performance of FDRs is dependent on their appropriate application and the use of effective design and construction practices. Many FDR performance problems can be traced back to inadequate design (particularly poor load transfer design), poor support conditions, poor construction quality, or the placement of FDRs in pavements that are too far deteriorated. Thus, project selection is very important to obtain the desired long-term performance. Important points for consideration in selecting FDR treatment include the following:

- If the existing pavement is structurally deficient or is nearing the end of its fatigue life, a structural overlay is needed to prevent continued cracking of the original pavement.
- If the original pavement has a severe materials-related problem (e.g., D-cracking or reactive aggregate), FDRs can be used to help maintain serviceability, but deterioration of the original pavement is likely to continue.
- Additional joints introduced by FDRs can add to pavement roughness, so diamond grinding may need to be considered following the installation of FDRs.
- As an alternative to FDR, nondeteriorated transverse cracks in JPCP may be repaired by retrofitting dowel bars, and longitudinal cracking may be addressed using cross-stitching.

In summary, the effectiveness of FDRs depends strongly on the selection of a suitable project and on the proper design and installation of the FDR (particularly in terms of providing good support conditions and effective load transfer).

4. Materials and Design Considerations

This section presents the materials and design considerations for FDRs of JPCP/JRCP, as well as special design considerations for FDRs of CRCP. For each pavement type, guidance is provided on selecting repair locations and boundaries, selecting repair materials, restoring load transfer, and determining when to open the repaired pavement to traffic.

Selecting Repair Locations and Boundaries

Jointed Plain Concrete Pavement/Jointed Reinforced Concrete Pavement

The first step in the installation of FDRs is the selection of the repair boundaries. Distressed areas must be identified and marked, with special consideration given to those areas of extensive distress that might require complete slab replacement.

For JPCP, all structural cracks are candidates for FDR since there is no embedded steel to hold the cracks tight. The rate at which the structural cracks in JPCP deteriorate depends on traffic, climate, and pavement structure.

JRCP designs often exhibit both deteriorated joints and midpanel cracks that also deteriorate under repeated heavy traffic loadings. Midpanel transverse cracks are common in this design, and the reinforcing steel in the slab is expected to hold the cracks tight. Unfortunately, some of these midpanel cracks may break down due to corrosion of the steel, or they may deteriorate because of “frozen” or locked doweled transverse joints (which force the cracks to absorb the movements the doweled joints were designed to accommodate). These cracks soon lose their aggregate interlock under repeated heavy traffic loadings. Some JRCP projects will have joints with very little deterioration but may exhibit transverse cracks in each slab that are open and essentially acting as joints.

The types of jointed pavement distresses that can be successfully addressed through FDRs are presented in the previous Table 6.1. Each agency should examine these recommendations and modify them as needed to develop an approach that more closely reflects local conditions.

One special distress to note is the occurrence of blowups, which, because of the potential safety hazard they create, almost always require immediate action to restore the pavement to a safe and serviceable condition. These emergency repairs are often done by agency maintenance forces and may use temporary patching materials until a more permanent repair can be performed at a later date. It is important that the entire area of the blowup be encompassed in the repair, which may include the need for removal and replacement for portions of the affected slabs (or even additional slabs) on both ends of the distressed area.

Sizing the Repair

After the repair locations are identified, the boundaries of each repair must be selected to encompass all of the significant deterioration in the slab and in the underlying layers. The extent of deterioration beneath the slab surface may be identified through coring and deflection studies; this is an important step as it is common for deterioration at the bottom of the slab to extend beyond the visible boundaries of deterioration at the surface (see Figure 6.1), particularly for pavements exhibiting MRD. In addition, because patching surveys are often done well in advance of the actual patching project, it is important that the appropriateness of the boundaries be confirmed during construction.

Engineering judgment is required in selecting repair boundaries, which should be based on performance history, production efficiency, and economics. The following general guidelines regarding dimensions for FDRs of JPCP/JRCP are recommended based on agency practices and experience (ACPA 2006, Darter 2017):

- **Repair Length**
  - Most state transportation departments use a minimum repair length of 6 ft (in the longitudinal direction) to minimize rocking, pumping, and breakup of the slab. In addition, the larger repair area allows additional room for compaction and dowel hole drilling equipment. The practices of a few selected state agencies related to repair length include the following (Darter 2017):
    - California, Missouri, Georgia, and Washington: 6 ft
    - Utah: 5 ft
    - Minnesota: 4 ft
  - The length of the FDR repair should not exceed 15 ft without an intermediate joint to prevent the development of transverse cracking within the repair. Consecutive slab replacements can be performed, but the repairs should maintain a maximum slab length of 15 ft.
  - In estimating quantities for an FDR that extends to an existing transverse joint, it is important to recognize that the existing dowel bar system in the adjacent remaining slabs will need to be removed to accommodate the new dowel bar system.

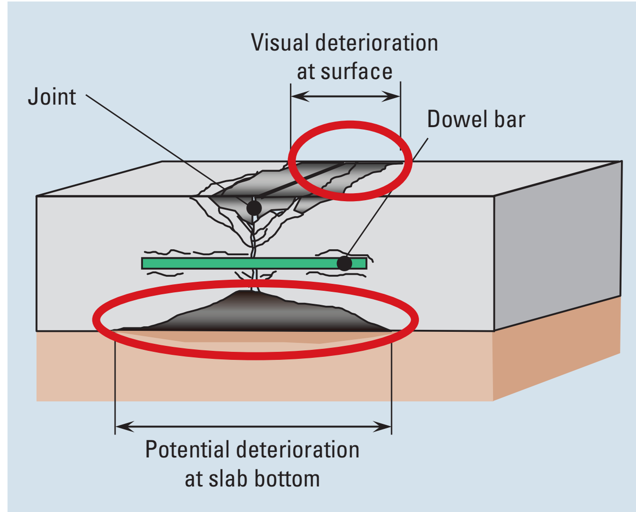

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Figure 6.1. Potential deterioration beneath a joint extending beyond the boundaries of visible surface deterioration

- **Repair Width**
  - Full-lane-width repairs are generally recommended because the repair boundaries are then well defined and the repair is more stable. However, some agencies allow partial-lane-width repairs (e.g., a half-lane width) provided that the longitudinal repair joint is kept out of the wheel path; this is also a common approach on lower volume roadways.
- **General Considerations for Boundaries**
  - Use straight-line sawcuts, forming rectangles in line with the jointing pattern.
  - Ensure the repair boundaries are a minimum of 2 ft from nearby transverse joints. Extend the repair boundary to the joint if it is within 2 ft.
  - Make one large repair if the individual repairs are 8 to 12 ft from each other in a single lane. This alternative requires two sawcuts instead of four, as well as one removal instead of two. Table 6.3 provides general guidelines for the threshold values for combining FDRs.

Figure 6.2 provides an example illustration of repair boundary selection when multiple distresses of different severities are present. Note that not all distresses require an FDR, and actual practices may depend on overall project conditions and traffic levels.

Table 6.3. Threshold distance between FDRs below which repairs should be combined

Pavement thickness (in.)	Threshold distance for 11 ft lane width (ft)	Threshold distance for 12 ft lane width (ft)
6	16	15
7	14	13
8	12	11
9	11	10
10	10	9
11	9	8
S12	8	8

Note: If the distance between full-depth patches is less than the threshold value, they should be combined into one repair.

Source: ACPA 2006

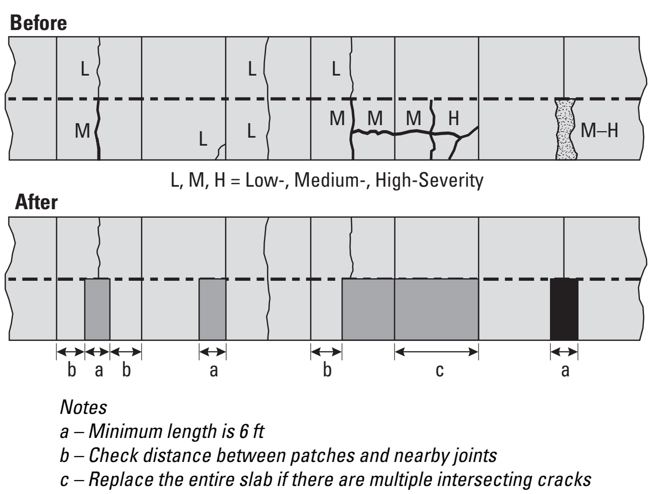

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Figure 6.2. Selection of FDR boundaries on JPCP/JRCP

Large Area Removal and Replacement

In some situations, the existing distress is so extensive that the repair of every individual deteriorated area within a short distance (e.g., 10 to 30 ft) would be either very expensive, impractical, or both. In this case, it is more cost-effective and productive to replace the entire area of distressed panels instead of doing a series of individual repairs focused on specific distresses. Many agencies directly consider this in their governing specifications in that they have established FDR categories by size of repair.

Multiple-Lane Repairs

On multiple-lane roadways, deterioration may occur only in one lane or across two or more lanes. If distress exists in only one lane, it is not necessary to repair the adjacent lanes, but the placement of a compressible insert at the longitudinal joint interface between the adjacent existing slab and the new repair slab is recommended to accommodate differential movements. When two or more adjacent lanes contain distress and require repair, one lane is generally repaired at a time so that traffic flow can be maintained.

Matching transverse joints in adjacent lanes is generally not necessary, as long as fiberboard or a similar bond breaker has been placed along the longitudinal joint to separate the adjacent lanes and accommodate differential movements. If the distressed areas in both lanes are similar and both lanes are to be repaired at the same time, however, it may be desirable to align repair boundaries in order to avoid small offsets and to maintain continuity.

Continuously Reinforced Concrete Pavement

The types of CRCP distresses that can be addressed through FDRs are identified in the previous Table 6.2, which should be evaluated by each agency and modified for use under their local conditions. Punchouts, illustrated in Figure 6.3, are the major structural distress addressed by FDR in CRCP.

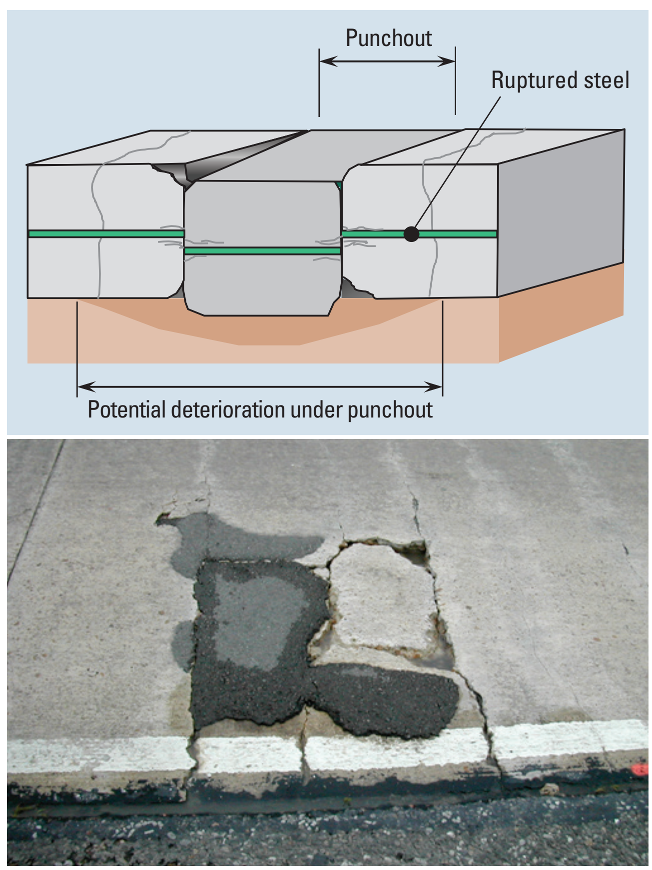

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Figure 6.3. CRCP punchout distress

Sizing the Repair

Deterioration of the base beneath a CRCP is likely occurring when a punchout appears or when there is settlement or faulting along the longitudinal lane joint. Field coring and deflection testing can provide information on the extent of deterioration that may have developed beneath the slab surface. In sizing the repair, it is important that all of the deterioration beneath the surface be included. General recommendations on CRCP repair dimensions include the following:

- A minimum repair length of 4 to 6 ft. TxDOT (2019) specifies a minimum length of 6 ft, while IDOT (2010) uses a minimum length of 4.5 ft.
- Repair boundaries should not be closer than 18 in. to adjacent nondeteriorated transverse cracks. This is to minimize the potential for additional deterioration outside of the repair area but may be a challenge where transverse cracks are closely spaced. In such cases, IDOT (2010) allows placement of the repair as close as 6 in. to an adjacent crack provided it is tight and not deteriorated.
- Both TxDOT (2019) and IDOT (2010) allow half-lane-width FDR of CRCP.

Figure 6.4 provides a recommended FDR layout in CRCP.

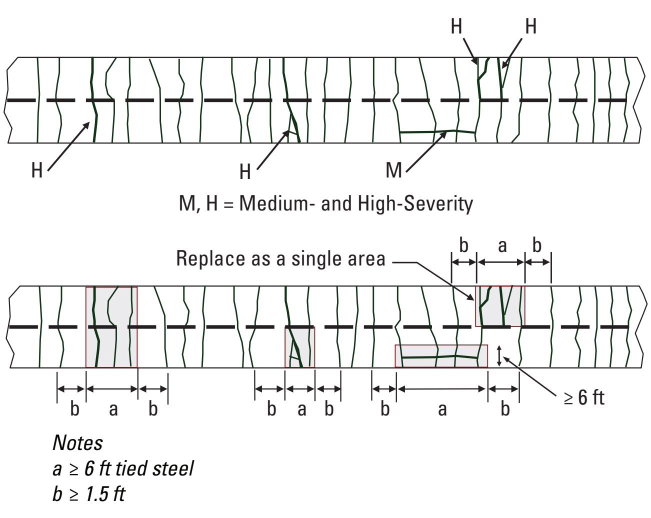

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Figure 6.4. Full-depth repair recommendations for a CRCP

Multiple-Lane Repair Considerations

If a distress such as a wide crack with ruptured steel occurs across all lanes of a CRCP, special FDR considerations are necessary because of the potential for significant movement of the CRCP once the repair area is opened up; this could lead to the following:

- Blowups in the adjacent lane
- Crushing of the new repair by expansion of CRCP during the first few hours of curing
- Cracking of the repair during the first night as the existing CRCP contracts

In order to minimize these problems, it may be necessary to place the concrete in the afternoon or evening to prevent the young repair from being crushed by expansion of the CRCP slab. In addition, it is recommended that the lane with the lowest truck traffic be repaired first to minimize early fatigue consumption on the heavier trafficked lane.

Selecting Repair Materials

The repair material for FDRs should be selected based on the available lane closure time. The current state of the art in concrete pavement repair is such that virtually any opening time requirement can be met (from 1 hour to 24 hours or more), using either conventional concrete or a proprietary repair material. A good rule of thumb in selecting the material for concrete pavement repair projects is to use the most conventional material that will meet the opening requirements. Faster-setting mixes often have special handling requirements and generally are more expensive; even so, their use can contribute to overall productivity and efficiency, meaning that the total costs for a project using such mixes may still be lower than if conventional mixtures were used.

Conventional PCC mixtures are widely used as repair materials for FDRs. Different constituent materials can be used in PCC FDRs to meet a range of opening times, as shown in Table 6.4. All proposed repair materials should be adequately tested and approved prior to use in an FDR project.

Table 6.4. Common ranges of constituent materials for high early-strength concrete

Mixture design parameter	4- to 6-hour concrete	6- to 8-hour concrete	20- to 24-hour concrete
Cement type	I/II or III or proprietary cement	I/II or III or proprietary cement	I/II or III
Cement content	650–895 lb/yd^3	715–885 lb/yd^3	675–800 lb/yd^3
w/cm ratio	0.34–0.40	0.36–0.40	0.40–0.43
Accelerator	Yes	Yes	None to yes

Sources: Compiled from Whiting et al. 1994, Van Dam et al. 2005, Sprinkel et al. 2019

Because these high early-strength mixes typically contain higher cement contents and multiple admixtures, however, it is not uncommon for them to experience increased shrinkage, altered microstructure, and unexpected interactions (Van Dam et al. 2005, Grove et al. 2009). For example, in Virginia, the use of high early-strength concrete with high cement contents (on the order of 800 lb/yd^3) caused excessive shrinkage cracking on many CRCP repairs, significantly reducing the life of the FDRs (Sprinkel et al. 2019). As a result, some agencies specify a maximum allowable shrinkage for their repair materials.

In addition, the long-term durability of these high early-strength mixtures is also potentially at risk. Guidelines are therefore available that summarize the state of the practice for high early-strength concrete repairs, including the identification of material properties that impact their performance, the selection of their constituent materials and mixture design properties, and the identification of relevant performance-related tests for both fresh and hardened concrete (Van Dam et al. 2005).

The proposed repair material for a project (using the same aggregates that will be used in the final mix) must be properly tested and approved to ensure that the desired strength requirements are met. In addition, consideration should be given to the durability requirements of the hardened concrete, which is largely a function of the exposure to freeze-thaw conditions and deicing chemicals, as well as the maximum coarse aggregate size. For most mixtures, air contents between 4.5% and 7.5% are recommended, with a maximum w/cm ratio of 0.45 (ACI 2016). The slump should be between 2 and 4 in. for overall placement, workability, and finishability.

In addition, there are a number of specialty cements and proprietary materials that have also been used successfully in FDRs. These materials are typically mixed on site using mobile mixers in quantities appropriate for the project.

One product that has seen widespread use in both PDR and FDR applications is CSA cement. CSA cement is a modified derivative of portland cement clinker and exhibits good durability and low shrinkage while gaining structural strength in approximately 1 hour at standard placement temperatures (Ramseyer and Perez 2009). The results from a recent evaluation of CSA FDRs of several California highways revealed excellent performance after 13 years of service (Ross 2019). California has used a CSA cement for highway panel replacement since 1994 (Ramseyer and Perez 2009).

In recognition of the need to target repair materials to opening times, Caltrans has developed the repair material matrix shown in Table 6.5 to meet a range of typical curing times (Caltrans 2015).

Table 6.5. Caltrans matrix of FDR materials based on available curing time

Typical curing time (hours)	Concrete mix type(s)
2–4	Specialty high early-strength cement mixes (including CSA). The cement may be portland, nonportland, or blended.
4–6	In addition to specialty cements, Type III portland cement with nonchloride accelerators and high-range water-reducing admixture may be used if shrinking and early-age cracking requirements are met.
<24	Type II portland cement with nonchloride accelerators
≥24	Conventional Type II portland cement*

** Preferred for lower cost and superior performance when sufficient strength can be attained before traffic opening*

Source: Caltrans 2015

Anticipated climatic conditions should also be considered when selecting a repair material. During hot, sunny, summer days, solar radiation can significantly raise the temperature at the slab surface, adding to the peak slab temperature and thermal gradient through the slab. When the ambient temperature is in excess of 90°F, it may be very difficult to place some of the rapid-setting materials because they set so quickly. Although a set retarder can be used with some of these materials to provide longer working times, a better solution may be to use a slower-setting mix. Temperature during installation and curing should also be closely monitored because adverse temperature conditions at the time of placement have been linked to premature failures (Yu et al. 2006, Darter 2017). Finally, water should not be added during times of elevated temperature as this could detract from the durability of the mixture.

Load Transfer Design in Jointed Plain Concrete Pavement/Jointed Reinforced Concrete Pavement

Transverse joint load transfer design is one of the most critical factors influencing the performance of FDRs. As defined in Chapter 3, load transfer is the ability to transmit wheel loads (and associated deflections, stresses, and strains) across a joint (or crack) in a concrete pavement. Poor load transfer allows differential movement between the slabs that can cause serious spalling, rocking, pumping, faulting, and even breakup of the adjacent slab or repair itself. In selecting a joint design for a particular FDR project, the performance of various joint designs under similar traffic levels within the agency should be used as a guide.

Because of their demonstrated effectiveness in providing improved performance (i.e., less faulting, rocking, and other joint-related distresses), the use of smooth dowel bars is highly recommended in the transverse joints of all FDRs. This is particularly important since the smooth faces of the repair joints in FDRs offer little aggregate interlock load transfer. The only exception to the recommended use of dowels in FDR may be on residential streets that are subjected to low truck-traffic levels (less than about 100 trucks or buses per day).

Table 6.6 summarizes recommended dowel-bar-related design details for different pavement thickness ranges (ACPA 2006).

As with new concrete pavement construction, dowel bars in FDRs are placed at mid-depth of the slab; furthermore, the use of larger, 1.5 in. diameter dowels is recommended for most FDRs installed on pavement facilities exposed to heavy truck traffic. Figure 6.5 shows dowel bars installed in the joint faces of an existing pavement on an FDR project.

Round, solid steel dowels conforming to AASHTO M 31 or ASTM A615 are commonly used for load transfer in concrete pavements. It is recommended that these dowel bars be coated for corrosion protection, which is generally accomplished through the application of a fusion-bonded epoxy coating under AASHTO M 284 (ASTM A775) or ASTM A934. Although the AASHTO M 254 specification requires coating thicknesses of 7 ± 2 mils, more recent recommendations call for an average epoxy coating thickness of 10 mils or more (Snyder 2011).

John Donahue, MoDOT, used with permission

Figure 6.5. Dowels installed in existing slabs of an FDR area

Table 6.6. Dowel requirements for FDRs in JPCP/JRCP

Pavement thickness (in.)	Vertical location in slab (in.)	Dowel diameter (in.)	Min. length (in.)	Spacing (in.)
≤6	Mid-depth (D/2)	0.75	14	12
6.5–8	Mid-depth (D/2)	1.0	14	12
8–9.5	Mid-depth (D/2)	1.25	14	12
10+	Mid-depth (D/2)	1.5	14	12

Source: ACPA 2006

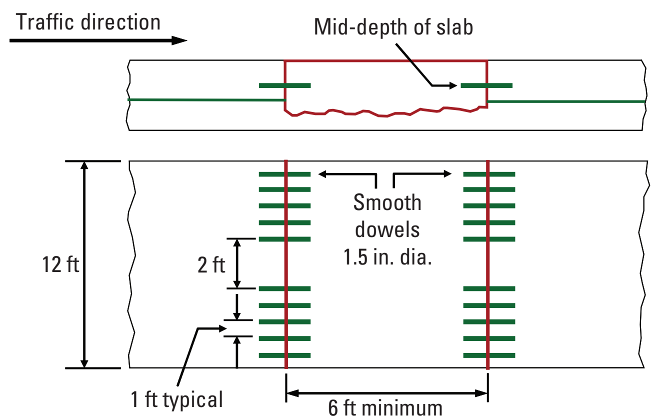
Table 6.7. FDR doweling practices of selected state transportation departments

State	Slab thickness (in.)	Location of dowels	Number of dowels across entire transverse joint (per lane)	Dowel diameter (in.)
California	≤9	Wheel paths	8 @ 12 in. spacing*	1.25
California	>9	Wheel paths	8 @ 12 in. spacing*	1.50
Georgia	<10	Uniform	11 @ 16 in. spacing	1.25
Georgia	>10	Uniform	11 @ 16 in. spacing	1.50
Minnesota	All	Uniform	11 @ 12 in. spacing	1.25
Missouri	All	Wheel paths	10 @ 12 in. spacing	1.0
Utah	All	Wheel paths	8 @ 12 in. spacing	1.50
Washington	All	Uniform	11 @ 12 in. spacing	1.50

**Use for truck lanes. Nontruck lanes use three dowels per wheel path.*

Source: Darter 2017

State transportation department practices vary in the number of dowel bars included in FDR designs. Some specifications require three, four, or five dowels per wheel path, whereas others require dowels across the entire lane width. Table 6.7 presents practices from some selected state transportation departments, with one layout depicted in Figure 6.6 (five dowel bars clustered in the wheel path).



Adapted from Smith et al. 2014, CP Tech Center

Figure 6.6. Example of dowel bar layout for FDR

Longitudinal Joint Considerations

When the repair length is less than 15 ft, a bond breaker board is typically placed along the length of the longitudinal joint to isolate the FDR from the adjacent slab and allow independent movement. The bond breaker, commonly a 0.2 in. thick fiberboard or similar material, should be configured for the full length of the repair and extend to the top height of the slab (see Figure 6.7).

At least one agency requires the placement of plastic sheeting on all faces of the existing concrete slabs (longitudinal and transverse) prior to repair placement in order to minimize friction (Darter 2017).

Tie bars are generally not required for isolated FDRs when they are less than 15 ft long. When the FDR is longer than 15 ft, tie bars are recommended for installation in the face of the adjacent slab at the longitudinal joint (ACPA 2006) and at the lane-shoulder joint if the existing shoulder is concrete. The tie bars should be epoxy coated and may range in size from a No. 4 (0.5 in. diameter) to a No. 6 (0.75 in. diameter) bar; they should be installed at mid-depth of the pavement slab and are commonly placed at 30 to 36 in. spacings.

IGGA, used with permission

Figure 6.7. Bond breaker along longitudinal lane-lane joint

Restoring Reinforcing Steel in Continuously Reinforced Concrete Pavement

As previously mentioned, it is important for CRCP designs to maintain the continuity of reinforcement through the FDR. The new reinforcing steel installed in the repair area should match the original in grade, quality, and number and is commonly affixed to the existing reinforcing steel using tie wires. Commonly up to 24 in. of existing steel must be exposed and the tied splices should be overlapped 16 to 20 in. depending on the bar diameter (Gulden 2013, Roesler et al. 2016).

In placing the bars, chairs or other means of support should be provided to prevent the steel from being permanently bent down during placement of the concrete, as the unsupported bar length should not exceed 4 ft. A 2 in. clearance is required between the end of the lap and the existing pavement, and a minimum of a 3 in. cover should be provided over the reinforcing steel. Figure 6.8 summarizes the sawing and repair details for CRCP repairs.

In addition to longitudinal steel, a few agencies (including Texas and Illinois) also place transverse steel in the repair to guard against longitudinal cracking and punchouts and to support the longitudinal steel (Gulden 2013). These transverse bars are tied to the longitudinal bars and are typically spaced 12 in. apart. It is recommended that all steel has a minimum cover of 3 in. and, as with FDRs on jointed pavements, FDRs on CRCP should be tied to the slabs in the adjacent lane when the repair length exceeds about 15 ft.

Several state transportation departments have adopted alternative approaches to the conventional practice of carrying the steel through the repair area for CRCP FDRs. These approaches include the following:

- TxDOT uses a single full-depth sawcut and then installs tie bars in drilled holes in both exposed transverse joint faces in the existing CRCP slab (Ryu et al. 2012). The epoxy-grouted tie bars are then connected to new longitudinal bars that are carried through the repair area (same size and spacing), as shown in Figure 6.9.

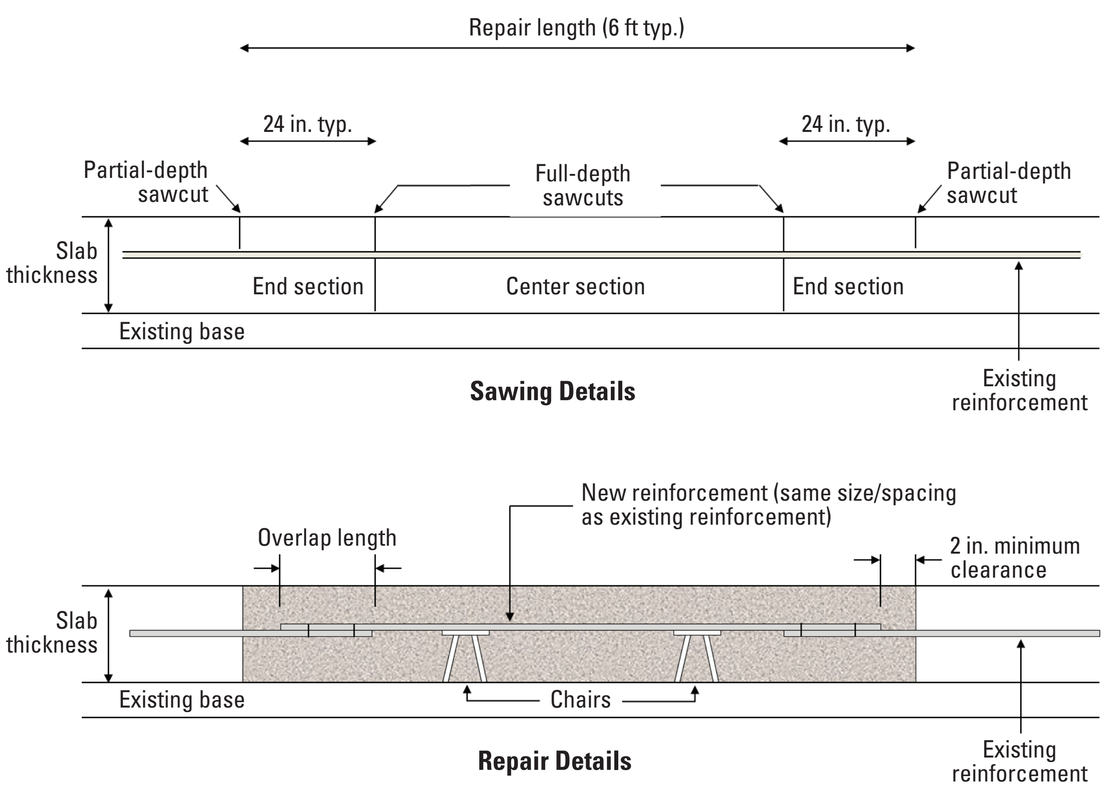


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Figure 6.8. Summary of sawing and repair details for conventional CRCP FDRs

This reduces the need for two sawcuts at each end of the repair, allows for the restoration of the base within the repair area, and significantly reduces the labor requirements and overall installation time (Tayabji 2011). This is important because closure windows for many CRCP repair projects are often limited to 8 to 10 hours, and this expedited procedure helps increase productivity. Experience has shown that the proper anchoring and installation of the tie bars are critical to the performance of these types of repairs, largely because these tie bars are relied upon to provide the load transfer at the joints (Ryu et al. 2012).

ACPA, used with permission

Figure 6.9. TxDOT method of CRCP repair

- The South Carolina Department of Transportation (SCDOT) employs doweled JPCP FDRs for addressing deterioration that is located in a single lane of a CRCP highway. In this methodology, epoxy-coated dowel bars are grouted into the existing transverse joint faces of the CRCP slab (much like conventional JPCP/JRCP repairs), with no attempts to maintain the continuity of the longitudinal steel (Tayabji 2011). These FDRs are working well in a number of projects in South Carolina. Details of the SCDOT method for panel lengths of 1.8 to 3.6 m (6 to 12 ft) are shown in Figure 6.10.

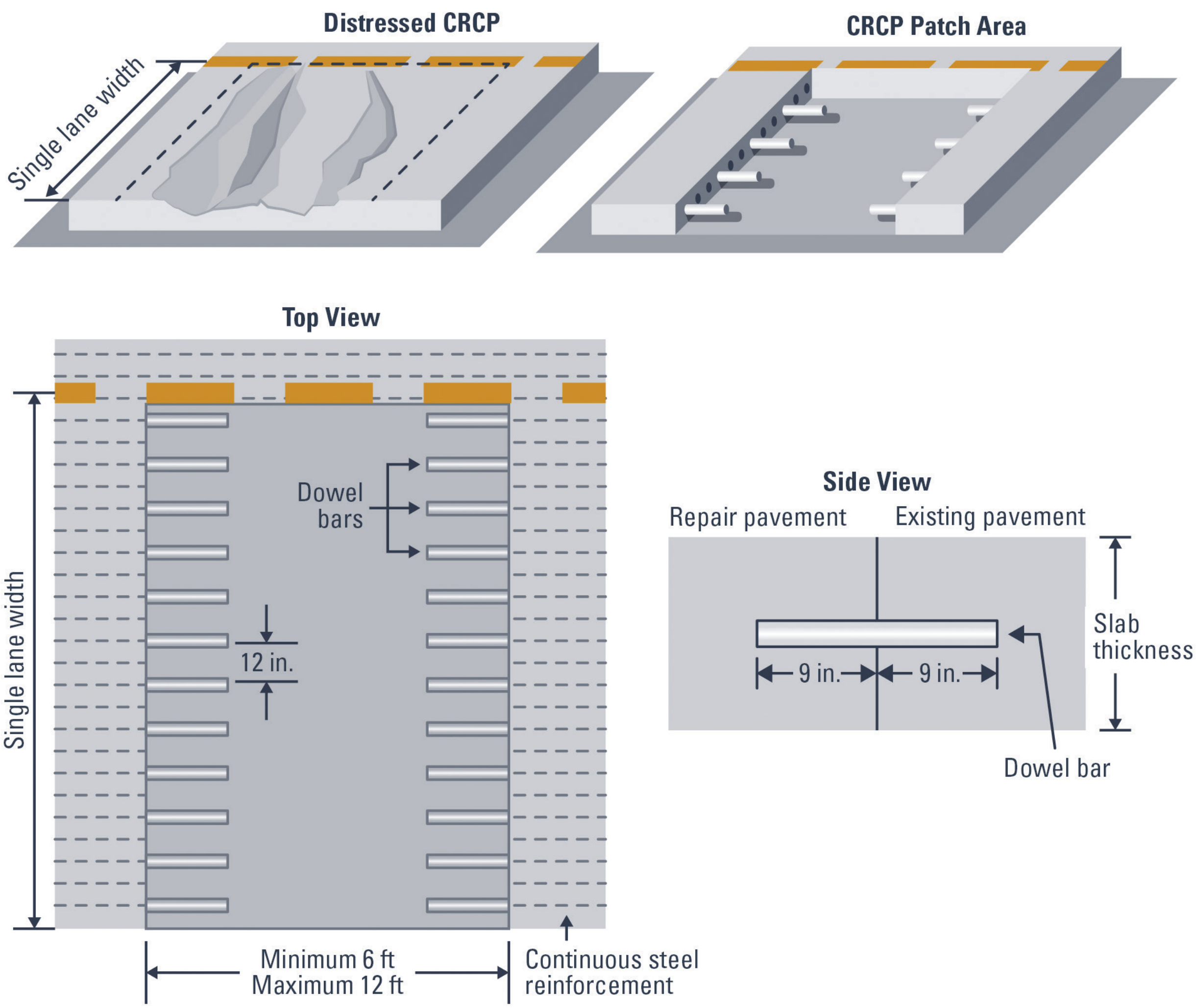

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Figure 6.10. Details of the South Carolina jointed FDR of CRCP

- The Illinois Tollway has developed a generic CRCP patching system featuring precast concrete for rapid overnight replacements (Gillen et al. 2018). The system employs a precast panel that has been cast to the same nominal thickness and the same nominal steel content as the existing pavement, but the key to the system is the creation of discrete "splice zones" between the existing pavement and the precast repair panel that allow for the continuity of the steel to be maintained; these splice zones are full lane width and 18 in. long (see Figure 6.11) and are later backfilled with high early-strength concrete.

Gillen et al. 2018, FHWA

Figure 6.11. Splice zone between new precast panel and existing CRCP

Opening to Traffic

There is not a clear consensus on what strength is required for opening concrete repairs to traffic. Factors such as the thickness of the repair, slab dimensions, expected traffic loadings, and expected edge loading conditions may all affect the minimum strength needed to ensure good performance.

A review of state highway practices suggests a range of values are specified for the opening of FDRs (see Table 6.8). Typical compressive strength values range from 2,000 to 3,000 lbf/in², while flexural strength values (third-point loading) range from 350 to 500 lbf/in².

Based on a review of state specifications and rehabilitation policies, Collier et al. (2018) recommended a compressive strength of 2,000 lbf/in² for the opening of FDRs. However, this opening strength value is considered to be conservative for the following reasons:

- Many state specifications and policies are based on the design strength of a repair to carry the traffic loadings expected over the entire life of the pavement and not necessarily on the minimum strength needed by the repair to carry immediate traffic (Grove et al. 2009).
- The concrete continues to gain strength over time (Khazanovich 2021, NCC 2021).

Consequently, lower strength values may be acceptable for early opening of FDRs, and this is supported by some research. For example, an accelerated load testing study of 9 in. thick slabs on a stiff base course supports the use of compressive strength values in the range of 1,600 lbf/in² for the opening of conventional concrete pavements (Tia and Kumara 2005). In addition, a study of early-age concrete pavement loading was recently conducted at MnROAD, in which truck loadings were applied to pavements at very low flexural strength levels, one as low as 73 lbf/in² (Khazanovich et al. 2021). The pavements were evaluated during, immediately after, and four years after the early loading, with the results indicating no long-term damage had developed (Khazanovich et al. 2021).

Table 6.8. FDR opening strength criteria and mix design parameters for selected agencies

State/ region	Early-opening-to-traffic (EOT) mix	Opening criteria	Common cement type*	Cement factor (lb/yd^3)	w/cm ratio	Air content (%)
AL	Class A slab replacement (VES)	6 hours (4,000 lbf/in^2, compressive @ 28 days)	Type III or Type I w/ nonchloride	NS	0.45	2.5–6
AZ	Class S accelerated strength patches (VES)	2,000 lbf/in^2, compressive @ 6 hours	Type III	520–752	NS	<7
CA	Rapid strength concrete	400 lbf/in^2, modulus of rupture	Specialty cements or Type II or III	505–675	NS	4–6
CO	Class E	3,000 lbf/in^2, compressive	Type III or HE	700	0.44	4–8
DC	Class C (HES)	NS (3,000 lbf/in^2, compressive @ 24 hours)	Type III	800	0.38	5–8
FL	Patching	1,600 lbf/in^2, compressive	NS	NS	NS	<6
GA	Class HES accelerated strength (HES)	2,500 lbf/in^2, compressive @ 24 hours	Type I or III	752	0.45	3–6
GA	Class HES (HES)	3,000 lbf/in^2, compressive @ 72 hours	NS	657	0.47	4–5.5
IA	Class M 5/10 hour (VES)	5 or 10 hours (500 lbf/in^2, flexural @ 48 hours)	Type I, II, or IS	[Volumetric] 0.15–0.16	0.328	8
IL	Class PP-1 patch (HES)	3,200 lbf/in^2, compressive @ 48 hours	NS	650–750 620–720 (Type III)	0.32–0.44	4–7
IL	Class PP-2 patch (HES)	1,600 lbf/in^2, compressive (3,200 @ 24 hours)	NS	735–820	0.32–0.38	4–6
IL	Class PP-3 patch (HES)	1,600 lbf/in^2, compressive (3,200 @ 16 hours)	Type I, II, or III with slag and microsilica	735	0.32–0.35	4–6
IL	Class PP-4 patch (VES)	1,600 lbf/in^2, compressive (3,200 @ 8 hours)	Calcium aluminate cement (type I)	600–625	0.32–0.50	4–6
IL	Class PP-5 patch (VES)	1,600 lbf/in^2, compressive (3,200 @ 4 hours)	No materials approved at this time	675	0.32–0.40	4–6
IN	High early (HES)	550 lbf/in^2, flexural @ 48 hours	Type I or III	564	0.42 (Type I) 0.45 (Type III)	5–8
KS	High early (HES)	450 lbf/in^2, flexural (2,400 lbf/in^2, compressive @ 24 hours)	Type III	NS	NS	6.5
MD	Mix 9 (HES)	3,000 lbf/in^2, compressive @ 24 hours	Type I or II	800	0.45	5–8
MD	Mix HE (VES)	2,500 lbf/in^2, compressive @ 6 hours	Type I or II	NS	NS	5–8
MN	Grade A mix 3A21HE (HES)	3,000 lbf/in^2, compressive @ 48 hours	Type I or I/II	615–750	0.40 (fly ash) 0.42 (slag)	7
MN	Grade A mix 3A41HE (HES)	3,000 lbf/in^2, compressive @ 48 hours	Type I or I/II	615–750	0.40 (fly ash) 0.42 (slag)	7
MO	FD concrete repair	2,000 lbf/in^2, compressive	Type III	560	0.50	>4
NC	Very HES repair (VES)	400 lbf/in^2, flexural @ 4 hours	Type III	526	0.559	3.5–6.5
NJ	Class V (VES)	350 lbf/in^2, flexural @ 6.5 hours	Type III	611	0.40	NS
NJ	Class E (HES)	3,000 lbf/in^2, compressive @ 72 hours	Type I or II	611	0.40	NS
NY	Class F (HES)	2,500 lbf/in^2, compressive (to open to construction traffic) 3,000 lbf/in^2, compressive (to open to general traffic)	Type I, II, or I/II	716	0.44	6.5
OH	Class QC MS RRCM (HES)	400 lbf/in^2, modulus of rupture @ 24 hours	Type III	800	0.50	4–8
OH	Class QC FS RRCM (VES)	400 lbf/in^2, modulus of rupture @ 4 hours	Type III	900	0.50	4–8
PA	High early (HES)	3,000 lbf/in^2, compressive @ 72 hours	Not Type III	752–846	0.40	6
PA	Accelerated strength patches (VES)	1,200 lbf/in^2, compressive @ opening, 1,450 lbf/in^2, compressive @ 7 hours	NS	587.5–752	0.47	6
St. Louis, MO	PCCPVES (VES)	3,500 lbf/in^2, compressive @ 4 hours	Type I/II, IL, or III	Type I/II: 850 Type III: 650	0.43	5.5
St. Louis, MO	PCCPHES (HES)	3,500 lbf/in^2, compressive @ 24 hours	Type I/II	700	0.43	5.5
TX	Class HES (HES)	255 lbf/in^2, flexural or 1,800 lbf/in^2, compressive @ <72 hours	Type I, II, III, I/II, or IL	NS	0.45	No limit
TX	Half-depth (HES)	255 lbf/in^2, flexural or 1,800 lbf/in^2, compressive @ <72 hours	Type I, II, III, I/II, or IL	NS	0.45	No limit
VA	Class A4 (HES)	2,500 lbf/in^2, compressive	Type I, II, or III	<800	0.45	6.5
WI	Special HES (VES)	3,000 lbf/in^2, compressive @ 8 hours	Type III	>846	0.42	6

** Many agencies also allow various specialty cements (e.g., calcium sulfoaluminate) or rapid-setting proprietary products for repairs.*

VES = very early strength, HES = high early strength, NS = not specified.

Source: Applied Pavement Technology, Inc.

It should also be noted that, in addition to its potential for inducing slab cracking, the early trafficking of doweled pavements can result in significant dowel bar bearing stresses, which can lead to "socketing" of the dowel bar and ultimately poor load transfer performance (Okamoto et al. 1994). Therefore, Whiting et al. (1994) recommended the use of the following compressive strength criteria in addition to the typical flexural strength requirements on fast-track projects to avoid the crushing of the concrete around the dowels:

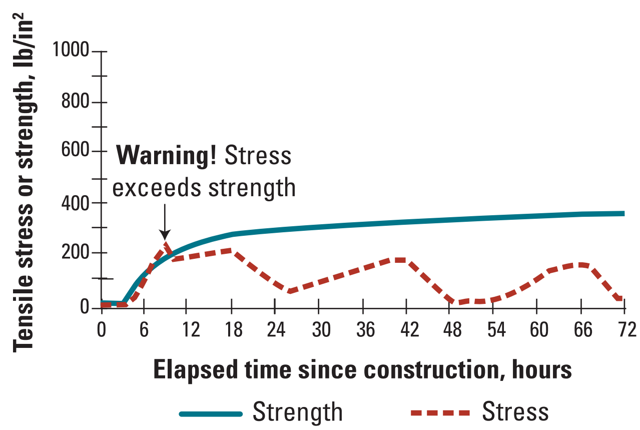
- 2,000 lbf/in² for concrete pavement slabs containing 1.5 in. diameter dowel bars
- 2,500 lbf/in² for concrete pavement slabs containing 1.25 in. diameter dowel bars

The characteristics of the FDR (e.g., slab thickness, slab size) are a final consideration in the selection of appropriate opening strengths. Thicker and shorter repairs exhibit lower critical stresses than thinner and longer repairs, allowing them to be opened to traffic sooner. This is reflected in the opening requirements suggested in Table 6.9 (ACPA 2006), and the abovementioned dowel-bearing stress criteria should be taken into account, with the highest value being the controlling factor.

A recent publication on concrete pavement opening requirements offers the following general recommendations (Delatte 2021):

- When developing high early-strength concrete mixtures, low heat of hydration, low shrinkage, and durability are more important than excessively high strength. Avoid high cement contents and Type III cement.
- Agencies should use conventional paving mixtures whenever extended curing times are available and opening strengths can be achieved (e.g., weekend closures).
- Agencies should consider the use of maturity, ultrasonic pulse velocity/impact-echo, and other NDT technologies in the assessment of the in-place concrete rather than relying on a fixed curing time for opening pavements to traffic.
- Slight increases in the thickness of the FDR can reduce the required opening strength.
- Maturity curves may also be used for project planning to select either conventional or high early-strength concrete based on the available time and the anticipated temperature conditions.

To assist in identifying those conditions that may contribute to the early cracking of FDRs, FHWA's HIPERPAV computer software program can be used. The program considers key environmental, structural design, mix design, and construction inputs and generates a graph showing the development of concrete strength gain and stress development over the first 72 hours after placement (see Figure 6.12).



Taylor et al. 2019, CP Tech Center

Figure 6.12. Example output plot from HIPERPAV showing possible risk of pavement cracking between about 6 and 10 hours after placement

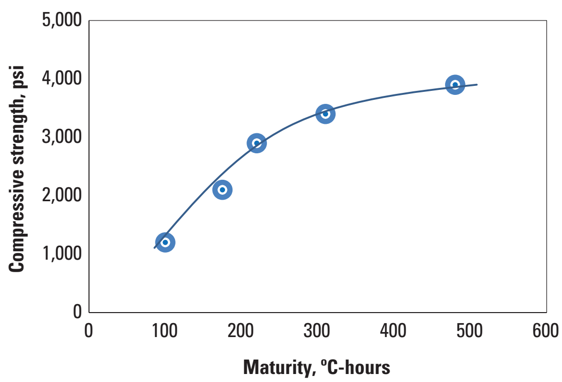
Table 6.9. Suggested minimum opening strengths for FDRs

Slab thickness (in.)	Compressive strength for repairs <10 ft (lbf/in²)	Third-point flexural strength for repairs <10 ft (lbf/in²)	Compressive strength for slab replacements (lbf/in²)	Third-point flexural strength for slab replacements (lbf/in²)
6.0	3,000	490	3,600	540
7.0	2,400	370	2,700	410
8.0	2,150	340	2,150	340
9.0	2,000	275	2,000	300
10.0+	2,000	250	2,000	300

Source: ACPA 2006

Maturity Methods

Maturity methods, conducted in accordance with ASTM C1074, can be used to assess the strength of in-place concrete pavements, including FDRs. Maturity methods account for time and temperature effects on concrete strength gain development and require that the time-temperature versus strength relationship be determined in the laboratory for the specific mixture used in the construction project. The resultant curve can then be used to estimate the time at which the desired concrete strength is expected to be reached (see the example plot). Additional details on the maturity method are available from the ACPA (2015) and ACI (2019).



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If the stress exceeds the strength at any time, a high potential for uncontrolled cracking is indicated. For such cases, adjustments can be made to mix properties, curing practices, or the time of concrete placement to reduce the potential for cracking. The latest version of this software, HIPERPAV III, version 3.3, was released in 2015 (Ruiz et al. 2015).

Full-Depth Repair of Composite Pavements

Full-depth repairs may also be used to address deterioration in existing composite pavements (e.g., asphalt overlays of concrete pavements). Typically, these FDRs will be used to address severe reflection cracking and pavement bumps or heaves that are caused by significant deterioration in the underlying concrete.

The same general design factors and construction steps used in FDR of bare concrete are still valid, with the following special considerations:

- Some additional coring or subsurface investigations may be needed to assess the degree of underlying deterioration
- Examination of the underlying concrete pavement (thickness and condition) is necessary to assess its ability to accept dowel bars
- The repair material should be placed to the entire thickness of the pavement (asphalt and concrete) to eliminate a two-stage repair process with concrete and asphalt

5. Construction

The construction and installation of FDRs involves the following steps:

1. Concrete sawing of repair boundaries
2. Concrete removal
3. Repair area preparation
4. Restoration of load transfer in JPCP/JRCP or of reinforcing steel in CRCP
5. Treatment of longitudinal joints
6. Concrete placement and finishing
7. Curing
8. Diamond grinding (optional)
9. Joint sealing for JPCP/JRCP

Step 1: Concrete Sawing of Repair Boundaries

Contractors often stage the various FDR activities in order to optimize productivity. As such, they often will perform all sawcutting on a project prior to proceeding with the next steps. In this case, it is very important to limit the traffic loading between the time of sawing and concrete removal to avoid pumping and erosion beneath the slab. It is generally recommended that no more than two days of traffic be allowed over the sawed repair areas before removal procedures begin.

When an asphalt shoulder is present, it is necessary to remove a portion of the shoulder along the repair to provide space for the outside edge form. This also prevents excessive damage to the shoulder when the old concrete is removed. The shoulder could be patched with asphalt concrete after the FDR is placed or, in some cases, the agency may just butt the FDR up against the existing shoulder, filling the removed asphalt area with the concrete repair material.

Jointed Plain Concrete Pavement/Jointed Reinforced Concrete Pavement

With the repair boundaries previously identified, full-depth sawcuts should be made around the entire periphery of the planned repair area to isolate the deteriorated concrete for removal; this includes transversely across the pavement as well as along the longitudinal lane-lane joint (and the longitudinal lane-shoulder joint if a concrete shoulder exists).

On hot days, it may not be possible to make the transverse cuts without first making a pressure-relief cut within the interior portions of the repair boundaries. A carbide-tipped wheel saw may be used for this purpose, but the wheel saw should be limited in the amount of its intrusion into the adjacent lane because it can produce a ragged edge that could potentially lead to excessive spalling along the joint. Hence, if wheel sawcuts are made, diamond sawcuts must still be made just outside the wheel sawcuts. In addition, to prevent subbase damage, the wheel saw must not be allowed to penetrate into the subbase more than 0.5 in. As an alternative to making a relief cut, the sawing operation could be scheduled such that it is performed during the cooler parts of the day or at night. Figure 6.13 illustrates the recommended sawing patterns for jointed pavements.

For JRCP repairs, there is no need to expose the reinforcing steel in the existing pavement because the repairs do not need to be tied into the existing pavement. In fact, for most FDR of jointed pavements, there is no need to provide reinforcing steel at all within the repair. Reinforcing steel may only be required for odd-shaped slabs (say, at intersections) or within repairs that are more than 15 ft long, although it is generally preferred to install an intermediate joint any time the repair length exceeds 15 ft.

Continuously Reinforced Concrete Pavement

The sawcutting procedures for CRCP will depend upon the repair methodology employed. In the traditional method, two sets of sawcuts are required in order to provide a rough joint face at the repair boundaries and to maintain the continuity of reinforcement throughout the repair. This is accomplished by first making a partial-depth cut at each end of the repair area, as shown in Figure 6.14.

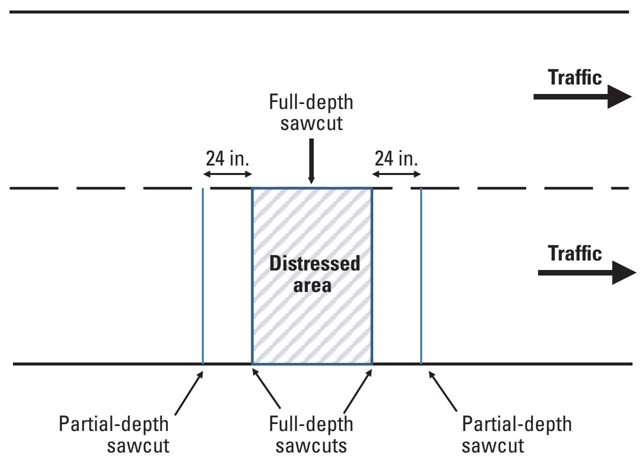

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Figure 6.14. Sawcut locations for FDR of CRCP

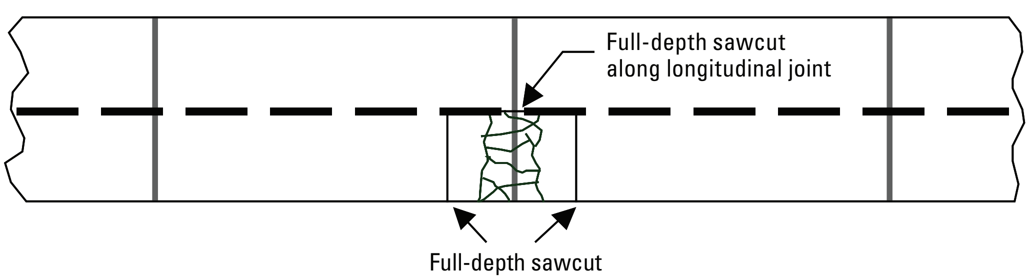

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Figure 6.13. Sawcut locations for FDR of JPCP/JRCP

This cut should be made to a depth of about one-fourth to one-third of the slab thickness and should be located at least 18 in. from the nearest tight transverse crack. The cut should not cross an existing crack, and adequate room should be left for the required lap distance and center area. If any of the steel reinforcement is cut, the length of the repair must be increased by the resulting additional lap length required. After the partial-depth cuts, two full-depth sawcuts are made within the repair area at a distance of 24 in. from the partial-depth cuts. To ensure good repair performance, the transverse joint faces must be rough and vertical (with all deteriorated material removed).

As previously mentioned, some agencies have used modified procedures in which a single full-depth sawcut in CRCP is employed and no efforts are made to tie in directly with the existing reinforcing steel. Instead, holes are drilled in the faces of the existing concrete slabs and new reinforcing steel (either tie bars in the case of the Texas method or dowel bars in the case of the South Carolina method) is anchored into the existing slab. The Texas method then ties new longitudinal steel to the tie bars and through the repair, while the South Carolina method adds no new steel. These procedures reduce the amount of hand chipping and thereby greatly increase productivity.

Step 2: Concrete Removal

Jointed Plain Concrete Pavement/Jointed Reinforced Concrete Pavement

Once the repair boundaries have been cut, the next step is to remove the deteriorated concrete from the repair area. This can be performed in two ways:

- **Lift-out method**—After the boundary cuts have been made, lift pins are placed in drilled holes in the distressed slab and hooked with chains to a front-end loader or other equipment capable of vertically lifting the distressed slab. The concrete is then lifted out in one or more pieces. Figure 6.15 shows various methods for the lift-out removal of distressed slabs.
- **Breakup and clean-out method**—After the boundary cuts have been made, the concrete to be removed is broken up using a jackhammer, drop hammer, or hydraulic ram, and then removed using a backhoe and hand tools. To prevent damage to adjacent concrete, large drop hammers should not be allowed, and large jackhammers must not be used near a sawed joint. Breakup should begin at the center of the repair area and not at the sawcuts.

Advantages and disadvantages of each removal method are listed in Table 6.10.

Table 6.10. Advantages and disadvantages of concrete removal methods

Method	Advantages	Disadvantages
Lift-out	This method generally does not disturb the subbase and does not damage the adjacent slab. It generally permits more rapid removal than the breakup and clean-out method on large FDR projects.	Disposal of large pieces of concrete may pose a problem. Large pieces must be lifted out with lifting pins and heavy lifting equipment or sawn into smaller pieces and lifted out with a front-end loader.
Breakup and clean-out	Pavement breakers can efficiently break up the concrete, and a backhoe equipped with a bucket including teeth can rapidly remove the broken concrete and load it onto trucks.	This method usually greatly disturbs the subbase and/or subgrade, requiring either replacement of subbase material or filling with concrete. It also has potential to damage the adjacent slab.

Source: Applied Pavement Technology, Inc.

John Donahue, MoDOT, used with permission (top) and ACPA, used with permission (bottom)

Figure 6.15. Lift-out method of slab removal

The lift-out method is generally recommended because it minimizes disturbance to the base (which is critical to FDR performance) and because of its high production rates, making it more suited for large projects. Depending on the condition of the distressed slab, however, it may require removal of several smaller slab segments if the entire slab cannot be lifted out. On smaller FDR projects, the breakup method is preferred.

Regardless of the method and equipment used, it is very important to avoid damaging the adjacent concrete slab and existing subbase. Steps should also be taken to avoid breakage of the concrete on the underside of the adjacent slab, which can lead to performance issues. If either surface spalling or underbreaking is observed, a new sawcut must be made outside of the damaged area.

Continuously Reinforced Concrete Pavement

The procedure for removing concrete from the center section of the CRCP repair area is the same as for JPCP/JRCP. In the traditional patching method, the challenging part lies in the removal of the concrete between the full-depth cuts and the partial-depth cuts as the goal is to expose the steel so that new steel can be lapped to carry it through the repair area. This can be accomplished using jackhammers, pry bars, picks, and other hand tools while being careful to avoid damage to the reinforcement. To prevent underbreaking of the bottom half of the slab, the face of the concrete below the partial-depth sawcut should be inclined slightly into the repair, as any significant underbreaking that occurs will require a new partial-depth sawcut outside the damaged area.

Separating the surrounding concrete from the reinforcing steel must be done without nicking, bending, or damaging the steel in any way. The use of a drop hammer or hydro-hammer should not be allowed in the lap area because this equipment can damage the reinforcement or cause spalling below the sawcut.

After the concrete has been removed, the reinforcement should be inspected for damage. Any bent bars must be carefully straightened. Bent reinforcement in the repair area will eventually result in spalling of the repair because of the large stresses carried by the reinforcement. If more than 10% of the bars are seriously damaged or corroded or if three or more adjacent bars are broken, the ends of the repair should be extended another lap distance. Figure 6.16 shows a CRCP repair with the reinforcing steel exposed.

Step 3: Repair Area Preparation

After the deteriorated concrete has been removed, the underlying base should be inspected for damage. Often in the case of a stabilized base, portions of the base can bond to the concrete that is removed and leave pockets of deterioration in the repair area. All subbase and subgrade materials that have been disturbed or that are loose should be removed and replaced either with a similar material or with concrete. However, because of the difficulty in adequately compacting granular material in a confined repair area, replacing the damaged portion of a disturbed base with concrete is often the best alternative. Before placing any new material, all excessive moisture present in the repair area, as determined by the project engineer, should be removed or dried out.

In the development of plans for a specific repair project, some nominal quantity estimate should be made for base/subbase repair so that a contingency item is not needed to be added later to the contract. Some agencies assume 10% of the total concrete repair area for their foundation layer repair estimate (Darter 2017).

Following any needed foundation repairs, a few agencies require the placement of bond breaker materials (typically plastic sheeting) on top of the base prior to the placement of the new concrete. For example, California specifies a bond breaker on top of a lean concrete base to allow the new concrete repair to move independently and prevent reflection cracking (Caltrans 2015). Similarly, Washington State requires the placement of a bond-breaking material on the existing base (as well as to line the existing concrete slab faces) to minimize friction.

Gulden 2013, FHWA

Figure 6.16. Prepared CRCP repair area with exposed reinforcing steel

Step 4: Restoration of Load Transfer in Jointed Plain Concrete Pavement/ Jointed Reinforced Concrete Pavement or Reinforcing Steel in Continuously Reinforced Concrete Pavement

Restoring Load Transfer in Jointed Plain Concrete Pavement/Jointed Reinforced Concrete Pavement

Smooth steel dowel bars are recommended for load transfer at both repair joints to allow uninhibited horizontal movement. The dowels are installed by drilling holes at mid-depth in the exposed faces of the existing slabs. Tractor-mounted gang drills can be used to drill several holes simultaneously while maintaining proper horizontal and vertical alignment at the same time, as shown in Figure 6.17.

Various types of gang drills are available, including self-propelled and mounted, which may take elevation references from either the slab (preferred) or the base course. Gang drills are now fitted with vacuum systems that capture the concrete dust produced by the drilling operation, in accordance with OSHA requirements.

The existing concrete joint face should be inspected prior to drilling to ensure that it is sound and is not exhibiting any signs of deterioration. Some pavements with large and particularly hard aggregates can tend to spall during drilling, making it difficult to properly anchor the dowel bars.

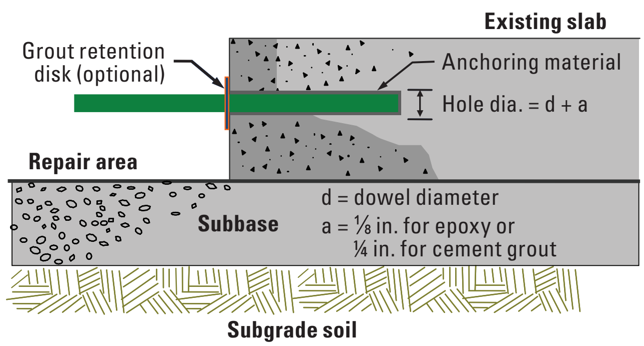
The dowel holes must be drilled slightly larger than the dowel diameter to allow room for the anchoring material. If a cement grout is used, the hole diameter should be 0.25 in. larger than the dowel diameter (ACPA 2006). A more viscous grout mixture provides better support for dowels than a very fluid mixture. If an epoxy mortar is used, smaller hole diameters (0.08 in. larger than the dowel diameter) are required because this type of material can often ooze out through small gaps and is somewhat more compliant than cement grout. Figure 6.18 illustrates a dowel bar anchoring installation.

Matt Zeller, Concrete Paving Association of Minnesota, used with permission

Figure 6.17. Example of a gang drill used for dowel bar installation

Proper anchoring of the dowels into the existing slab is a critical construction step. Studies have shown that poor dowel embedment procedures often result in poor performance of the repair because of spalling and faulting caused by movement of the dowels (Snyder et al. 1989, Darter 2017, Bruhn 2019). The dowel must therefore be fully encased in the anchoring material in order to prevent looseness and socketing of the dowel (see Figure 6.19) once the repair is opened to traffic; merely dipping the dowel in the anchoring material and placing it in the hole is not adequate and therefore not acceptable.

Dowel anchoring effectiveness can be verified by taking cores of the FDR above the encapsulated dowels.



Adapted from Snyder et al. 1989, FHWA

Figure 6.18. Dowel bar anchoring in existing slab

MnDOT, used with permission

Figure 6.19. Socketing around dowel bar on core taken from FDR

The following procedures are recommended for effectively anchoring dowel bars for FDRs (Snyder et al. 1989, ACPA 1995, Darter 2017):

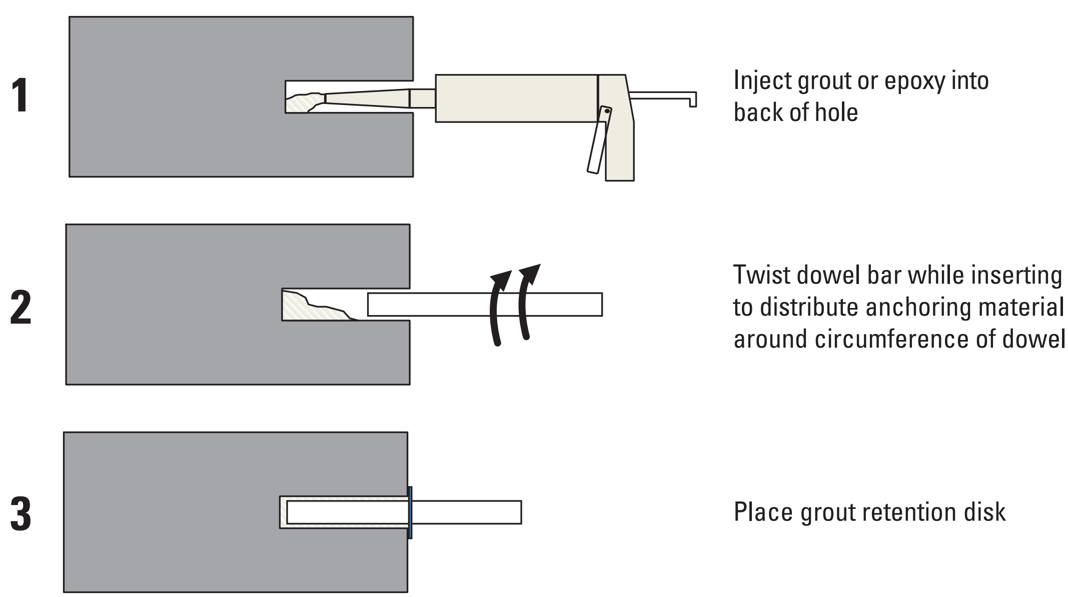
- Remove debris and dust from the dowel holes by blowing them out with compressed air. If the holes are wet, they should be allowed to dry before installing dowels. Check dowel holes for cleanliness before proceeding.
- Place quick-setting, nonshrinking cement grout or epoxy resin in the back of the dowel hole. Cement grout is placed by using a flexible tube with a long nose that places the material in the back of the hole. Epoxy-type materials are placed using a cartridge with a long nozzle that dispenses the material to the rear of the hole.
- Insert the dowel into the hole with a slight twisting motion so that the material in the back of the hole is forced up and around the dowel bar. This ensures a uniform coating of the anchoring material over the dowel bar.
- A number of agencies place a grout retention disk (a thin, donut-shaped plastic disk) over the dowel and against the slab face, as illustrated in Figure 6.20. The disk has an overall diameter of about 3 to 3.5 in. with the inner diameter sized to fit snugly over the dowel bar being used on the project. The grout retention disk prevents the anchoring material from flowing out of the hole and helps create an effective face at the entrance of the dowel hole (the location of the critical bearing stress).

IGGA, used with permission

Figure 6.20. Grout retention disk

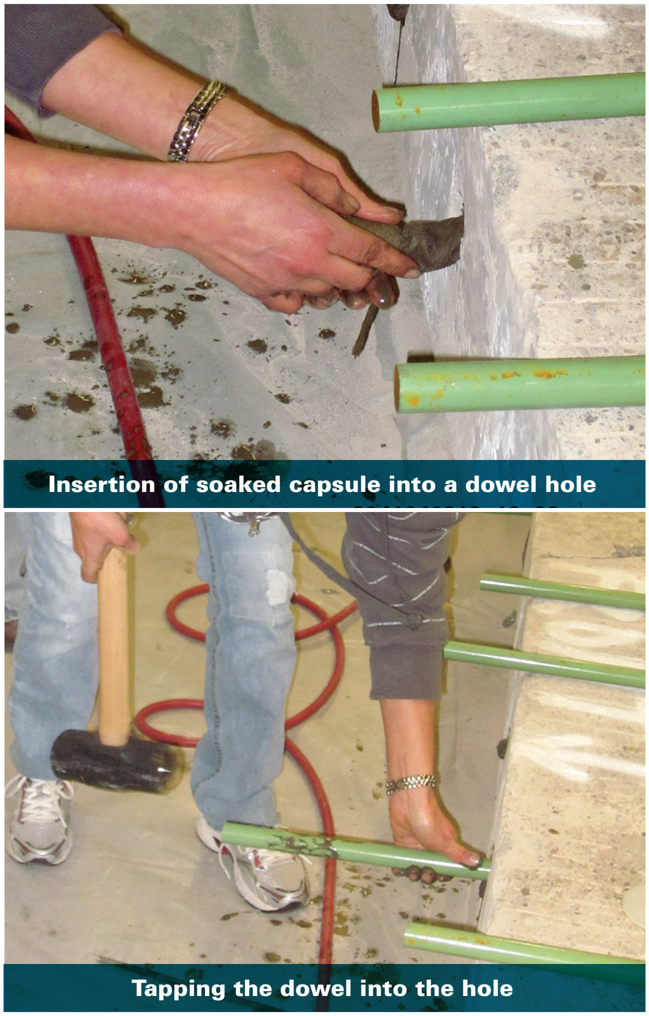
The dowel bar installation process is illustrated in Figure 6.21.

Some agencies have adopted other methods for anchoring dowel bars, including the use of grout bags or grout capsules that contain a cementitious, nonshrink grout material that is premixed dry and encapsulated in a water-permeable wrapping. Prior to installation, the grout capsule is saturated in water and then placed in the clean, dry dowel hole. After this, the dowel bar is inserted into the hole, which breaks the capsule and distributes the fast-setting grout material around the dowel bar (see Figure 6.22).



Adapted from ©ACPA 2006, used with permission

Figure 6.21. Dowel bar installation process



Photographs provided by MnDOT, used with permission

Figure 6.22. Installation of a grout capsule for dowel anchoring

Alternatively, the capsule can be first broken in two before soaking and then inserted into the hole as a way to help promote better distribution of the grout.

After anchoring of the dowels, a bond-breaking material such as a form or release oil should be lightly applied to the exposed ends of the dowel bars to facilitate horizontal movement. If steel reinforcement is to be provided within the repair (such as in a long repair or in an odd-shaped panel), the steel should be placed with a minimum of a 3 in. cover and 2.5 in. edge clearance.

Restoring Reinforcing Steel in Continuously Reinforced Concrete Pavement

As mentioned previously, most CRCP repair procedures require that the continuity of reinforcement be maintained through the repair. The splicing of the reinforcement bars should be conducted using the detailed design information presented previously. Most agencies also require the provision of transverse steel to help position the longitudinal bars and to control any potential longitudinal cracking. Figure 6.23 shows a CRCP repair with both longitudinal and transverse steel.

Step 5: Treatment of Longitudinal Joints

As described previously, a bond breaker board or the addition of tie bars may be required as dictated by the length of the repair. When the repair length is less than 15 ft, a bond breaker board is typically placed along the length of the longitudinal joint to isolate it from the adjacent slab (see Figure 6.24).

Generally, a fiberboard or similar material is used and configured to fit snugly within the repair area depth and length and to sit flush with the longitudinal face of the repair. For longer repair areas (typically more than about 15 ft), tie bars should be installed along the face of the adjacent slab using procedures similar to those used for installing dowel bars. The tie bars are typically spaced at 30 to 36 in. intervals.

Tayabji 2011, FHWA

Figure 6.23. CRCP repair with both longitudinal and transverse steel

ACPA, used with permission

Figure 6.24. Placing a bond breaker board along a longitudinal lane-lane joint

Step 6: Concrete Placement and Finishing

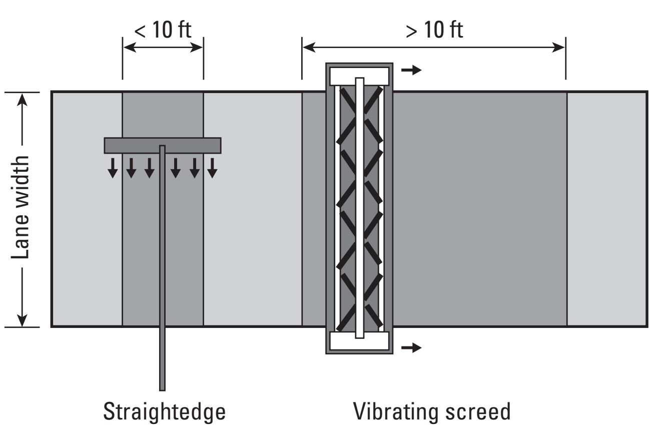
Critical aspects of concrete placement and finishing for FDRs include attaining adequate consolidation and a level finish with the surrounding concrete. Special attention should be given to ensure that the concrete is well vibrated around the edges of the repair and that it is not overfinished. Ambient temperatures should be between 40°F and 90°F for any concrete placement (ACPA 2006). The addition of extra water at the construction site for workability should not be allowed because this will decrease the concrete's strength and increase its shrinkage.

For repairs less than 10 ft long, the surface of the concrete should be struck off with a screed perpendicular to the centerline of the pavement, whereas repairs longer than 10 ft should be struck off with the screed parallel to the centerline of the pavement (see Figure 6.25).

The repair should be struck off two or three times to ensure that its surface is flush with the adjacent concrete. After placement, the surface should be textured to match, as much as possible, the texture of the surrounding concrete.

On longer repairs that require an intermediate joint, the timing of sawing is very important. Sawing too early can cause spalling along the sawcut or dislodging of aggregate particles, whereas sawing too late can lead to random cracking in the repair. Good practice dictates that the joints be sawed as soon as possible without causing significant raveling.

On some FDR projects, it may be necessary to restrict the time of concrete placement to late in the afternoon, depending on the climatic and pavement conditions.



Adapted from ©ACPA 2006, used with permission

Figure 6.25. Recommended finishing direction depending on the size of repair

In some cases where concrete has been placed in the morning, expansion of the adjacent slabs in the afternoon has resulted in crushing of the repair concrete. If significant signs of movement and other pressure-related damage are apparent within the project (bridge pushing, joint spalling, etc.) and are expected to continue, the use of an expansion joint at one or both ends of the repair may need to be considered. These joints should be kept to less than 1 in. and they must be doweled.

Step 7: Curing

Moisture retention and temperature management during the curing period are critical to the ultimate strength of the concrete. Proper curing is even more important when using set-accelerating admixtures or high early-strength mixtures. Therefore, as soon as the bleed water has disappeared from the surface of the concrete pavement (typically within about a half hour of concrete placement), the approved curing procedure should commence to prevent moisture loss from the pavement (ACPA 2006).

Typical curing methods include wet burlap, impervious paper, pigmented curing compounds, and polyethylene sheeting, with white-pigmented curing compounds (specified under AASHTO M 148) most commonly used at an application rate of 200 ft²/gal. However, the membrane-forming curing compounds that meet the requirements of AASHTO M 148 in fact exhibit variable capacities in reducing moisture loss, with some moisture loss occurring depending on both how the compound is applied and the ambient conditions (Van Dam 2018).

Work conducted by Hajibabaee et al. (2016) found that a solvent-based curing compound (i.e., poly-alpha-methylstyrene) was more effective than two water-based curing compounds (i.e., either a wax-based or resin-based compound) in providing water retention, possibly due to better surface wetting that produces fewer imperfections. At least one agency (MnDOT) therefore specifies the use of poly-alpha-methylstyrene for curing FDRs. Figure 6.26 shows an FDR that has received a curing compound, with more details on concrete curing provided by Van Dam 2018 and Taylor et al. 2019.

Matt Zeller, Concrete Paving Association of Minnesota, used with permission

Figure 6.26. Curing compound on installed FDR

On projects with very early opening time requirements (4 to 6 hours), it may be necessary to use insulation blankets to obtain the required strength within the available time. The insulation blankets promote rapid strength gain by keeping the internal temperature of the concrete high, thus accelerating the rate of hydration. Insulation blankets, however, are generally not needed on hot summer days and in those cases could actually lead to premature cracking. In cold weather, the insulation blanket should not be removed when there is a large difference between the concrete and air temperatures because the rapid cooling of the pavement surface following the removal of the insulation blanket can lead to cracking of the repair slabs.

As described earlier, the time when the repairs can be opened to traffic varies based on a number of factors, including slab thickness, slab dimensions, climatic conditions, curing regimen, and anticipated early traffic loadings, among other items. The strength values can be determined through cylinder/beam breaks of samples cast with the repairs, through maturity testing, or through the age-based characteristics of the specified materials.

Step 8: Diamond Grinding (Optional)

Diamond grinding is performed on many FDR projects, particularly when the number of repairs may affect overall ride quality. It is an effective wrap-up treatment after FDR construction to help restore overall smoothness and to blend the repairs in with the existing pavement. Chapter 9 provides more details on diamond grinding.

Step 9: Joint Sealing for Jointed Plain Concrete Pavement/Jointed Reinforced Concrete Pavement

Many agencies seal the transverse and longitudinal repair joints in accordance with their established sealing policy. This is intended to reduce spalling (by lowering the initial point-to-point contact between the existing slab and newly placed repair) and to minimize the infiltration of water and incompressibles. Chapter 10 provides more information on joint sealing.

6. Full-Depth Repair Using Precast Slabs

During the last decade, a number of agencies have implemented precast paving technologies for the repair, rehabilitation, and reconstruction of roadway pavements. This is an engineered system using prefabricated concrete panels that are fabricated or assembled at a plant under controlled conditions, transported to the project site, and then installed on a prepared foundation. The advantages of using precast pavement systems in the repair and rehabilitation of concrete pavements include the following (Smith and Snyder 2019, Tayabji 2019a):

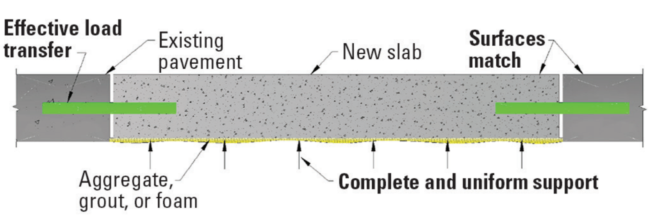
- **Better-quality concrete**—With precast panels, there are no issues related to the quality of the fresh concrete that is delivered to the project site nor are there concerns about the paving equipment operation or the uniform placement of the concrete.
- **Improved concrete curing conditions**—Curing of precast panels occurs under controlled conditions at the precast concrete plant.
- **Minimal weather restrictions on placement**—The construction season can be extended because precast panels can be placed in cooler weather or even during light rainfall.
- **Reduced delay before opening to traffic**—On-site curing of concrete is not required. As a result, precast panels can be installed during nighttime lane closures and be ready to be opened to traffic the following morning.
- **Elimination of construction-related early-age failures**—With precast panels, issues related to late or shallow sawing do not develop.

For all of these reasons, precast repairs offer an attractive alternative to cast-in-place repairs in situations where high traffic volumes and consideration of user delay costs favor more expeditious rehabilitation solutions (Tayabji and Hall 2010).

Approximately two-thirds of the precast concrete pavement projects constructed to date are classified as "intermittent" repairs, in which precast panels were placed as FDRs at isolated joints, cracks, or even as full slab replacements (Smith and Snyder 2019). On larger FDR projects where precast repairs are being used, the common practice is to establish a series of "standard" panel lengths (e.g., 6, 9, and 12 ft long) to accommodate the varying degrees of deterioration in the existing pavement; panel widths are typically for the full lane (Smith and Snyder 2019). In addition to the repair of standard segments of roadways, precast FDR can also be used in other applications, including for ramps, intersections, bridge approaches, toll plazas, and bus pads (Smith and Snyder 2019). A schematic of a precast FDR is shown in Figure 6.27.

California, Michigan, Minnesota, New Jersey, New York, Ontario, and the Illinois Tollway are just a few of the transportation departments that have used precast slabs in repair applications. Initial evaluations of some of these projects generally indicate that well-designed and well-installed precast repairs perform well and have the potential to provide long-term service (Tayabji et al. 2012, Tayabji 2019a).

Paralleling the requirements for effective, cast-in-place repairs, items of particular importance to the performance of precast slabs in repair applications include the provision of both adequate load transfer at the joints and good support under the repair (Tayabji et al. 2012, Tayabji 2019a).



Recreated from Smith and Snyder 2019, ©2019 National Precast Concrete Association, used with permission

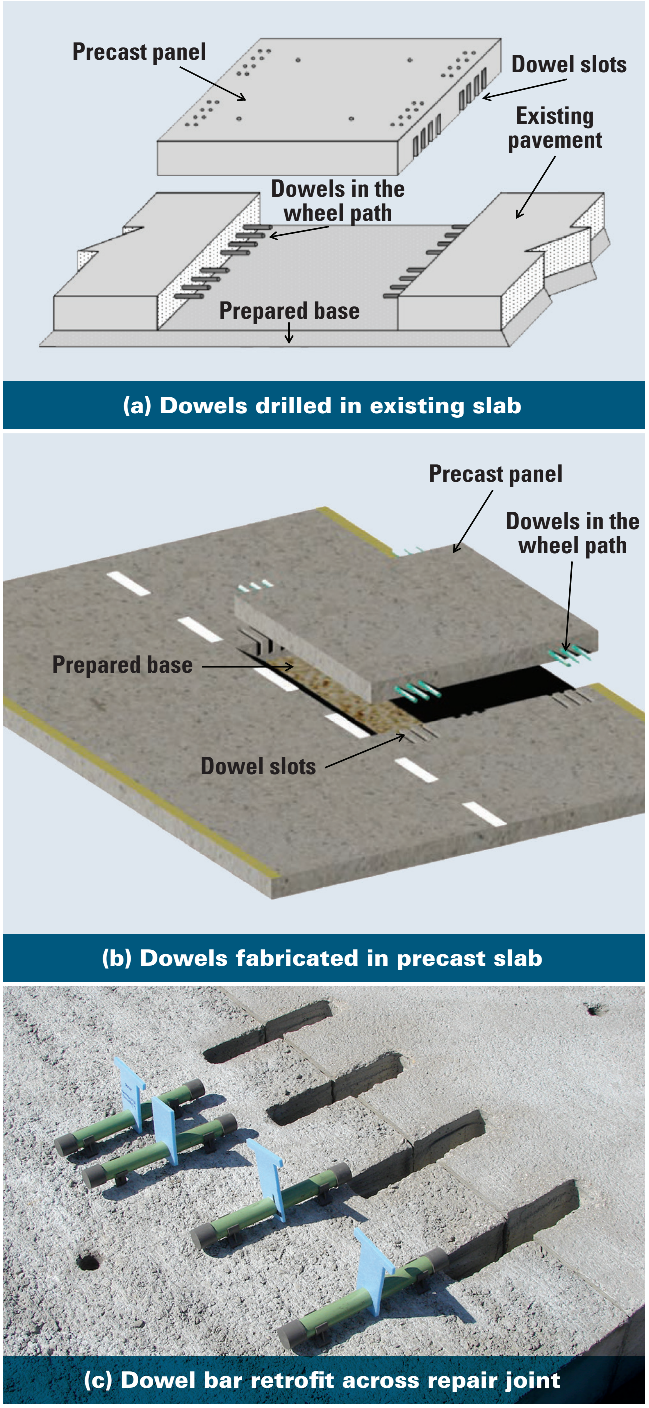
Figure 6.27. Design of precast FDR

Precast Systems

There are a number of different systems available for FDR using precast slabs (Tayabji et al. 2012). Each of these systems essentially shares the same components, consisting of the fabrication of the slab at an off-site precast plant, preparation of the repair area (including proper repair area sizing and preparation of the base), installation of (system-dependent) load transfer devices, panel placement, and (system-dependent) grout undersealing.

It is the load transfer provisions and panel support methodologies that are often what differentiates these various systems, with the most common load transfer and panel support methods described below:

- **Load transfer**—Effective load transfer must be provided at the joints between the existing pavement and the precast repair. Typically, this is accomplished via four dowel bars used in each wheel path installed through one of several methods available:
  - *Drilling and installing dowel bars in the existing pavement*—This is similar to what is done for conventional FDRs, but it requires precast slabs with slots at the bottom to accommodate the dowel bars (see Figure 6.28a). A porthole in the precast slab is then used to accommodate the pumping of grout into the dowel slots.
  - *Using dowel bars and slots at the surface*—In this method, conventional slots (typically 2.5 in. wide) are cut into the surface to accept dowel bars. Two primary techniques as well as several alternatives are available for providing the necessary load transfer (Tayabji et al. 2012):
    - *Partial dowel bar retrofit technique*—Dowel slots are cut into the existing pavement before panel placement in order to accommodate the dowel bars cast in the slab (see Figure 6.28b). The dowel slots are then patched using an approved patch material (similar to DBR).
    - *Full dowel bar retrofit technique*—After the precast panel has been placed in the repair area, slots are created on the surface of both the existing pavement and the precast repair slab. The dowel bars are then placed in the slots and patched, following DBR procedures (see Figure 6.28c).

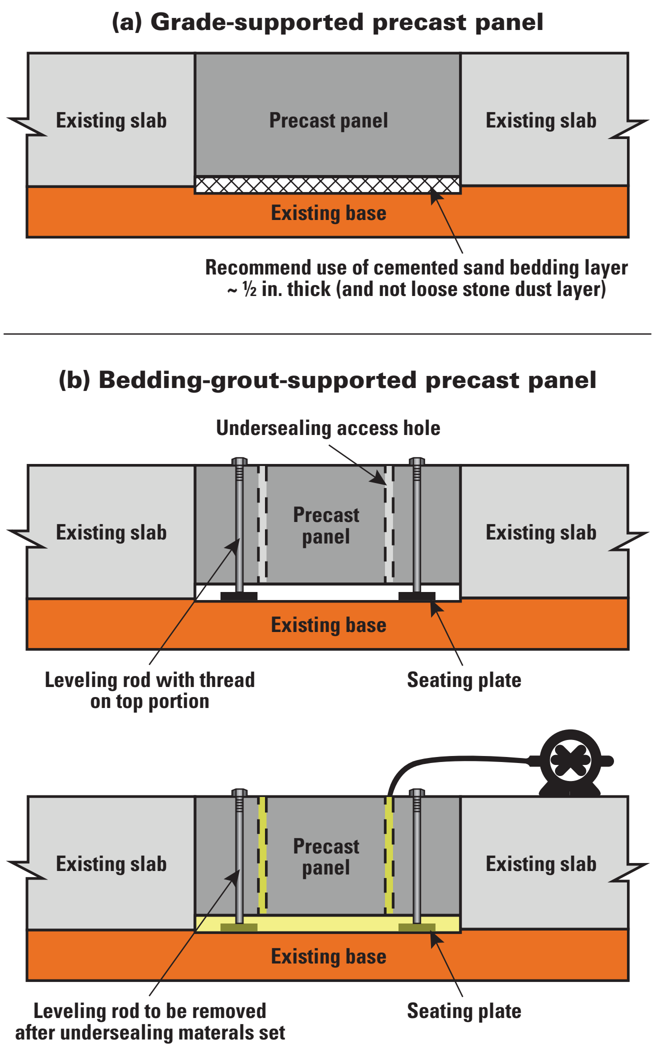


Adapted from Tayabji et al. 2012, SHRP 2 (top and center) and Shiraz Tayabji, used with permission (bottom)

Figure 6.28. Selected load transfer alternatives for precast FDR

- *Alternative techniques*—Several alternative systems are available for the provision of load transfer, featuring narrow-mouth slots at the surface, full-depth slots to accommodate dowels from adjacent slabs, or various duct/slot combinations. Additional details are available elsewhere (Smith and Snyder 2019, Tayabji 2020).

- **Panel support**—An interlayer (bedding layer) of material is needed between the base and the bottom of the precast panel to serve as grade control and to ensure the panel is fully supported. Because of time constraints, it is very unlikely that a new base will be installed for a precast repair. Therefore, the thickness of a precast repair panel nominally matches the thickness of the existing pavement but with some slight thickness reductions for the bedding and support materials (Smith and Snyder 2019). Two panel support methods are commonly used:
  - *Grade supported*—Panels are placed over a thin layer of cemented granular material or cemented sand (see Figure 6.29a). This bedding layer is about 0.5 in. thick and is placed over the graded and compacted base (Tayabji 2019a). Because this method provides little means for adjustment, surface grinding is almost always required. Subsealing is performed when using a cemented granular bedding layer to fill any voids that may exist in the panels (Tayabji 2019a).
  - *Bedding grout supported*—Panels are set about 0.25 to 0.5 in. over the completed base using leveling lifts; then a fast-setting flowable grout is used to fill the gap beneath the panels (Tayabji 2019a) (see Figure 6.29b). The grout is introduced through grout ports at the panel surface.



After and adapted from Tayabji 2019a, FHWA, used with permission

Figure 6.29. Panel support methodologies for precast FDR

General Construction Steps

Although different precast systems exist for the FDR of concrete pavements, each system follows the same general construction steps. These general steps, and some of the considerations associated with each, are summarized below (Tayabji et al. 2012, Smith and Snyder 2019):

1. **Panel installation staging and lane closures**—Access to the project site must be obtained in order to accommodate the entry and exit of material haul trucks, the positioning of construction-related equipment, and the delivery of the precast panels.

   Available construction windows will dictate the sequencing and other planning of the preinstallation and installation activities. The work zone will require closure of two lanes, the lane undergoing repairs and an additional lane for construction traffic (especially for the trucks delivering the precast panels). In some cases, two separate closures may be required, in which on the first day the existing concrete is removed and the new repair panel is placed and on the second day the remaining activities (such as load transfer provision and undersealing) are performed.

2. **Removal of distressed concrete sections**—As with conventional FDRs, the repair boundaries for FDRs using precast panels are identified and full-depth sawcuts are made to isolate the deteriorated concrete so that it may be removed. The repair should be the full lane width, and the sawcutting of the distressed area should be carried out as closely as possible to the installation time of the precast panels. Agencies often establish standard repair dimensions in order to facilitate the repair process, but it is critical to not have an excessively large repair area; ideally, joint widths along the repair perimeters should not exceed 0.38 to 0.5 in. The lift-out method of pavement removal is preferred, with care taken to avoid damage to the base and to the adjacent slabs that are to be left intact.

   In the event that the removal of the distressed concrete significantly spalls or otherwise damages the adjacent slabs, there are two options available: (1) cut back the deteriorated slab and repair the entire area with a rapid-set concrete for a permanent, cast-in-place solution or (2) put in a dummy slab as an interim solution until a new precast panel of the new dimensions can be cast (i.e., for a period of about 2 weeks).

3. **Bedding support provision**—Bedding support options for precast panels were discussed previously.

4. **Load transfer provision**—Load transfer options for precast panels were described previously. Commonly, four dowel bars are used in each wheel path for FDRs using precast panels. As with cast-in-place repairs, it is common to eliminate longitudinal lane tie bars and instead provide isolation material along the longitudinal joints for all intermittent repairs of 15 ft or less (Smith and Snyder 2019).

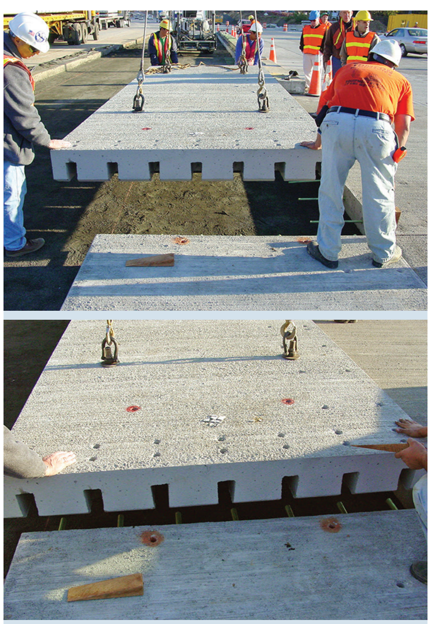
5. **Panel placement**—Once the base (or base and bedding) is prepared and set to the desired elevation using a template that matches the thickness of the precast panel, the panel installation process can begin. The panel installation requires the panel delivery trucks to be positioned in the adjacent lane next to the repair area. The panel is then handled by a crane and carefully lowered into position so that it is centrally located within the repair area such that the dowel bar and slot systems (if present) are aligned. Some systems use a slightly thinner slab that then requires the injection of a grout or polyurethane material beneath the slab to slightly raise it to the desired elevation.
6. **Post-panel installation activities**—Several post-panel-installation activities are required, depending on the type of precast repair system being used. This can include the grouting or patching of the dowel slots, the undersealing of the precast panel (which is required for all precast FDR systems to ensure that full support exists beneath the slab), surface grinding (as required by the system for rideability), and joint sealing. FDRs using precast panels typically can be opened to traffic after the dowel or bedding grouts have reached acceptable strength levels, although the system with the slots on the bottom can be opened to traffic prior to grouting as part of a staged operation.

Figure 6.30 shows FDR precast panel installation using three different DBR systems.

A series of checklists for precast jointed concrete pavement panel fabrication and installation is available from the FHWA (Tayabji 2019b).

It should be noted that careful planning and coordination is required for a precast patching project so that production rates can meet the installation demand. Typical fabrication rates within a plant are about 6 to 8 panels per day, while panel installation rates for repairs can be 15 to 20 panels per night for a 6- to 8-hour lane closure. As a result, several weeks of panels will usually need to be prepared and stockpiled in advance before installation can begin (Tayabji 2020).

The cost of precast slabs for repair activities has come down substantially during the last few years as the technology has evolved and contractors have become more experienced. Pricing in the early 2000s was about \$900/yd² installed, but 2020 costs ranged from about \$350 to \$450/yd² depending on the project size and other logistics (Tayabji 2020).



Dowel bars drilled in existing slab

Dowel bars fabricated in precast panel

Precast panel using full dowel bar retrofit installation

Photographs provided by Shiraz Tayabji, used with permission

Figure 6.30. Precast panel placement using different DBR systems

7. Full-Depth Repair of Utility Cuts

Different types of utilities (e.g., storm and sanitary sewers, water mains, and gas and power lines) must periodically be accessed for repair or maintenance. This requires cutting into the street to gain access, which can disrupt the uniformity of the pavement and compromise its overall structural integrity. For example, some common failure modes associated with utility repairs include (Suleiman et al. 2010, Iowa SUDAS 2021):

- Settlement of the utility cut restoration, caused by poor compaction of the trench backfill due to a combination of large lift thicknesses and the equipment used or by wet and/or frozen conditions
- A "bump" forming over the restoration, resulting from the uplift/heaving of the backfill soil caused by frost action, from the settlement of the surrounding soil, or from not properly finishing the surface of the utility cut flush with the surrounding pavement
- Deterioration adjacent to the repair caused by weakening of the materials surrounding the trench as a result of the stress-state change created by the excavation and the loss of lateral support/confinement

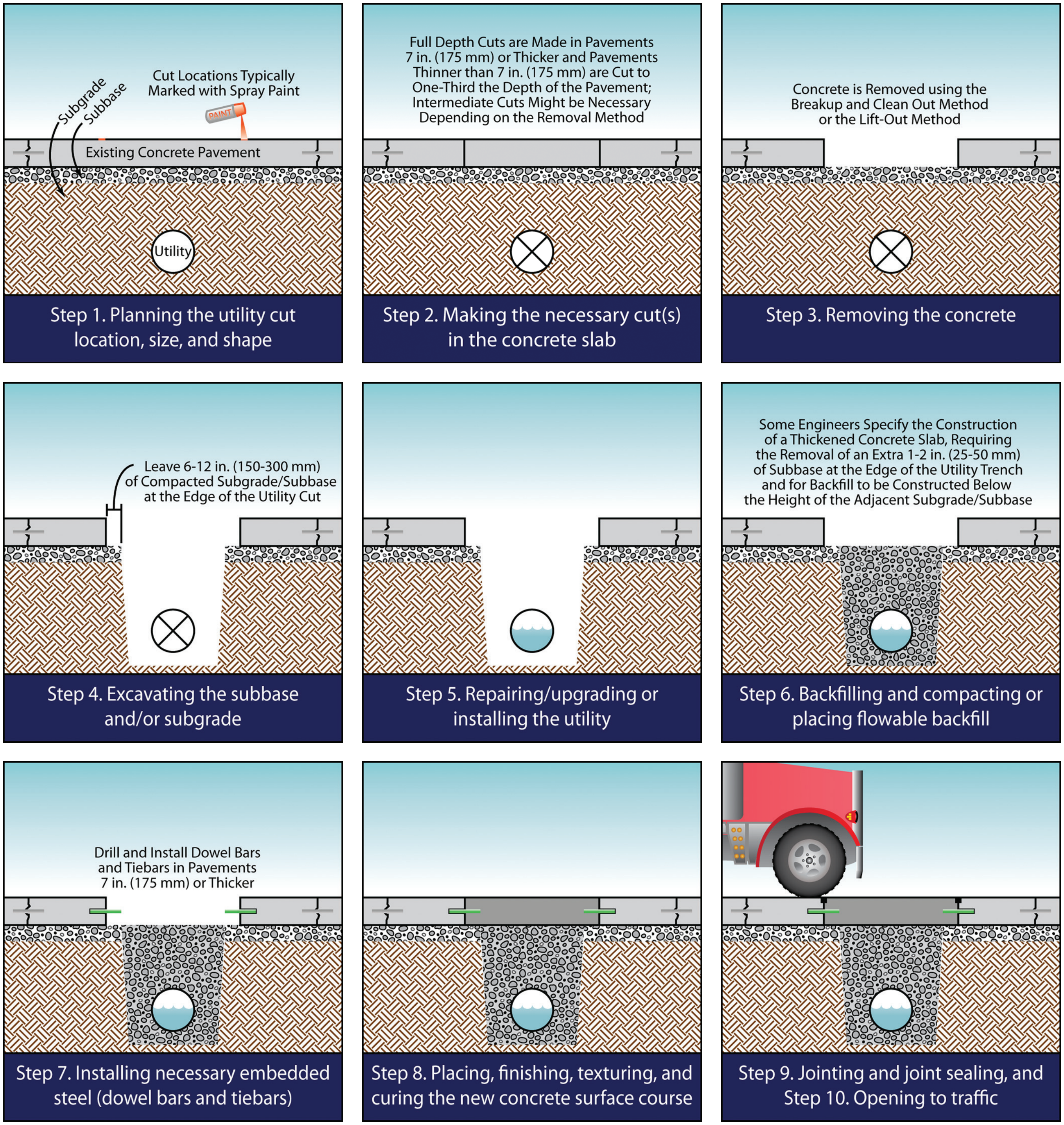
Therefore, it is imperative that effective utility cut repairs be installed in order to restore and maintain the structural and functional capacity of the pavement. This section briefly discusses the topic of utility cut restoration in concrete pavements, including the various steps associated with the process, the key factors governing success, and the recommended materials and procedures for performing utility cut restoration. The focus is on permanent, long-term repairs constructed using cast-in-place concrete although, as presented in the previous section, precast materials can also be used for utility cut repairs.

General Construction Steps

The recommended steps for utility cut repairs in concrete pavements are summarized in Figure 6.31 with additional details highlighted below (ACPA 2014):

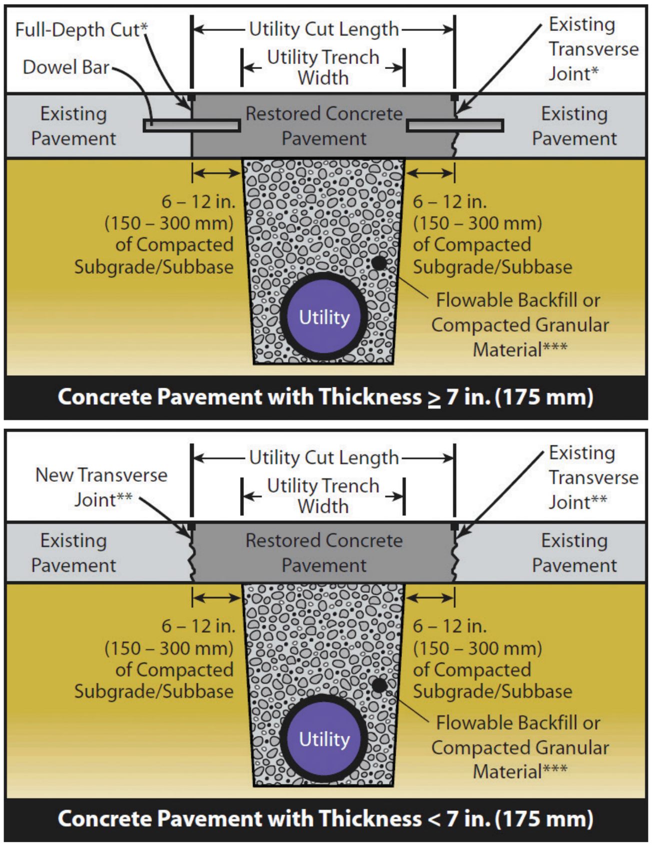
1. **Planning the Utility Cut Location, Size, and Shape**

   - Utility cuts in existing concrete pavements should be made at least 6 to 12 in. beyond the edges of the required trench to prevent the existing concrete from being undermined during the utility repair/installation and to provide support for the restoration patch; some agencies recommend a minimum cutback of 3 ft. An illustration of a utility cut repair for concrete pavements is shown in Figure 6.32.
   - Utility cuts in the slab interior should be located at least 2 ft away from any joints or edges to avoid leaving small sections of concrete that may crack/break under load. If it is determined that a cut will occur within this zone, the cut boundary should be extended to the joint or edge.
   - Utility cut edges should line up closely with joints in the existing pavement to avoid "sympathy cracking."
   - A part of the initial utility cut process may include the creation of utility-locating potholes that, without damaging the utility, provide visual confirmation of the location of the utility and any other subsurface obstructions. One method of accomplishing this while eliminating potential damage to the utility is through the use of pneumatic or hydro excavation that displaces the subsurface materials, which are then vacuumed from the area to reveal the utility.



ACPA 2014, used with permission

Figure 6.31. Summary of the utility cut repair process



**A full-depth cut should be made at any utility cut boundary that is not an existing joint for thicknesses of 7 in. and greater.*

*** For pavements thinner than 7 in., utility cut boundaries that are not at an existing joint should be cut to a depth of about one-third of the slab thickness and the remainder of the depth removed with a jackhammer to provide aggregate interlock load transfer.*

**** Some agencies have had success with up to a 2 ft layer of natural soil above the backfill but below the restored concrete pavement surface course. This layer is used to mitigate differential frost heave or settlement between the utility patch and surrounding pavement.*

ACPA 2014, used with permission

Figure 6.32. Utility cut restoration repair for various concrete pavement thicknesses (≥7 in. and <7 in.)

2. **Making Cuts in the Concrete Slab**
    - For concrete pavements thinner than 7 in., dowel bars can be excluded from the transverse joints as long as sufficient aggregate interlock can be provided for load transfer. Such aggregate interlock can be achieved by sawing partial depth (one-third of the slab thickness) and jackhammering through the remaining depth (since the jackhammer chipping produces a roughened face).
    - For concrete pavements of 7 in. thickness or more, dowel bars are required for load transfer across transverse joints. Since aggregate interlock is therefore not necessary, full-depth sawcuts can be made at any utility cut boundary that is not an existing joint in order to ease removal.
3. **Removing Concrete**
    - The breakup-and-clean-out method of concrete removal is commonly used for local utility cut repairs, but it must not damage the adjacent pavement or excessively undercut (i.e., remove support) beneath it. The breakup-and-clean-out method is accomplished using jackhammers, pavement breakers, and backhoes.
    - The alternative method is to use the lift-out procedure, which should employ full-depth vertical cuts to isolate the slab; this is then followed with the insertion of lifting pins into the slab, so the slab can be lifted out vertically using a backhoe or front-end loader with a chain attached to the pins. (It is important that the slab be removed vertically to prevent the slab from spalling the adjacent pavement.)
4. **Excavating the Base and/or Subgrade**
    - After the removal of the concrete, the excavation is made through the underlying layers to the depth of the utility using a backhoe or similar equipment. Some hand removal is often needed. During this operation, excavation equipment should be kept as far away from the trench area as possible to minimize trench wall sloughing and undercutting of the pavement.
    - The need for shoring to prevent cave-ins will be dictated by the type of subgrade soil, its condition at the time of excavation, and the depth of the utility trench. Local requirements and specifications should be consulted.
5. **Repairing/Upgrading or Installing the Utility**
    - The necessary repairs are made to the utility.
6. **Backfilling the Trench**
    - While the previously removed materials can be used to backfill the trench, many agencies prefer the use of select granular materials, stabilized products, or flowable fill to provide better support for the new slab.
    - Select granular backfill material should be placed in 6 to 8 in. lifts and adequately compacted (95% of density as determined by AASHTO T 99) to minimize settlement.
    - Some agencies use a cement-treated sand or soil as the backfill material. The amount of cement used in such compacted mixes should be only enough to "cake" the material and allow for future excavation rather than to produce a hardened soil-cement.

- The use of flowable fill—a controlled low-strength, self-leveling material made with cement, supplementary cementitious materials (SCMs), and water—has become very common as a backfill material as it easily fills the utility trench area, requires no compaction, and gains strength rapidly (generally within a few hours). It can be placed to the level where the bottom of the new concrete slab will be and can help expedite the overall utility repair construction process.

7. **Installing the Necessary Embedded Steel**
   - All necessary subgrade/subbase and/or backfill compaction should be completed prior to installing dowel bars and/or tie bars into the existing concrete pavement.
   - Recommended dowel bar sizes and drilled hole diameters for utility cut restorations are provided in Table 6.11. The anchoring of the dowel bars should follow the same general procedures as described in Section 5.
8. **Placing, Finishing, Texturing, and Curing the New Concrete Surface**
   - As with other FDRs, a fiberboard bond breaker should be placed along the longitudinal joint of the existing concrete pavement.

Table 6.11. Dowel size recommendations for utility cut restoration

Adjacent pavement thickness (in.)	Dowel diameter (in.)
≤7	No dowel
7–8	1.0
8–10	1.25
10 and above	1.5

Source: ACPA 2014

   - All concrete placement, consolidation, and finishing techniques should follow the procedures described in Section 5. Final surface texturing should match the existing concrete pavement as much as possible. Effective curing techniques should be followed to ensure proper strength and durability.
9. **Jointing and Joint Sealing**
   - Sawed joints should be one-third the slab thickness for any interior contraction joints and the minimum depth necessary for sealant reservoir creation for joints on the utility cut perimeter.
   - Longitudinal and transverse joints should be sealed if the original pavement has sealed joints.
10. **Opening to Traffic**
    - The concrete mixture chosen for the utility cut restoration should be capable of achieving the required strength at the projected time of opening to traffic. Table 6.12 provides recommended minimum required compressive strengths for opening to traffic.

Table 6.12. Minimum opening strength for utility cuts

Utility cut thickness (in.)	Required compressive strength for utility cut length <10 ft (lbf/in^2)	Required compressive strength for utility cut length >10 ft (lbf/in^2)
6	3,000	3,600
7	2,400	2,700
8	2,150	2,150
9+	2,000	2,000

Source: ACPA 2014

8. Quality Assurance

Quality assurance practices for FDRs mirror those for the placement of conventional concrete pavement. Paying close attention to the quality of the material handling and construction procedures during construction greatly increases the chances of minimizing premature failures on FDR projects. This section summarizes key portions of the *Full-Depth Repair of Portland Cement Concrete Pavements* checklist that was created by the CP Tech Center for the FHWA to guide state and local highway agencies on the design and construction of well-performing FDRs (FHWA 2019). These checklist items are divided into the general categories of preliminary responsibilities, project inspection responsibilities, and cleanup responsibilities.

Preliminary Responsibilities

Agency and contractor personnel should collectively conduct a review of the project documentation, project scope, intended construction procedures, materials usage, and associated specifications. Such a collective review is intended to minimize any misunderstandings in the field between agency designers, construction personnel, and inspectors. Specific items for this review are summarized in the next sections.

Project Review

An updated review of the current project's condition is warranted to ensure that the project is still a viable candidate for FDR. Specifically, the following items should be verified as part of the project review process:

- Verify that pavement conditions have not significantly changed since the project was designed and that an FDR is still appropriate for the pavement.
- Check the estimated number of FDRs against the number specified in the contract.
- Agree on quantities to be placed but allow flexibility if deterioration is found below the surface.

Document Review

The key project documents should be reviewed prior to the start of any construction activities. Some of the critical project documents include the following:

- Bid/project specifications and design
- Applicable special provisions
- Traffic control plan
- Manufacturer's specific installation instructions for the selected repair material(s)
- Manufacturer's MSDSs
- Applicable OSHA requirements

Materials Checks

A number of materials-related checks are recommended prior to the start of an FDR project. Specifically, agency and contractor personnel should collectively verify the following:

- The concrete repair material is being produced by a supplier listed on the agency's approved/qualified supplier list as required by the contract documents.
- The mix design for the material has been sampled and tested prior to installation as required by the contract documents. If applicable for determining opening times, verify that maturity curves for the specific mixture have been developed.
- The repair material has been sampled and tested prior to installation and is not contaminated.
- The load transfer units (dowels) meet specifications and are properly coated with epoxy (or other approved material) as well as free of any minor surface damage in accordance with contract documents.
- Dowel-hole anchoring materials meet specifications.
- Bond-breaking materials (typically asphalt-impregnated fiberboard) meet specifications.
- Joint sealant material meets specifications.
- Sufficient quantities of materials are on hand for completion of the project.
- All materials certifications required by contract documents have been provided to the agency prior to construction.

Equipment Inspections

All equipment that will be utilized in the construction of FDRs should be inspected prior to construction. The following items should be verified as part of the equipment inspection process prior to the start of an FDR project.

Concrete Removal Equipment

- Verify that concrete saws and blades are in good condition and of sufficient diameter and horsepower to adequately cut the required repair boundaries.
- Verify that all equipment required for concrete removal is on site and in proper working order and of sufficient size, weight, and horsepower to accomplish the removal process (e.g., front-end loader, crane, forklift, backhoe, skid steer, and jackhammers).

Repair Area Preparation Equipment

- Verify that the plate compactor is working properly and is capable of compacting the subbase material.
- Verify that the gang drills are calibrated and aligned and are sufficiently heavy and powerful enough to drill multiple holes for the dowel bars.
- Verify that air compressors have oil and are equipped with and use properly functioning moisture filters/traps. Check the airstream for water and/or oil by passing the stream over a board and examining for contaminants.

Testing Equipment

- Verify that the concrete testing technicians meet the requirements of the contract documents for training/certification.
- Ensure the required material test equipment is available on site and is in proper working condition. (Equipment typically includes a slump cone, pressure-type air meter, cylinder molds and lids, as well as a rod, mallet, ruler, and 10 ft straightedge.)
- Ensure that sufficient storage area on the project site has been specifically designated for the storage of concrete cylinders.

Placing and Finishing Equipment

- Verify that handheld concrete vibrators are the proper diameter and are 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 for the freshly placed concrete, should it be required.

Weather Requirements

Immediately prior to the start of the construction project and on a daily basis thereafter, the following weather-related concerns should be checked:

- Verify that air and surface temperatures meet manufacturer and contract requirements for the placement of the repair material (commonly 40°F and rising but no more than 90°F).
- Repairs should not be performed if rain is imminent. To prevent rain damage, repairs that have been completed should be covered with polyethylene sheeting.

Traffic Control

The developed traffic control plan should be reviewed by field personnel prior to construction. The traffic control plan should be developed to provide maximum safety to the construction crew, with consideration also given to construction sequencing, productivity, and overall work quality. In developing the traffic control plan, the following traffic-related items should be verified:

- The traffic control setup complies with the Federal *Manual on Uniform Traffic Control Devices (MUTCD)* or local agency traffic control procedures.
- Traffic control personnel are trained/qualified in accordance with contract documents and agency requirements.
- The pavement is not opened to traffic until the repair meets minimum strength requirements.
- Signs are removed or covered when no longer needed.
- Any unsafe conditions are reported to a (contractor or agency) supervisor.
- All workers are wearing the required PPE.

Project Inspection Responsibilities

During the construction process, careful project inspection by construction inspectors helps ensure well-performing FDR installations. Specifically, the following checklist items (organized by construction activity) summarize the recommended project inspection items.

Repair Area Removal and Cleaning

- Verify that removal boundaries are clearly marked and the cumulative removal area is consistent with contract documents.
- Verify that the repair size is large enough to accommodate a gang-mounted dowel drilling rig, if one is being used. (Note: The minimum longitudinal length of an FDR is usually 6 ft.)
- Verify that boundaries are sawed vertically the full thickness of the pavement.
- Verify that concrete is removed by either the breakup or lift-out method, minimizing disturbance to the base or subbase as much as possible. (Note: The sawcut-and-lift-out method is preferred.)
- Verify that any disturbed base or subbase is recompacted, including any added material.
- Verify that adjoining concrete is not damaged or undercut by the concrete removal operation.
- Ensure that removed concrete is disposed of in accordance with contract requirements.

Repair Preparation

- Verify that dowel holes are drilled perpendicularly to the vertical edge of the remaining concrete pavement using a gang-mounted drill rig.
- Verify drill holes are thoroughly cleaned using compressed air.
- Verify that approved cement grout or epoxy is placed in the dowel holes from back to front.
- Verify that dowels are inserted with a twisting motion, spreading the grout along the bar inside the hole. A grout-retention disk can be used to keep the grout from seeping out of the hole.
- Verify that the dowels are installed in transverse joints to the proper depth of insertion and at the proper orientation (i.e., parallel to the centerline and perpendicular to the vertical face of the sawcut). (Typical tolerances measured perpendicularly to the sawed face are 0.25 in. misalignment per 12 in. of dowel bar length.)
- Verify that tie bars are installed at the proper location, to the proper depth of insertion, and to the proper orientation in accordance with contract documents. When the length of the repair is 15 ft or greater, tie bars are typically installed in the manner used for dowels. When the length of the repair is less than 15 ft, a bond breaker board is usually instead placed along the length of the repair to isolate it from the adjacent slab.
- Ensure that tie bars are checked for location, depth of insertion, and orientation (i.e., perpendicular to the centerline and parallel to the slab surface).

Placing, Finishing, and Curing Repair Material

Concrete for FDRs is typically placed from ready mix trucks or mobile mixing vehicles.

- Verify that the fresh concrete is properly consolidated using several vertical penetrations of the concrete surface with a handheld vibrator.
- Verify that the surface of the concrete repair is level with the adjacent slab using a straightedge or vibratory screed.
- Verify that the surface of the fresh concrete patch is finished and textured to match adjacent surfaces.
- Verify that adequate curing compound is applied to the surface of the fresh concrete immediately following finishing and texturing. (Note: 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 the concrete attains the required strength.

Resealing Joints and Cracks

- Verify that joints are cleaned and sealed according to contract documents.

Cleanup Responsibilities

- Verify that all concrete pieces and loose debris are removed from the pavement surface and disposed of in accordance with contract documents.
- Verify that mixing, placement, and finishing equipment are properly cleaned for the next use.
- Verify that all construction-related signs are removed when opening the pavement to normal traffic.

9. Troubleshooting

Table 6.13 summarizes some of the more common problems that a contractor or inspector may encounter in the field during the construction of FDRs, whereas Table 6.14 presents some of the performance problems that may be observed some time after installation. Recommended solutions for these issues are provided in their respective tables.

Table 6.13. Potential FDR construction problems and associated solutions

Problem	Typical solutions
Undercut spalling (deterioration on the bottom of the slab) is evident after the removal of concrete from the patch area	• Saw back into the adjacent slab until sound concrete is encountered
Saw binds when cutting full-depth exterior cuts	• Shut down the saw and remove the blade from the saw • Wait for the slab to cool, then release the blade if possible (or make another full-depth angled cut inside the area to be removed to provide a small pie-shaped piece adjacent to the stuck saw blade) • Make transverse sawcuts when the pavement is cool • Use a carbide-tipped wheel saw to make pressure-relief cuts 4 in. wide inside the area to be removed
Lifting out deteriorated slab damages an adjacent slab	• Adjust the lifting cables and position the lifting device to ensure a vertical pull • Resaw to remove the broken section of the adjacent slab • Ensure the lifting device is capable of performing the operation
Slab disintegrates when attempts are made to lift it out	• Complete removal of the patch area with a backhoe or manual labor, taking care to avoid damaging the adjacent concrete slab and existing subbase • Angle the lift pins and position the cables so that the fragmented pieces are bound together during lift-out • Keep lift height to an absolute minimum on fragmented slabs
Patches become filled with rainwater or groundwater seepage, saturating the subbase	• Pump the water from the patch area or drain it through a trench cut into the shoulder • Recompact the subbase to a density consistent with the contract documents, adding material as necessary • Allow small depressions in the subbase to be filled with aggregate dust or fine sand before the repair material is placed. Permit the use of aggregate dust or fine sand to level small surface irregularities (of 0.5 in. or less) in the surface of the subbase before the concrete repair is placed
Grout around the dowel bars flows back out of the holes after the dowels are inserted	• Place the grout or epoxy in the back of the hole first • Use a twisting motion when inserting the dowel • Add a grout retention disk around the dowel bar to prevent the grout from leaking out
Dowels appear to be misaligned once they are inserted into the holes	• If the misalignment is less than 0.25 in. per 12 in. of dowel bar length, do nothing • If the misalignment is greater than 0.25 in. per 12 in. of dowel bar length on more than three dowel bars per joint, resaw the FDR patch boundaries beyond the dowels and redrill holes • Use a gang-mounted drill rig referenced off the slab surface to drill the dowel holes

Sources: Adapted from FHWA 2019, ACPA 2006

Table 6.14. Potential FDR performance problems and prevention techniques

Problem	Potential solutions
Longitudinal cracking in the patch	• Verify repair dimensions to ensure they are not excessively wide • Verify the proper isolation material and technique have been used • Verify the proper curing material and application have been used • Determine if extreme environmental conditions occurred during placement
Transverse cracking in the patch	• Verify repair dimensions to ensure they are not excessively long • Verify joints are active and not locked and dowels are properly sized and located • Verify the proper curing material and application have been used
Surface scaling	• Investigate the adequacy of the mix design • Investigate whether excess water was applied during placement or finishing • Investigate whether the surface was overfinished • Verify the proper curing material and application have been used
Spalling of transverse or longitudinal joint	• Verify steel placement is correct and transverse joints are not locked • Verify there are no incompressibles in joints • Verify no point-load conditions have occurred in the repair area
Deterioration of material surrounding the repair	• Investigate whether inadequate boundary marking or removal techniques were used • Investigate whether full-depth sawing techniques were used
Repair settlement	• Investigate the technique used for base preparation • Investigate the presence of excess moisture • Investigate the effectiveness of load transfer devices

Source: Adapted from FHWA 2019

10. Summary

The full-depth repair of a concrete pavement involves the full-depth (and generally full-lane-width) removal of a deteriorated portion of an existing concrete slab and its replacement with an appropriate concrete repair material that meets the durability and traffic opening demands of the project. Full-depth repairs may be necessary to address distresses (such as deteriorated cracks and joints, corner breaks, and blowups) that are adversely affecting ride or safety. Such repairs, when properly constructed, can prevent or retard further deterioration and can thereby contribute to the continued long-term performance of the pavement. Full-depth repairs are also often used to prepare distressed concrete pavements for a structural overlay.

Long-lasting FDRs are dependent upon many items, including appropriate project selection, effective load transfer design, and effective construction procedures. This chapter provided guidance on recommended design and construction procedures to install effective FDRs of both jointed and continuously reinforced concrete pavements.

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Chapter 7
Retrofitted Edgedrains

1. Introduction

Subsurface drainage systems are commonly believed to contribute to the improved performance of both asphalt and concrete pavements (Hall and Crovetti 2007).

Although there is some research indicating that drainage can effectively extend concrete pavement life (see, for example, Darter et al. 1985, Cedergren 1987, Smith et al. 1998, Christopher 2000, Ji and Nantung 2015), other studies suggest that certain design and construction factors may have a bigger effect on performance than drainage (Harrigan 2002). For instance, a permeable base in a doweled JPCP was observed to make only a minimal contribution to performance, whereas the same permeable base in a nondoweled JPCP design significantly improved performance (Harrigan 2002). Moreover, an evaluation of Long-Term Pavement Performance (LTPP) Specific Pavement Studies (SPS) field sections found that the installation of pavement drainage did not have an impact on concrete pavement smoothness or faulting, although the lack of pavement drainage did lead to increased longitudinal and transverse cracking (Hall and Correa 2003).

In this vein, it is postulated that many of today's pavements are less vulnerable to the detrimental effects of excessive moisture, largely because of the addition of key design features such as thicker slabs, doweled joints, widened slabs, tied shoulders, and stabilized or nonerodible bases (Hall and Crovetti 2007). Nevertheless, positive drainage may still be required for existing concrete pavements exposed to excessive moisture throughout the year if they were constructed without these modern design features.

Although the ideal time to address drainage concerns is during a pavement's initial design and construction, several state and local highway agencies have installed edgedrains on existing pavements to alleviate moisture-related problems. The purpose of retrofitted edgedrains is to collect water that has infiltrated into the pavement structure and remove it from beneath the pavement structure where it could contribute to distress development.

Retrofitted edgedrains are most commonly used on concrete pavements that are not exhibiting significant structural deterioration but have begun to show signs of moisture-related distresses, such as pumping and joint faulting. Agencies typically install edgedrains in an effort to delay or slow the development of such moisture-related distresses, but it is important that only the right pavements be targeted and that effective installation procedures be followed for the anticipated benefits to be obtained.

Urban concrete pavements may also benefit from drainage, and many have been constructed with edgedrains that are tied into the roadway drainage inlets. However, retrofitting edgedrains in an urban environment is very difficult, given the limited available room for construction as well as potential utility conflicts.

The remainder of this chapter presents information regarding the process of retrofitting existing concrete pavements with edgedrains. Included are discussions of key definitions, retrofitted edgedrain project selection, the limitations and effectiveness of the retrofitted edgedrain method, design considerations, and construction considerations. Also included is a summary of recommended maintenance activities to help ensure the long-term effectiveness of retrofitted drainage systems.

2. Purpose and Project Selection

Purpose of a Pavement Drainage System

Water that accumulates beneath a pavement structure can reduce the load-carrying capacity of the pavement and contribute to the development of critical moisture-related distresses such as pumping, faulting, and corner breaks. The purpose of a pavement drainage system, therefore, is to remove excess water that infiltrates the pavement structure to reduce the development of moisture-related damage. The overall goal is to minimize the amount of time that water is beneath the pavement and particularly the period of time that the underlying pavement layers are in a saturated condition.

When an existing pavement begins showing signs of moisture-related damage, an agency generally has two options for improving the pavement's drainage: (1) wait to redesign the subdrainage system when reconstruction of the pavement is required or (2) retrofit the existing pavement with an edgedrain system.

When a pavement is reconstructed, the designer has the luxury of conducting a complete pavement subsurface drainage analysis to optimize the selection of all components of the pavement drainage system. Pavement subsurface drainage analysis and design methods are documented in several references (FHWA 1992, ERES Consultants, Inc. 1999, Christopher et al. 2006, Arika et al. 2009). In addition, the comprehensive computer program DRIP is available to perform detailed drainage analyses (Mallela et al. 2002, AASHTO 2015)—though it should be pointed out that Neshvadian et al. (2017) have documented improvements needed to DRIP's jointed concrete pavement moisture infiltration model.

A detailed drainage analysis can also be conducted on an existing concrete pavement to determine its ability to drain water from beneath the pavement, but in this case the pavement layers are already in place and little can be done to improve each layer's ability to drain. Because of this, the goal in retrofitted drainage projects is to shorten the drainage path (i.e., the distances that water must travel to get out from beneath the pavement structure).

Figure 7.1 presents pavement cross sections before and after installation of a retrofitted edgedrain that illustrate a shorter flow path to the edgedrain location than to the pavement sideslope. It is important to realize, however, that the permeability of the granular base will still play a significant role in determining how quickly the water can be removed from the pavement structure.

In addition, edgedrains can help intercept water that infiltrates at the lane-shoulder joint. Olson and Roberson (2003) point out that as much as 85% of the water from a rain event can be prevented from entering a pavement system in the first place simply by sealing and maintaining the lane-shoulder joints.

Project Selection for Retrofitting Edgedrains

The presence of moisture-related distress is a good indicator of projects with poor drainage, but an excessive amount of pavement deterioration may suggest that it is too late for the addition of subsurface drainage to slow distress development.

It is not always clear if retrofitted edgedrains are an appropriate rehabilitation option for a given project.

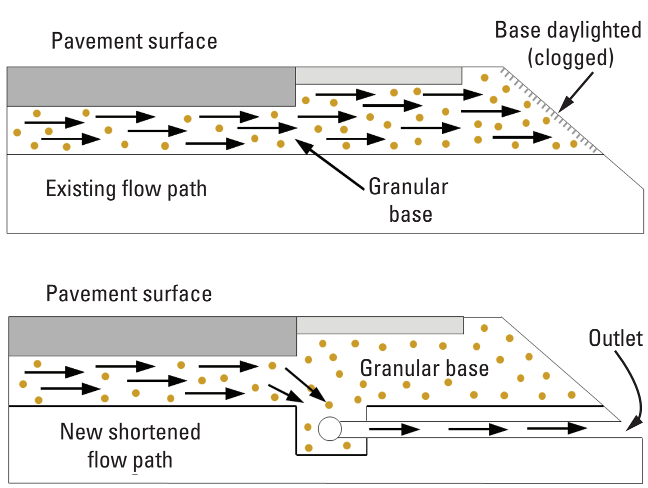

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Figure 7.1. Before (top) and after (bottom) addition of longitudinal edgedrain to improve granular base drainability

Thus, as a first step in identifying candidate projects for retrofitted edgedrains, a comprehensive distress and drainage survey should be conducted to assess current pavement conditions, identify the sources of water, and assess the condition and erodibility of the base material.

The types of moisture-related distresses present provide a good indication of the appropriateness of installing retrofitted edgedrains. A good candidate project for retrofitted edgedrains is a pavement that is showing early signs of moisture damage, is relatively young, and is exhibiting only a minimal amount of cracking. On the other hand, pavements exhibiting any of the following conditions are considered poor candidates for retrofitted edgedrains (Wells 1985, FHWA 1992, ERES Consultants, Inc. 1999, ITD 2007):

- More than 10% of the slabs exhibit cracking.
- A high number of transverse joints are spalled.
- Significant pumping is present throughout the project (unless the voids under the pavement are to be corrected through slab stabilization).
- Other significant distresses (such as edge punchouts, transverse cracking, longitudinal cracking, and corner breaks) are present that would require extensive patching to return the pavement to an adequate level of service.
- The existing base contains more than 15% fines (material passing the No. 200 sieve), which may be too impermeable for an effective retrofitted subdrainage installation.

In summary, retrofitted edgedrains are *not* effective at prolonging the service life of existing concrete pavements that are already exhibiting significant structural and moisture-related deterioration or have highly erodible bases.

Other factors to consider in evaluating the suitability of an existing concrete pavement project for retrofitted edgedrains are whether its geometrics (longitudinal and transverse slopes) are acceptable and whether the depth and condition of its roadside ditches are adequate. It is important that these pavement characteristics be adequate (or improved during the edgedrain installation) so that water can be removed effectively. In addition, consideration should be given to providing edgedrains only in critical drainage areas (such as on curves or in low areas) and not necessarily throughout the entire length of a project.

3. Limitations and Effectiveness

The performance of pavements with retrofitted edgedrains has been mixed. For example, a national study of pavement drainage showed varying results in terms of the benefits of retrofitted drainage on pavement performance, with noted reductions in faulting on some projects but no such reduction on other projects (Harrigan 2002). Retrofitted edgedrains have even been found to have contributed to the further deterioration of some pavement structures (Gulden 1983, Wells and Nokes 1993) as a result of either the drains removing base and soil material from beneath the pavement slabs (leading to poor support conditions) or clogging of the outlets (leading to pavement saturation and therefore reduced support conditions).

A review of LTPP SPS-1 and SPS-2 sites (Hall and Correa 2003) rated drainage functionality as good for only 50% of the concrete sections and 55% of the asphalt sections. In other words, edgedrain clogging is not uncommon—a study in Iowa found that about 65% of drainage outlets along concrete pavements were blocked by either tufa (due to the use of recycled concrete), sediment, or soil (Ceylan et al. 2013). Similar investigations in Kentucky have also indicated the blockage of a significant number of headwalls and outlets (Ashurst and Rister 2019).

Overall, this inconsistent performance of retrofitted edgedrains can be attributed to a combination of improper usage (i.e., poor project selection), improper design, damage during installation, lack of maintenance, or the failure during edgedrain construction to perform other needed pavement repairs. Indeed, a study of edgedrains in California found that more than 70% of edgedrains were not performing efficiently, but their overall poor performance was attributed to design flaws, improper construction, and/or lack of maintenance (Bhattacharya et al. 2009).

In considering subsurface drainage for an existing pavement, a design engineer is forced to deal with the existing pavement's materials and conditions. As previously mentioned, perhaps the biggest issue is the condition and permeability of the base course, because this could significantly limit the ability of water to migrate from beneath the pavement to the edgedrains.

For an existing pavement to be a good candidate for retrofitted edgedrains, it is often suggested that the base course contain no more than about 15% fines—ideally even less to provide some degree of permeability. However, even when the base course exceeds 15% fines, it should be noted that there can be some benefit to the use of retrofitted edgedrains for removing surface infiltration water that enters at the lane-shoulder joint (Christopher et al. 2006). Since the lane-shoulder joint is a primary entry point of surface infiltration water, retrofitted edgedrains—by virtue of their installation at that location—can remove that water regardless of the permeability or gradation of the pavement's base course.

At the national level, limited guidance is available on the effectiveness of pavement drainage. For example, the *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures* (ARA, Inc. 2004) states that "the current state of the art is such that conclusive remarks regarding the effectiveness of pavement subsurface drainage or the need for subsurface drainage are not possible." *The Mechanistic-Empirical Pavement Design Guide: A Manual of Practice* (AASHTO 2020) states that this places the burden on the individual roadway agency to assess the value of providing subsurface drainage based on local climatic and subsurface conditions, pavement designs, and design practices.

However, the *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice* (AASHTO 2020) further notes that studies have shown that subsurface drainage improvement via retrofitted edgedrains can reduce faulting, especially for nondoweled JPCP. This is accomplished by retrofitted edgedrain design reducing the amount of precipitation infiltrating into the pavement structure. Nevertheless, it cannot be overemphasized that the proper installation, construction, and maintenance of retrofitted edgedrain systems are critical to their functionality and performance (Daleiden 1998, Ashurst and Rister 2019).

4. Materials and Design Considerations

Materials Considerations

Types of Edgedrains

Historically, the following three types of edgedrain systems have been used on retrofitted drainage projects:

- Pipe edgedrains
- Prefabricated geocomposite edgedrains (PGEDs)
- Aggregate drains (sometimes called French drains)

For each of these edgedrain alternatives, it is important that they be placed deep enough in the existing pavement structure to effectively collect the infiltrated water (Bhattacharya et al. 2009). More detailed descriptions of each of these three types of edgedrain systems are provided in the following sections.

Pipe Edgedrains

A pipe edgedrain system consists of a perforated longitudinal conduit placed in an aggregate-filled trench running along the length of a roadway. Water is discharged from the pipe edgedrain into the nearest ditch through regularly spaced transverse outlet pipes connected to the longitudinal drainage pipe.

Perforated corrugated plastic is commonly used for the longitudinal collector pipe, although increasingly rigid, smooth-walled plastic pipe is being used instead because it lies flat in the trench and is less susceptible to crushing. To prevent the infiltration of fines, the trench is partially lined with geotextile fabric in areas where it comes into contact with either the subbase or subgrade materials, and then it is backfilled with stabilized or nonstabilized open-graded material.

A typical cross section of a pavement retrofitted with a pipe edgedrain system is presented in Figure 7.2.

Prefabricated Geocomposite Edgedrains

PGEDs, also known as "panel" or "fin" drains, consist of a flat, extruded plastic drainage core wrapped with a geotextile filter. Figure 7.3 shows an assortment of typical PGEDs, while Figure 7.4 depicts a recommended PGED installation detail.

PGEDs are typically 0.5 to 1 in. thick and are manufactured in long strips that are coiled into rolls. Their size and the incorporation of a geotextile filter directly into their design mean that PGEDs can be placed in narrower trenches than conventional pipe edgedrain installations.

Although PGEDs generally have less drainage capacity than pipe edgedrains, this is typically not a problem on most retrofitted drainage projects since high water inflows are not normally expected (because the existing dense-graded base course materials typically do not have high permeabilities).

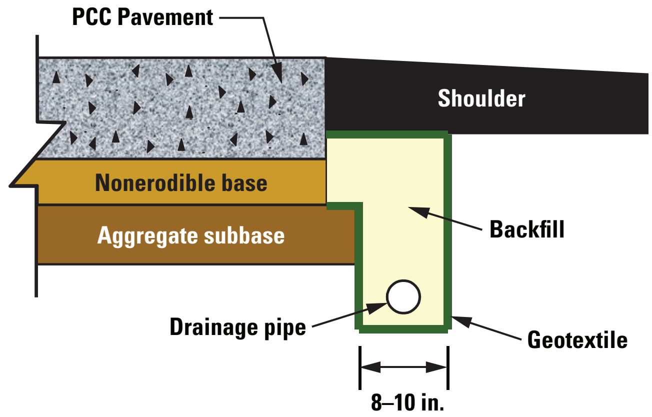

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Figure 7.2. Recommended design for a pipe edgedrain

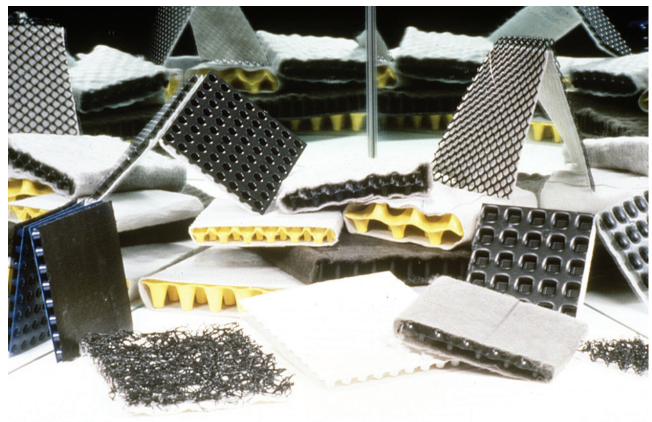

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Figure 7.3. Typical prefabricated geocomposite edgedrains

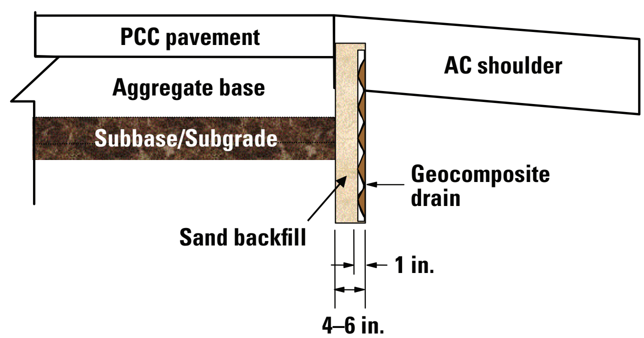

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Figure 7.4. Recommended installation detail for geocomposite edgedrains

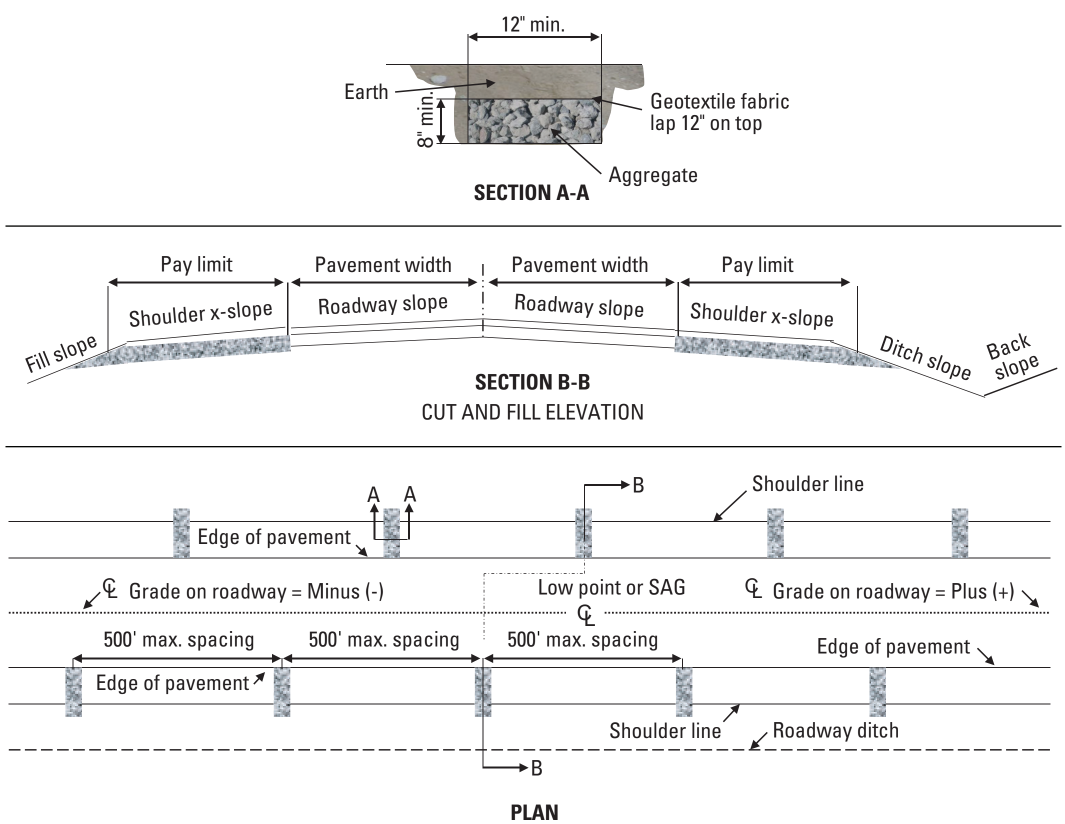
The main advantage of PGEDs is that they are easier and cheaper to install than conventional pipe edgedrains. One disadvantage of PGEDs, however, is their susceptibility to damage during construction. For example, if proper care is not taken during the backfilling operation, crushing, bending, or buckling of the drainage core may occur (Koerner et al. 1994). This can lead to siltation and clogging issues in PGEDs, which has prompted several highway agencies to prohibit their use.

Nevertheless, generally good performance has been obtained with PGEDs, and it has been reported that most failures have been predictable and have been related to poor drainage design, misapplication of the treatment, or improper construction techniques (Christopher 2000). Furthermore, installing a PGED on the shoulder side of the trench (as shown in Figure 7.4) helps to minimize buckling and allows for more effective filling of any voids that may develop in the base beneath the slab during the trenching operation (Fleckenstein et al. 1994, Koerner et al. 1994).

Aggregate Drains

Aggregate drains, consisting of a free-draining aggregate trench constructed at the edge of a pavement, have not typically been recommended because they have a relatively low hydraulic capacity and cannot be maintained (FHWA 1992). Still, some agencies (for example, Missouri and Ohio) have used aggregate drainage systems effectively for pavements without other subsurface drainage (particularly on lower volume roadways) or to provide localized, spot drainage improvements.

Aggregate drains are physically cut into the edge of a pavement and configured such that the bottom of the drain is at or below the bottom of the pavement's aggregate base. Figure 7.5 illustrates an aggregate drainage system that is used in Missouri.



Adapted from MoDOT 2013

Figure 7.5. Aggregate drainage system used in Missouri

Backfill Material

Backfill/filler material is placed in the trench around the pipe or alongside the geocomposite and serves the following functions:

- Acts as a drainage medium to provide a means by which water is moved from the pavement layers to the drainage pipe
- Acts as a filter system that prevents or restricts fines from moving into and clogging the drainage system (although its effectiveness for this purpose may diminish over time)
- Supports and confines the drainage pipe or geocomposite, providing protection to the drainage system both during construction and after it is in service
- Provides stabilization to the soil around the drainage trench

There are specific procedures available for designing the backfill/filler material to ensure that the drainage apparatus, be it a pipe or geocomposite product, does not become clogged with fines. Recommended gradations for the backfill/filler material are found in numerous references (FHWA 1992, ERES Consultants, Inc. 1999, Mallela et al. 2002).

For pipe edgedrains, the backfill material for the trench should be at least as permeable as the base material. In a permeable base section, the backfill material will usually be the same as the base material. Also, AASHTO No. 57 gradation aggregate generally provides sufficient permeability and stability for use as a nonstabilized backfill material. Nonstabilized pea gravels are not recommended as the backfill material for retrofitted edgedrains because they cannot be compacted satisfactorily. Proper compaction of the backfill material is important to avoid settlement over the edgedrain, yet overcompaction should also be avoided to prevent damage to the drain itself.

Design Considerations

The design of edgedrains is a multistep process that mainly consists of calculating the amount of water that is expected to infiltrate a pavement and then selecting edgedrain details that will allow the drainage system to effectively remove this water. The general philosophy is that each segment of the drainage system should be adequately sized to meet the current capacity demands as water moves toward the outlet, as shown in Figure 7.6.

In addition to properly sizing the components of an edgedrain system, however, it is important to design filters (geotextile or aggregate) that are effective at preventing fines from entering and clogging the edgedrain over the life of the system (Christopher 2000). The grade of the invert (bottom elevation of a pipe) must also be established to maintain flow, and the outlets must be spaced and sized appropriately to prevent backup in the edgedrain system (Christopher 2000).

The following sections provide an abbreviated explanation of the major considerations associated with designing effective retrofitted edgedrains, with more detailed information provided elsewhere (Moulton 1980, FHWA 1992, ERES Consultants, Inc. 1999, Arika et al. 2009).

As previously noted, the DRIP computer program is available at the AASHTOWare Pavement ME Design website and can be used to perform the detailed drainage analyses required for effective retrofitted edgedrain design (Mallela et al. 2002, AASHTO 2015).

Estimating the Design Flow Rate

The first step in the design of retrofitted edgedrains is the determination of the net inflow of water, as the subdrainage system must be adequately sized to handle the flow of water to which it will be subjected.

As previously mentioned, for most pavement rehabilitation projects, surface infiltration is the primary concern. Groundwater, meltwater, and subgrade outflow are generally relatively small and are often ignored in the drainage analysis for retrofitted edgedrains. However, if these sources of water are determined to be critical on a project, then drainage treatments specific to these sources of inflow will be required (see Christopher et al. 2006, Arika et al. 2009).

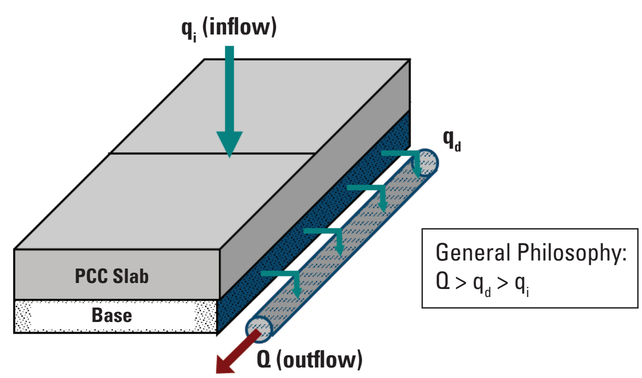


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Figure 7.6. Sizing elements of a pavement drainage system based on estimated water inflow into the pavement (q_i), amount of water the pavement's base course can accept (q_d), and amount of water the edgedrain must therefore discharge (Q) in units of volume per time

The amount of surface infiltration is a function not only of pavement cracking and surface permeability but also of the ability of the base course to accept and remove water. As shown in the previous Figure 7.6, the actual infiltration will be the lesser of two values: (1) the amount of water that could enter through a pavement's cracks and joints (q_i) or (2) the amount of water that the pavement's base course is able to accept (q_d).

The design flow rate, Q, is the estimate of the amount of infiltrated water that will be required to be discharged through the edgedrain system (in units of volume per time). This value is typically estimated through knowledge of detailed information about the base (i.e., width, thickness, and permeability) and encountered slopes (i.e., cross slope and longitudinal edgedrain slope).

Details of the available methods for computing this design flow rate are described elsewhere (Moulton 1980, FHWA 1992, ERES Consultants, Inc. 1999). However, it should also be noted that these calculations are now automated in the previously mentioned DRIP software that is available from the AASHTOWare Pavement ME Design website (AASHTO 2015).

Edgedrain Collector Selection

The two types of longitudinal edgedrains commonly used for retrofitted drainage projects are pipe edgedrains and PGEDs. It is important that the selected collector type be compatible with the existing pavement structure as well as with the surrounding materials.

For pipe edgedrains, several types of drainage pipes of various lengths and diameters have been used successfully in edgedrain collector systems. State and local highway agencies have characteristically used flexible corrugated polyethylene (CPE) or smooth rigid polyvinyl chloride (PVC) pipe, adhering to AASHTO M 252 or AASHTO M 278, respectively (see Figure 7.7). CPE pipe has commonly been used, but many agencies are moving toward the use of PVC pipe instead because rigid pipe lies flat in the edgedrain trench and is less susceptible to crushing.

For PGEDs, product selection should be based on an evaluation following the test procedures outlined in ASTM D6244, Standard Test Method for Vertical Compression of Geocomposite Pavement Panel Drains.

Edgedrain Collector Sizing

Edgedrains must be sized so that their capacity is larger than the expected design flow rate. The specific diameter of a pipe edgedrain is often selected as the minimum diameter that facilitates maintenance (e.g., cleaning) activities and allows a reasonable distance between outlets (Christopher 2000). Pipe diameters typically range from 1.5 to 8 in., with 4 in. being the most common. The larger sizes are usually preferred because they are easier to clean and maintain. A typical cross section for a PGED, on the other hand, has a width of 0.5 to 1.0 in. and a height of 12 to 18 in. (Fleckenstein et al. 1994).

As mentioned previously, the computation of the actual flow capacity (required to determine appropriate edgedrain size) is described in several other publications (Moulton 1980, FHWA 1992, ERES Consultants, Inc. 1999, Christopher et al. 2006, Arika et al. 2009). These computations can be done manually, but they are completely automated in the DRIP software program (AASHTO 2015). Figure 7.8 provides screenshots from the DRIP program, showing its inputs and design calculations.

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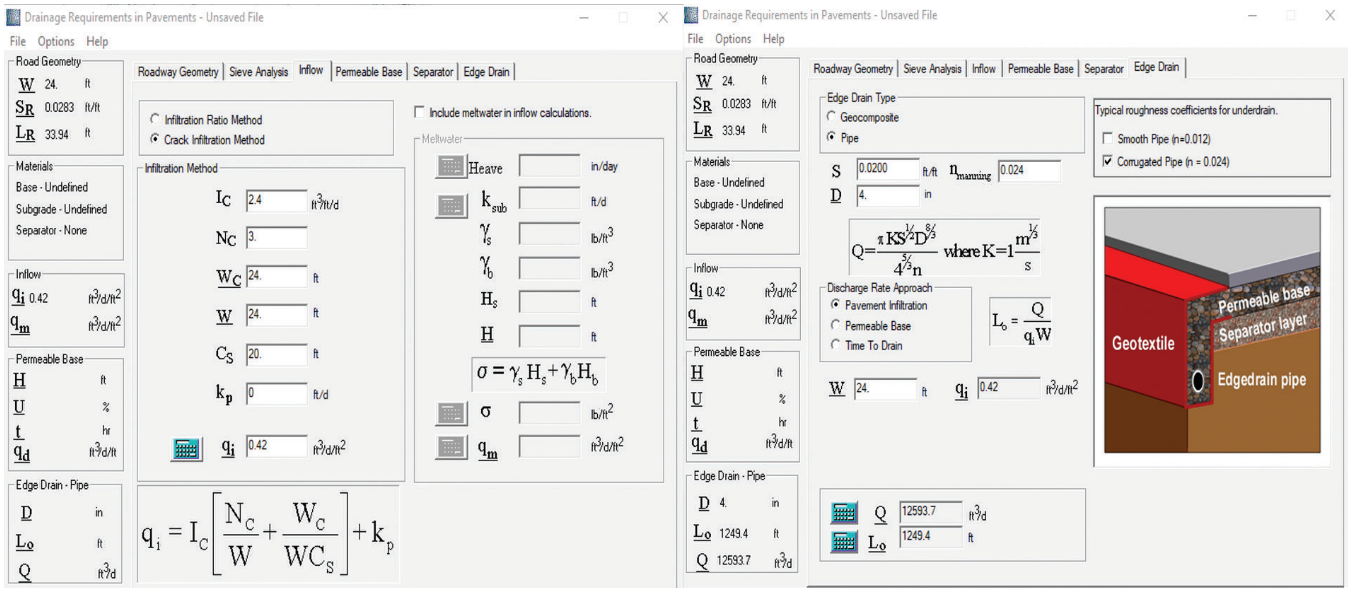
Figure 7.7. Corrugated (left) and rigid (right) pipe edgedrains



FHWA and ARA, Inc.

Figure 7.8. Screenshots from DRIP drainage design calculations

Edgedrain Location

The design depth for edgedrain collector pipes should consider the down elevation available for outletting the water collected, the likelihood and depth of frost penetration, and economics. Where significant frost penetration is not likely and no attempt is being made to remove or draw the groundwater, it is recommended that the trench depth be deep enough to allow the top of the pipe to be located 2 in. below the subbase/subgrade interface. When significant frost penetration is expected, the trench should be constructed only slightly deeper than the expected depth of frost. In ditch sections, the maximum depth of the collector trench is limited by the depth of the ditch, as discussed on the following page in the Outlet Considerations section.

The location of the drain within the trench is also a major concern for retrofitted PGEDs. As described previously, the recommended approach is to place PGEDs on the shoulder side of the edgedrain trench (see the previous Figure 7.4). Studies have shown that this approach helps minimize voids within the trench, alleviate the problem of soil loss through the geotextile filters, and avoid the bending and buckling of the PGED (Koerner et al. 1994).

Grade Considerations

In most cases, edgedrain collector pipes are placed at a constant depth below the pavement surface. This results in the pipe grade being the same as the pavement grade. When the pavement grade is very flat, however, other means must be employed to ensure that water can flow through the pipe.

The most practical solution is to use smooth pipe and decrease the outlet spacing where flat grades exist. The other option is to increase the grade of the edgedrain. Previous guidance recommends grades of at least 1% for smooth pipes and at least 2% for corrugated pipes (Moulton 1980). This latter solution, however, can be impractical for very flat areas. For instance, the use of a 1% grade over a 660 ft long flat section would require an edgedrain to be 6.6 ft deep on the low side.

Trench Width

The required width of an edgedrain trench is a function of construction requirements, drainage requirements, and the permeability of the trench material. Depending on the size of the edgedrain pipe, many agencies use a trench width of 8 to 10 in. to allow proper placement of the pipe and appropriate compaction of the backfill material around the pipe. For PGEDs, a narrower trench of 4 to 6 in. is typical, but this makes avoiding damage to the geocomposite during compaction of the backfill difficult.

Filter Design

Geotextile materials play a pivotal role in edgedrain systems. Acting as a filter layer, the geotextile must simultaneously allow water to pass and prevent fines from passing, and it must perform both of these functions throughout the life of the drainage system (Koerner et al. 1994). For both pipe and PGED systems, it is recommended that geotextiles line the trench wherever the backfill material comes into contact with the subbase or subgrade.

Geotextiles consist of either woven or nonwoven mats of polypropylene or nylon fibers. These fabrics are used in place of graded filter material, permitting greater use to be made of locally available aggregate gradations without special processing. To be effective in an edgedrain system, the selected geotextile must have the following three characteristics (Koerner et al. 1994):

- Its voids must be sufficiently open to allow water to pass through the geotextile and into the downstream drain without building excessive pore water pressure in the upstream soil.
- Its voids must be sufficiently tight to adequately retain the upstream soil materials so that soil loss does not become excessive and potentially lead to clogging of the downstream drain.
- It must perform the previous two conflicting tasks—simultaneously requiring open voids and tight voids, respectively—over the anticipated service life of the retrofitted drainage system without excessively clogging.

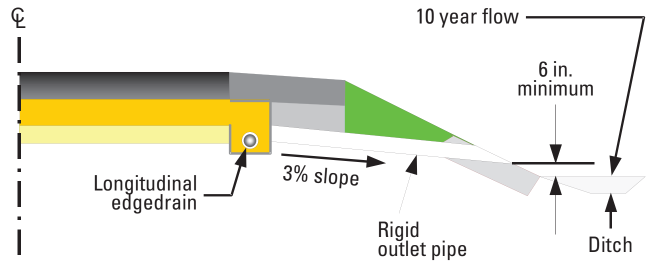
Geotextiles should therefore be designed considering both the subbase and subgrade soils using the filter criteria in the FHWA geosynthetics design manual (Holtz et al. 1998). If geotextile fabrics are not used, the gradation of the aggregate used to fill the trench must be designed to be compatible with the subbase and subgrade soils using standard soil mechanics filter criteria (Christopher 2000).

Clogging of edgedrains can result when geotextile materials are used that do not account for the properties of the surrounding soil (Bhattacharya et al. 2009). In particular, soils and aggregate bases with significant fines are prone to clogging geotextile materials.

Outlet Considerations

The outlet pipe is either a 4 in. diameter, nonperforated, smooth-walled PVC pipe or a high-density polyethylene (HDPE) pipe and should be placed at a minimum slope of 3% (Christopher 2000). Good compaction control of the backfill below, around, and above the pipe is required to avoid transverse shoulder sags (Christopher 2000).

The outlet end should be placed at least 6 in. above the 10-year ditch flow line and be protected with a headwall and splash block that is blended into the slope. Designers should confirm the 10-year flow elevations with the hydraulics staff of their agency to ensure they have the latest information. Figure 7.9 illustrates a recommended outlet pipe configuration.



Adapted from FHWA 1992

Figure 7.9. Outlet pipe configuration

Ashurst and Rister 2019, Kentucky Transportation Center

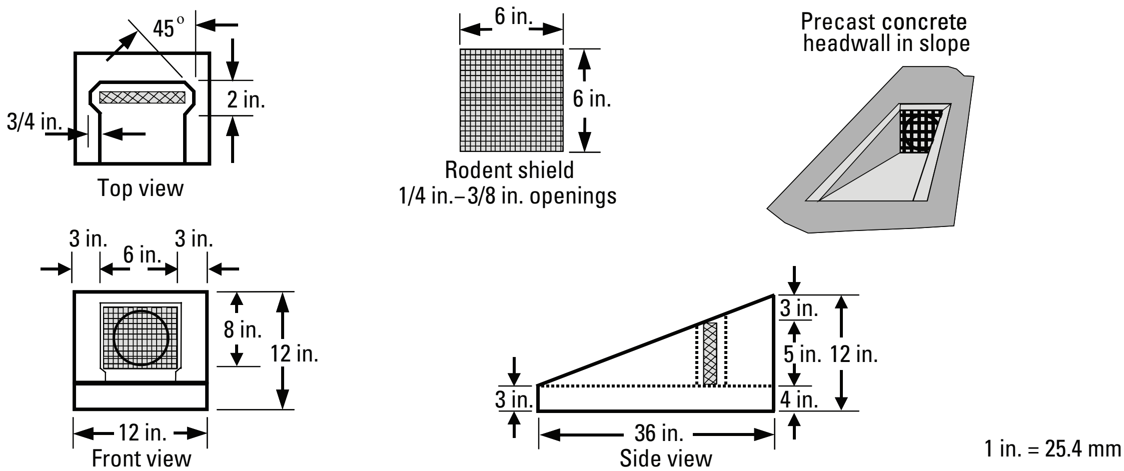
Figure 7.10. Edgedrain outlet pipe holding water due to inadequate ditch grade

Failure to provide the necessary grade separation may result in the outlet holding water, as shown in Figure 7.10.

If high ditch flows are expected, flap valves can be used to prevent backflow into the drainage system. Ashurst and Rister (2019) have noted that designers may want to consider alternative outlets such as dry wells (underground, porous walled structures that allow water to slowly soak into the ground).

The location of outlets is necessarily controlled in part by topography and roadway geometrics, in that the locations must permit free and unobstructed discharge of the water. It is particularly important to accommodate low points and the sags of vertical curves. In general, the recommended outlet spacing is between 250 and 300 ft to facilitate the cleaning of the system, but this will also depend on the anticipated outflows and the topography of the project. For example, projects with particularly flat slopes may require closer outlet spacings (Christopher 2000).

Headwalls are recommended at outlet locations because they protect the outlet pipe from damage, prevent slope erosion, and facilitate the location of the outlet pipes (FHWA 1992). Headwalls can be either cast in place or precast elements and should be placed flush with the slope to facilitate mowing operations.

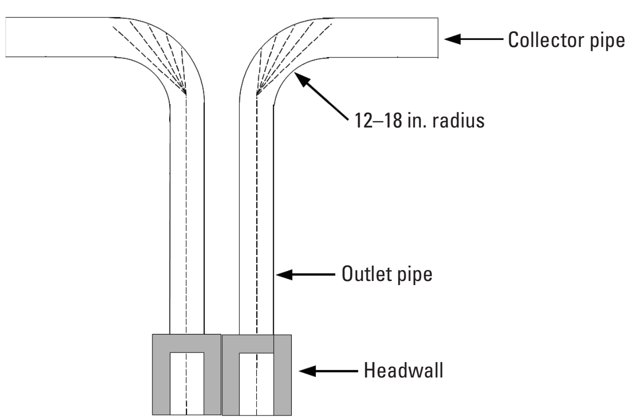


Adapted from FHWA 1992

Figure 7.11. Recommended details for precast headwall with rodent screen

To prevent animals from nesting in the pipe, the headwall should be provided with a removable screen or similar device that allows easy access for cleaning; however, one study suggested that these screens may contribute to blockage of the outlet (Ceylan et al. 2013). A precast headwall with a removable rodent screen is illustrated in Figure 7.11.

If pipe edgedrains are used, the outlet pipes should be connected to the collector pipe through elbows with minimum radii of 12 to 18 in. This alignment facilitates access for cleaning and flushing the pipe. A dual outlet system is also recommended to allow video inspection and maintenance from either end. A recommended dual outlet system design is shown in Figure 7.12.



After FHWA 1992

Figure 7.12. Dual outlets to facilitate pipe edgedrain cleaning and inspection

Other Repair Considerations

It is critical that the potential need for other repairs to the existing pavement be considered when designing a retrofitted edgedrain project. If the pavement does not receive needed repairs prior to (or at the same time as) the installation of retrofitted edgedrains, the effectiveness of the retrofitted edgedrains will be limited (ERES Consultants, Inc. 1999).

For instance, concrete pavements that exhibit visible pumping and noticeable faulting should be evaluated for possible slab stabilization (see Chapter 4) prior to the installation of edgedrains. The preservation treatments of joint resealing (see Chapter 10), DBR (see Chapter 8), and diamond grinding (see Chapter 9) should also be considered as appropriate. Without these repairs where needed, continued pumping, faulting, and loss of support can be expected, even with the addition of the retrofitted edgedrain system.

5. Construction Considerations

Proper construction and maintenance are extremely important to ensure the effectiveness of the edgedrain system. The construction steps involved in retrofitting edgedrains for an existing pavement differ slightly depending on the type of edgedrain being used.

Pipe Edgedrains

Step 1: Trenching

It is important to maintain correct line and grade when installing longitudinal edgedrains. A mechanical track-driven trencher is often used to create a trench along the edge of the pavement. A large diameter, carbide-tipped wheel saw may also be used. The spoils from the trench must be expelled from the trench and any excess, loose, or foreign material swept away.

As described previously, where significant frost penetration is not likely and no attempt is being made to remove or draw the groundwater, it is recommended that the trench depth be deep enough to allow the top of the drain to be located 2 in. below the subbase/subgrade interface. When significant frost penetration is expected, the trench should be constructed only slightly deeper than the expected depth of frost to ensure that the system can function during freezing periods.

In ditch sections, the maximum depth of the collector trench is limited by the depth of the ditch. Outlets from the system should be located 6 in. above the ditch flow line to preclude the backflow of water from the ditch. Similarly, if the system is to outlet into a storm drain system, the outlet invert should be at least 6 in. above the 10-year expected water level in the storm drain system (as shown previously in Figure 7.10).

Step 2: Placement of Geotextile

When pipe edgedrains are used, the trench should be lined with a geotextile to prevent the migration of fines from the surrounding soil into the drainage trench. If a permeable base is present, it should have a direct path to the trench backfill to allow a direct path for water into the drainage pipe. The geotextile must satisfy the filter requirements for the specific subgrade soil (as presented in Holtz et al. 1998). Figure 7.13 shows the placement of a geotextile in a trench, with the CPE pipe laid as well.

John Donahue, MoDOT, used with permission

Figure 7.13. Geotextile-lined trench with CPE pipe

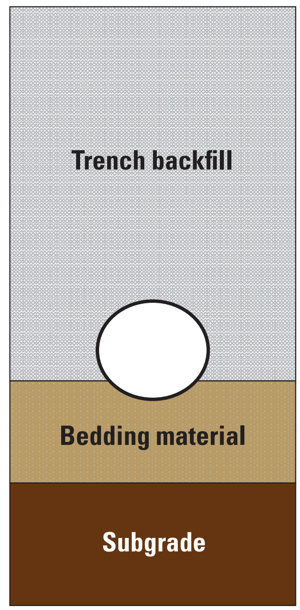


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Figure 7.14. Corrugated polyethylene pipe installed within groove in bedding material

Step 3: Placement of Drainage Pipes and Backfilling

If a layer of bedding material will be placed prior to placing the drainage pipes, the bottom of the trench is grooved after placing the bedding material to accommodate the CPE pipe (see Figure 7.14).

When placing CPE pipes, extra care is also required to prevent overstretching of the pipes during installation. The typical limit for tolerable longitudinal elongation of CPE pipes is 5% (ERES Consultants, Inc. 1999).

The backfill material should be placed using chutes or other means to avoid dumping the material onto the pipe from the top of the trench. To prevent the displacement of drainage pipes during compaction, the backfill material should not be compacted until the trench is backfilled above the level of the top of the pipes. To avoid damage to the pipes, a minimum of 6 in. of cover over the drainage pipe is recommended before compacting (ERES Consultants, Inc. 1999).

Achieving adequate consolidation in a narrow trench can be difficult, but inadequate compaction can lead to settlement, which in turn will result in shoulder distresses. Some agencies use treated permeable materials to backfill drainage trenches in order to avoid the settlement problem. Generally, several passes of an approved vibrating pad, plate, or compactor are used to consolidate the backfill material, usually toward the goal of a minimum density of 95% as measured by a standard Proctor test (AASHTO T 99).

A Minnesota study showed that satisfactory compaction can be achieved by running two passes (two lifts, one pass per lift) with a high-energy vibratory wheel (Ford and Eliason 1993). Each pass of the vibratory wheel was found to be effective in achieving the target density to a depth of 12 in. The Minnesota study also showed that the degree of compaction can be verified easily using a DCP. (See more about DCP testing in Chapter 3.)

Automated equipment has been developed that can be used to install either smooth-walled or corrugated plastic pipes. Figure 7.15 shows the equipment for installing and backfilling longitudinal edgedrains in an actual installation process. Productivity for this equipment is about 3 mi/day.

John Donahue, MoDOT, used with permission (left), Kevin Merryman, Iowa DOT, used with permission (right)

Figure 7.16. Rigid (left) and corrugated (right) lateral outlet pipes

Step 4: Headwalls and Outlet Pipes

Placing the lateral outlet pipe, constructing the headwalls, and marking the outlet drains with outlet markers are the final steps in the installation of the pipe edgedrain system. When placing the outlet pipe, it is important to avoid high or low spots in the outlet trench and to make sure that the exposed end is not turned upward or otherwise elevated. Examples of outlets are shown in Figure 7.16.

Precast headwalls are recommended to prevent clogging and damage from mowing operations. A rodent screen or wire mesh placed over the ends of the pipe should also be used to keep small animals out. Figure 7.17 shows several different types of headwall installations.

Kevin Merryman, Iowa DOT, used with permission

Figure 7.15. Automated equipment installing CPE pipe edgedrains

John Donahue, MoDOT, used with permission (left) and ©2022 Applied Pavement Technology, Inc., used with permission (center and right)

Figure 7.17. Various headwall installations

Geocomposite Edgedrains

Step 1: Trenching

For geocomposite edgedrains, the trench should be cut 4 to 6 in. wide and deep enough to place the top of the panel drain 2 in. above the bottom of the pavement surface layer. Typical dimensions for a geocomposite edgedrain consist of an inside cross-sectional thickness of 0.5 to 1 in. and a depth of 12 to 18 in.

Step 2: Installation of the Geocomposite Edgedrain

As shown in the previous Figure 7.4, the edgedrain should be placed on the shoulder side of the trench, and the trench should be backfilled with coarse sand to ensure intimate contact between the geotextile and the material being drained. Achieving this contact is very important to prevent loss of fines through the geotextile.

Maintaining the verticality of the drain panel in the trench during the backfill operation is also important (Elfino et al. 2000).

When required, splices should be made prior to placing the drain in the trench and using the splice kits provided by the manufacturer. The splice should not impede the open flow area of the panel. Vertical and horizontal alignment of the drain should be maintained throughout the splice, and the splice should not allow infiltration of backfill or any fine material.

Step 3: Headwalls and Outlets

Prior to any backfilling, the PGEDs should be connected to drainage outlets. As with pipe edgedrains, it is recommended that headwalls be used on the outlets to prevent clogging and damage from mowing operations. Finally, all outlet drains should be clearly marked with outlet markers.

Step 4: Backfilling

For geocomposite edgedrains, excessive compaction during the backfilling process can cause problems. Excessive compactive forces can cause crushing and buckling of the geocomposite edgedrain panels, so the use of vibrating plates and compactors should be done carefully.

Coarse sand, placed in 6 in. lifts, has been successfully used as backfill material and compacted by flushing or puddling with water (Koerner et al. 1994). If coarser aggregates are used, the maximum aggregate size should be limited to 0.75 in. to enhance placement around the PGED. The cuttings from the drainage trench are not a suitable backfill material when installing a geocomposite edgedrain.

If the panel design is not symmetrical about the vertical axis, the panel should be installed with the rigid or semi-rigid back facing the sand backfill (Fleckenstein et al. 1994).

Aggregate Drainage Systems

Aggregate drainage systems are physically cut in at the pavement-shoulder interface using a trencher or backhoe. The dimensions for this type of drainage system vary, but the trench is often about 12 in. wide and at least 8 in. deep, although the depth will depend on the pavement and shoulder thickness and underlying base course thickness.

It is generally desired that the bottom of the aggregate drain be located at or below the bottom of the pavement aggregate subbase at the point of contact, while the top of the aggregate drain be no higher than the bottom of the shoulder's aggregate base at the point of contact. The trench should be sloped to the ditch at a grade of at least 8%.

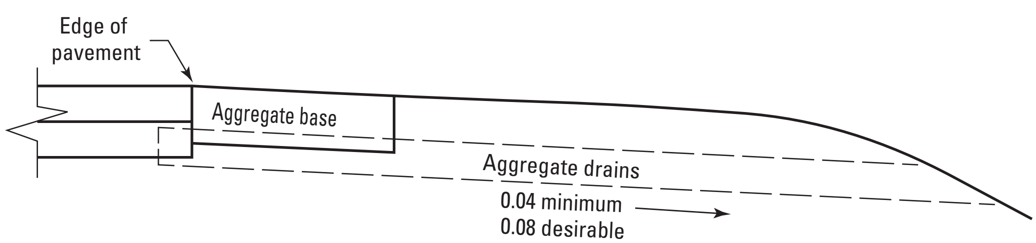
Figure 7.18 illustrates an aggregate drainage system design from the Ohio Department of Transportation (ODOT) that specifies a minimum slope of 0.04 in./in. (4%) and a desirable slope of 0.08 in./in. (8%).

After the trench is cut, it is recommended that it be lined with an appropriate geotextile material to prevent the migration of fines. Sufficient geotextile material should be placed so that it can totally encapsulate the aggregate material. The specified aggregate drainage material is then placed in the trench in no more than about 6 in. lifts to ensure adequate compaction. The geotextile material is then wrapped over the top of the aggregate base, and the top of the trench is covered with earthen backfill.

6. Maintenance

Neglected and poorly maintained drains can be worse than having no drains at all. It cannot be overemphasized that all subdrainage features, whether installed during initial construction or retrofitted, must be adequately maintained to perform properly. Some of the problems that can occur over the life of a drainage system include the following (Christopher 2000):

- Crushed or punctured outlets
- Outlet pipes that are clogged with debris, rodent nests, mowing clippings, vegetation, and/or sediment (see Figure 7.19)
- Edgedrains (both pipe drains and fin drains) that are filled with sediment, especially for slopes of less than 1%
- Missing rodent screens at outlets
- Missing outlet markers
- Erosion around outlet headwalls
- Shallow ditches that have inadequate slopes and that are clogged with vegetation



Adapted from ODOT 2020

Figure 7.18. Example of aggregate drainage system from the ODOT

John Donahue, MoDOT, used with permission

Figure 7.19. Clogged edgedrain outlet pipes

Ashurst and Rister (2019) performed a comprehensive study of edgedrain performance in Kentucky. The project involved the inspection of 10 roadway segments, including the assessment of several components of their edgedrain systems. For all 126 headwall installations inspected, the researchers found that the headwalls were in good condition and free of structural issues. However, roughly 29% of the outfall waterways prevented the flow of water from the headwall (see Figure 7.20); 65% of the outlet waterways were blocked to some extent by gravel, mud, silt, or other debris (see Figure 7.21); and 61% of the outlet pipes were otherwise obstructed.

Rodent screens were either missing or not functioning in 25% of the inspected locations (see Figure 7.22).

Ashurst and Rister 2019, Kentucky Transportation Institute

Figure 7.20. Outfall waterway obstructed by vegetative growth

Ashurst and Rister 2019, Kentucky Transportation Institute

Figure 7.21. Outlet waterway blocked by gravel

Ashurst and Rister 2019, Kentucky Transportation Institute

Figure 7.22. Rusted, nonfunctioning rodent screen

Approximately 75% of the problems found during inspections in this study were related to maintenance, with the remainder related to construction.

Adequate maintenance should begin in the design stage so that when a system is constructed, it can be adequately maintained. Examples of design considerations include using independent dual pipe outlets to facilitate maintenance, specifying the placement of outlet markers 24 to 36 in. above the ground and suitably marked to locate transverse outlets, installing concrete headwalls with permanent anti-intrusion protection (screens), and specifying proper connectors to accept periodic flushing or jet rodding of the edgedrain system. Permanent markers on the pavement, backslope or foreslope, and concrete headwalls also serve as a reminder of the existence of the system and the need for its maintenance.

It is recommended that routine drainage-related maintenance activities be conducted at least twice a year. Examples of some of these maintenance activities include the following:

- Mowing around drainage outlets
- Inspection of the drainage outlets and flushing if necessary
- Removal of vegetation and roadside debris from pipe outlets, daylighted edges of the granular base, and ditches
- Replacement of missing rodent screens, outlet markers, and eroded headwalls
- Inspection of ditches to ensure that adequate slopes and depths are maintained. (It is generally recommended that roadside ditches be 3 to 4 ft wide, have a depth 4 ft below the surface of the pavement, and have a minimum longitudinal slope of 1%.)

Video cameras are a valuable tool that can be used to inspect the condition of drainage systems. Since first promoted by the FHWA in the late 1990s, more than 17 state DOTs have reported using a video camera for routine inspection of drainage systems, for investigation of potential drainage issues, or as an acceptance item after the system has been installed (Christopher et al. 2006).

A study by Daleiden (1998) that involved conducting video inspections of in-service edgedrains to assess their performance revealed that only 30% of the in-service edgedrains examined were fully functional.

The common causes for the poor performance of retrofitted pipe edgedrains were discovered to be improper installation, pipe clogging due to fines, and pipe crushing. For PGEDs, the major causes were improper installation (e.g., crushed or buckled geocomposite panels) and clogging caused by the caking of fines on the geotextile material.

After implementing a video camera inspection quality control program, the Kentucky Transportation Cabinet determined that the number of edgedrain failures had decreased from 20% to less than 2% (Fleckenstein and Allen 2000). Figure 7.23 shows a video camera system.

Even when all design parameters have been properly evaluated and included in the design, the effect of retrofitted subdrainage on pavement performance may not be as expected, and its potential benefits discussed earlier may not be attainable. An evaluation program that provides feedback data will enable agencies to assess the effectiveness of their edgedrain installations and make design improvements for future projects. However, such evaluation programs must not involve only short-term evaluations because many moisture-related distresses take time to develop.

ERES Consultants, Inc. 1999, NHI

Figure 7.23. Video system (left) and camera head (right) for drainage system inspection

7. Summary

Pavement engineers are often faced with older concrete pavements that are displaying moisture-related damage, which may be attributed to factors such as inadequate initial drainage, damage to existing subsurface drainage systems, or inadequate drainage system maintenance. To address these drainage-related problems, one rehabilitation option is the retrofitting of the existing pavement with edgedrains.

To date, the field performance of retrofitted edgedrains has been mixed, ranging from reducing pavement deterioration to having a detrimental effect on a few projects. The cases of poor retrofitted edgedrain performance have generally been attributed to inappropriate use, improper construction or installation, or lack of maintenance.

Complicating matters is that design engineers of retrofitted edgedrain systems must work with existing pavement materials, which may have limited drainability. Nevertheless, there still may be some benefit to using retrofitted edgedrains because they can remove water that enters at the lane-shoulder joint. In the end, an agency must determine the benefit of installing subsurface drainage based on local conditions, experience, and practices.

The installation of retrofitted edgedrains should be considered on projects in which the following conditions are met:

- The primary source of the water affecting pavement performance is surface infiltration.
- The pavement is less than 15 years old.
- The base material has less than 15% of its material passing the #200 sieve.
- The pavement is in relatively good condition (i.e., there are limited signs of severe moisture damage and less than 10% of pavement slabs are cracked).

A variety of edgedrain systems have been used on retrofitted drainage projects, with each having slightly different characteristics. Prefabricated geocomposite edgedrains and aggregate drainage systems are less expensive to install than pipe drains but can be difficult to maintain (i.e., they are nearly impossible to clean if they become clogged). Typically, geocomposite edgedrains also have lower hydraulic capacities than pipe drains, although newer materials are changing this trend.

Pipe edgedrains, on the other hand, generally have higher hydraulic capacities than aggregate or geocomposite edgedrain systems but cost more to install. Regardless of the type of drainage system chosen, however, the proper construction and installation of these systems are important to ensure their long-term effectiveness.

The edgedrain performance experiences of several state transportation departments highlight the need for regular maintenance of edgedrain systems. This begins with regular inspection and monitoring and includes such items as installing and maintaining reference markers at outlet locations, clearing debris and vegetation at outlets, and flushing/rodding the edgedrain system as needed. In addition, video cameras for the inspection of drain conditions have proven to be a valuable tool in the monitoring of edgedrain effectiveness.

Table 7.1 summarizes some of the critical considerations in the selection, design, construction, and maintenance of retrofitted edgedrain systems.

Table 7.1. Summary of critical considerations for retrofitted edgedrains

Phase of project	Consideration
Project selection	• Retrofitted edgedrains are most appropriate for an existing pavement with moisture-related distresses (e.g., pumping and/or faulting) but little cracking or other signs of structural deterioration (i.e., less than 10% of slabs exhibit cracking) when the agency desires to extend the service life of the pavement for several years after restoration. • The existing base should have less than 15% fines (material passing the #200 sieve). • The geometrics of the project must be acceptable in terms of the transverse and longitudinal slopes. • Retrofitted edgedrains may be used only in localized areas where specific moisture problems exist (versus over an entire project).
Design	• Anticipated water outflow levels that can realistically be removed from the pavement must be determined (e.g., via the FHWA's DRIP program). Inputs such as rainfall intensity may be determined by consulting data sources such as the Modern-Era Retrospective analysis for Research and Applications (MERRA) (NASA 2017). Designers should also consult with their agency's hydraulics staff for the latest guidance. • Geotextile material should be selected based on the base and subgrade materials. • Edgedrains must be properly sized and placed in the proper location (in terms of horizontal offset and vertical position). • Effective backfill material with proper gradation for the existing pavement must be selected. • Outlet spacing should be determined based on projected outflow and slopes of the project (typical spacing is 250 to 300 ft). • Proper elbow radii must be selected for outlet pipes to facilitate cleaning.
Construction	• Pipe drains must be placed in the proper vertical and horizontal position, while PGEDs should be placed against the shoulder side of the trench. • Aggregate drains should be placed at or below the bottom of the pavement base. • Backfill material must be placed to avoid damaging pipe drains or PGEDs and carefully compacted; a minimum of 6 in. cover is recommended over drainage pipes before compacting. • Rigid outlet pipes should be installed, hooked up to collector pipes, and placed at least 6 in. above the 10-year ditch flow line (or 10-year water level in the storm drain system). • Headwalls should be installed for each outlet location.
Maintenance	• Drainage outlets must be marked, mowed around, and regularly inspected for condition and functionality. • Headwalls must be inspected and maintained. • Video inspection of pipe drains must be performed regularly and the system flushed as needed. • Vegetation and debris must be removed from pipe outlets, daylighted edges, and ditches. • Ditches should have adequate slopes and depths; it is generally recommended that roadside ditches be 3–4 ft wide, have a depth 4 ft below the surface of the pavement, and have a minimum longitudinal slope of 1%.

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Chapter 8

Dowel Bar Retrofit, Cross-Stitching, and Slot-Stitching

1. Introduction

Dowel bar retrofit is the installation of dowel bars in slots across existing transverse joints or cracks to increase their ability to effectively transfer wheel loads across slabs, reduce deflections, and prevent faulting. The existing concrete must be sound—the presence of any deterioration in the lower portions of the slab on a given project renders DBR an unsuitable treatment for that project. Figure 8.1 shows an example of DBR on a concrete pavement project.

Doweled concrete pavements normally exhibit adequate load transfer, but nondoweled JPCPs typically show lower levels of load transfer because they rely only on the aggregate interlock of the abutting joint faces for load transfer. Aggregate interlock is only effective if the opposing joint faces remain in close contact, as the degree of interlock and shear capacity decreases rapidly as joint openings increase above 0.03 in. (FHWA 2019a). Transverse cracks in both JPCP and JRCP also rely on aggregate interlock for good performance and may exhibit poor load transfer if aggregate interlock is not maintained.

Concrete pavements with poor load transfer that are subjected to heavy traffic can rapidly develop pumping, loss of support, corner breaks, and faulting, all of which significantly detract from the performance of the pavement. Although diamond grinding and patching could be done to help maintain these pavements, the root cause of the distresses—poor load transfer—is not addressed by these treatments, so deterioration will still continue. By instead providing load transfer through the retrofit installation of dowel bars, however, the source of the deterioration is addressed and the performance capabilities of such pavements are restored. (It should be noted that, after dowel bar retrofitting, diamond grinding of the pavement surface is commonly also done to restore rideability.)

Two additional preservation techniques related to DBR—cross-stitching and slot-stitching—are also presented in this chapter. Cross-stitching and slot-stitching are preservation methods designed to strengthen nonworking (i.e., not opening and closing) longitudinal joints and cracks that are in relatively good condition (IGGA 2010). Cross-stitching includes drilling holes at an angle through a nonworking longitudinal joint or crack and epoxying or grouting a deformed tie bar into the drilled hole. Slot-stitching, on the other hand, is similar to DBR except that a deformed tie bar is grouted into slots cut across a nonworking longitudinal joint or crack to hold it together. Currently, cross-stitching is more commonly used than slot-stitching.

WSDOT, used with permission

Figure 8.1. Completed DBR project

This chapter presents information on the application and installation of the DBR, cross-stitching, and slot-stitching preservation techniques. The focus of the chapter is on DBR, but separate sections are included at the end of the chapter on cross-stitching and slot-stitching.

2. Purpose and Project Selection

Load Transfer Efficiency

In order to select good candidate projects for DBR, it is first important to understand the concept of LTE and how it is measured. LTE is a quantitative measurement of the ability of a joint or crack to transfer load from one side to the other. It may be defined in terms of either deflection load transfer or stress load transfer. Deflection LTE is more commonly used because it can be easily measured for existing pavements using an FWD. The most common mathematical formulation for expressing deflection LTE is as follows:

$$LTE = \frac{\Delta_{UL}}{\Delta_L} \times 100 \qquad \textbf{(8.1)}$$

Where:

LTE = Load transfer efficiency, percent

Δ_{UL} = Slab deflection on the unloaded side of the joint

Δ_L = Slab deflection on the loaded site of the joint

The concept of deflection load transfer is illustrated in Figure 8.2.

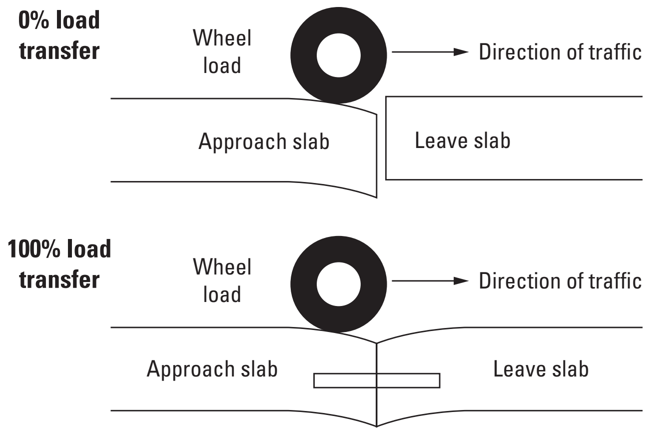

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Figure 8.2. Deflection load transfer concept

If no load transfer exists, then the unloaded side of the joint experiences no deflection when the wheel is applied on the approach side of the joint, and the LTE computed from Equation 8.1 is 0%. If perfect load transfer exists, both sides of the joint experience the same magnitude of deflection under the wheel loading, and the LTE computed from Equation 8.1 is 100%. Dowel bars are a proven and effective means of providing high levels of load transfer.

LTE should be measured during cooler temperatures (ambient temperatures of less than 70°F) and during the early morning when the joints are not tightly closed. In addition, LTE must be determined using a device such as the FWD that is capable of applying loads comparable in magnitude and duration to that of a moving truck-wheel load. LTE is generally measured either in the outer wheel path, which is subject to higher repeated truck-traffic load applications, or at the slab corner, which is the location with the highest deflection potential and is often considered the more critical location. Deflection measurements for the determination of LTE should be taken with sensors placed as closely to the joint or crack as possible and evenly spaced on either side.

The magnitude of the deflections should be considered in addition to the LTE. This is because it is possible for slabs to exhibit very high deflections yet still maintain a high LTE. In this case, even though the LTE is high, the large deflections can nevertheless lead to pumping of the underlying base course material, faulting, and perhaps even corner breaks. Similarly, a low LTE may not be significant if the magnitudes of the deflections are low.

A useful parameter to help assess deflection magnitude is the differential deflection (DD), which is the relative displacement between the loaded and unloaded sides of a joint and is computed as follows:

$$DD = \Delta_L - \Delta_{UL} \quad \textbf{(8.2)}$$

The DD should be computed along with the LTE over the entirety of a project to gain a more complete understanding of the load transfer characteristics of the project's joints and cracks. In assessing DD, it is desirable that DDs be limited to 5 mils (0.005 in.) or less (Odden et al. 2003, Snyder 2011) and that peak corner deflections be limited to 25 mils (0.025 in.) or less (Snyder 2011).

Selecting Candidate Projects for Dowel Bar Retrofit

Key to the success of DBR is the selection of an appropriate project. The following are general characteristics associated with good candidate pavements for DBR (ACPA 1998, Caltrans 2015):

- Pavements with structurally adequate slab thickness but exhibiting low load transfer due to the lack of dowels and poor aggregate interlock
- Relatively young pavements in good condition but with the potential to develop faulting, working cracks, and corner cracks unless load transfer is improved

On the other hand, pavements exhibiting significant slab cracking, joint spalling, or MRD, such as alkali-silica reactivity or D-cracking) should not be considered candidates for DBR. It is essential that sound concrete exists throughout the depth of the slab, and coring may be needed to ensure that this is the case (Darter 2017).

Recommendations for determining the suitability of DBR for a given project include the following:

- Deflection load transfer of 60% or less, faulting greater than 0.10 in. but less than 0.25 in., and differential deflections of 0.01 in. or more (ACPA 1998)
- Pavements between 25 and 35 years old that exhibit average faulting of less than 0.125 in. (WSDOT 2018)
- Projects with less than 70% load transfer and exhibiting either less than 2% third-stage cracking (i.e., panels cracked in three or more pieces) or between 2% and 5% third-stage cracking but faulting less than 0.6 in. (Caltrans 2015)

Dowel bar retrofit may also be used in other applications, including at transverse cracks (if the cracks are fairly uniform and have not widened or faulted excessively) or in preparation for an overlay. In the former application, DBR helps to maintain structural integrity and improves ride quality at a fraction of the cost of an FDR, whereas in the latter application, DBR helps to reduce the incidence and severity of reflection cracking, spalling, and other deterioration of the overlay (which may therefore enable a thinner overlay design). Figure 8.3 shows a DBR installation through a transverse crack in a concrete pavement.

Figure 8.3. Dowel bar retrofit at a transverse crack

3. Limitations and Effectiveness

Since its introduction in the 1990s, DBR has become an established pavement preservation technique used by many state transportation departments. As of 2017, at least 26 state transportation departments had demonstrated experience with DBR (IGGA 2017). Performance expectations vary, but most agencies anticipate a minimum service life of 10 to 15 years (Caltrans 2015, WSDOT 2018). That said, there are numerous examples of DBR projects that have achieved much longer service lives; for example, several projects in Washington State are performing well after 20 or more years of service and DBR projects in Minnesota and Utah are expected to last 20 years or more (Darter 2017).

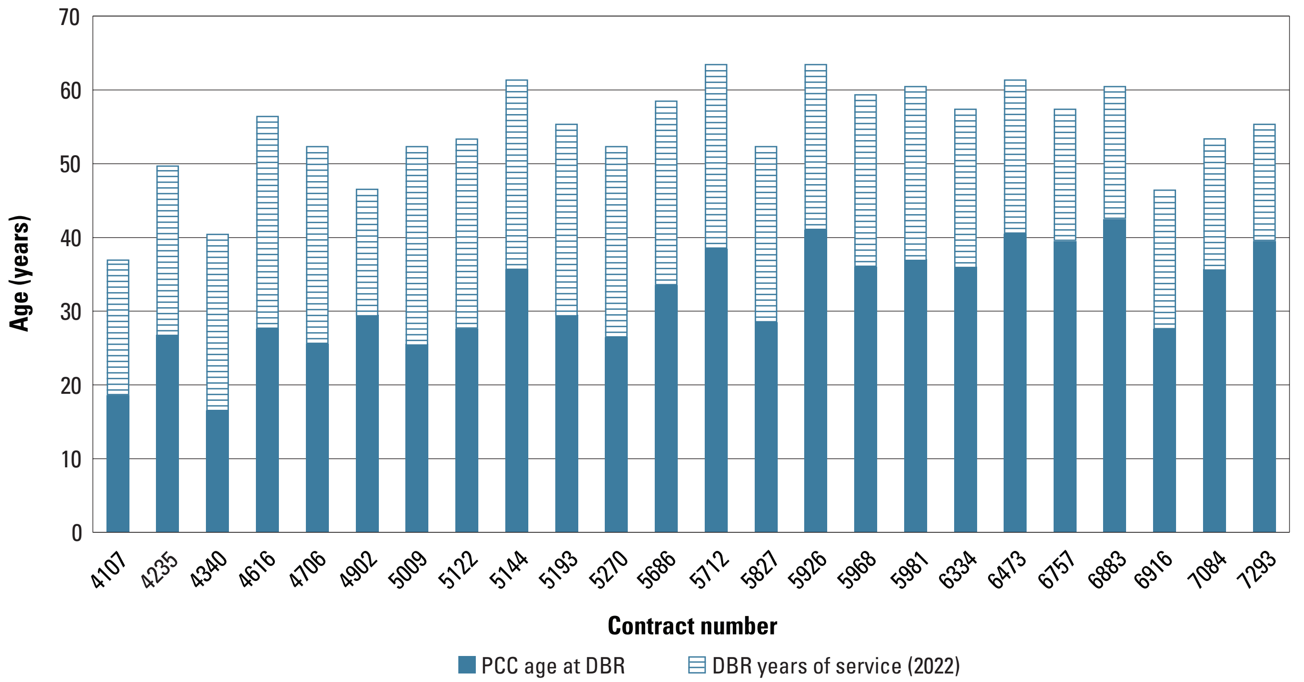
The Washington State Department of Transportation (WSDOT) has a DBR program dating back to 1992 (Pierce 1994, Pierce 1997) and has periodically monitored the performance of its projects. In a 2008 review of approximately 180 lane miles of retrofitted concrete pavement (representing approximately 380,000 dowel bar slots), it was noted that less than 10% of the DBR slots exhibited any form of distress (after 2 to 14 years of service) (Pierce and Muench 2009).

In 2022, the WSDOT reviewed the performance of its DBR projects that were constructed before 2007 (see Figure 8.4), representing about 270 lane miles of pavement. This WSDOT assessment revealed outstanding performance of these DBR projects, with only four projects taken out of service (three projects reconstructed and one overlaid) during the evaluation period. The service life of the DBR-rehabilitated pavements ranged from about 15 to 28 years, with the average age of the in-service concrete pavements reaching about 54 years as a result of the DBR installations.

From 1994 to 1999, MnDOT constructed several different test sections for the evaluation of dowel bar length, configurations, patching materials, and overall effectiveness. The results of these studies indicated that DBR is effective in preventing the faulting of midpanel cracks and in extending the service life of nondoweled concrete pavements (Burnham and Izevbekhai 2009).

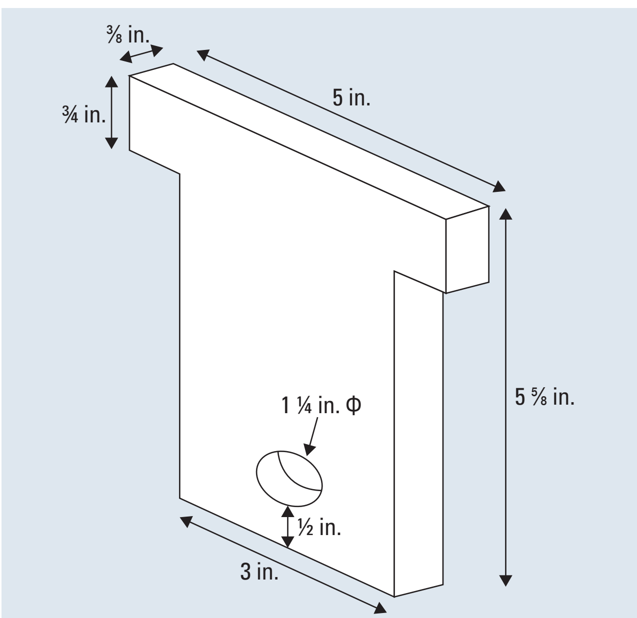
Although there has been good documented success with the DBR technique, it should be noted that a few states have experienced a few issues, particularly with backfill materials. For example, on some of its early projects, WisDOT found that the patching material used to backfill the DBR slots was deteriorating at some of the joints (Bischoff and Toepel 2002). In response to these observed material problems, WisDOT developed modified patching materials to reduce unwanted shrinkage (Bischoff and Toepel 2004). Similarly, the California Department of Transportation (Caltrans) reported poor materials and workmanship on several early DBR projects (Shatnawi et al. 2009).

Along the same lines, the North Dakota Department of Transportation (NDDOT) constructed several DBR test sections and identified distress within the dowel bar slots that appeared to be related to shrinkage cracking, lack of bond, movement of the foam core board, or lack of consolidation of the patching materials (Pierce 2009). Because of contractor challenges in keeping the foam core boards vertical within the dowel bar slots, the NDDOT now requires the use of a notched foam core board insert that is configured to fit over a dowel bar in order to keep it in alignment with the transverse joint (see Figure 8.5). A notched foam core board is also required by Caltrans and the Idaho Transportation Department (ITD).



Adapted from WSDOT, used with permission

Figure 8.4. Performance of DBR projects in Washington



Adapted from Dunn and Fuchs 1998, NDDOT (top); WSDOT, used with permission (bottom)

Figure 8.5. Typical notched foam core board insert (top) and dowel bar assemblies with foam core board inserts (bottom)

As highway agency experience has shown, it is important that the DBR treatment be targeted to the proper pavement (i.e., one that is not exhibiting significant structural deterioration). Furthermore, the installation of DBR must be viewed as a system, with each part of that system—from the slot preparation to the materials to the patching and consolidation—being critical to the long-term performance of DBR projects. Also, for maximum effectiveness, the application of additional preservation treatments, such as crack sealing, joint resealing, and slab stabilization, should be considered.

4. Materials and Design Considerations

When designing a DBR project, it is important to select an appropriate patching material to fill the slot and to determine the proper size (diameter and length) and configuration (number of slots) for the dowel bars. This section provides recommendations for some of these key factors.

Patching Material

The patching material (sometime referred to as the backfill material) is the substance used to encase the dowel bar after it has been placed in the slot in the existing pavement. Various materials can be used for this application, including conventional concrete mixtures and other cementitious and noncementitious, rapid-setting proprietary products. The selection and installation of an appropriate patching material are factors critical to the performance of a DBR project (ACPA 2006). Desirable properties of the patching material include little or no shrinkage, thermal compatibility with the surrounding concrete (e.g., similar coefficients of thermal expansion), good bond strength with the existing (wet or dry) concrete, and the ability to rapidly develop sufficient strength to carry the required load so that traffic can be allowed on the pavement at the specified time (ACPA 1998).

Many agencies maintain a qualified products list (QPL) of suitable patching materials for PDRs, and these also work well in DBR applications. Table 8.1 summarizes recommended tests and material properties for DBR patching materials, based on a summary of agency specifications.

For cementitious products, one of the most important factors to control is the water content of the patching material to reduce the probability of shrinkage cracks and debonding (Rettner and Snyder 2001). ASTM C928, Standard Specification for Packaged, Dry, Rapid-Hardening Cementitious Materials for Concrete Repairs, governs the use of packaged cementitious repair materials for concrete repairs, including materials for use in DBR patching.

The patching material should be extended with fine and coarse aggregate. Local requirements for concrete sand can be used for the fine aggregate portion, and the coarse aggregate should also meet local concrete aggregate quality requirements (IGGA 2013). In addition, the coarse aggregate gradation should meet the following sieve size requirements (IGGA 2013):

- 100% passing the ⅜ in. sieve
- 0% to 15% passing the #4 sieve
- 0% to 5% passing the #8 sieve
- 1.0% (maximum) passing the #200 sieve

A special consideration in certain parts of the country is the presence of traffic with studded tires. In these areas, the patching material should incorporate a hard aggregate so it can better resist wear (Darter 2017).

Concrete Materials

Concrete can be used as a patching material for DBR. It is cheaper than proprietary materials, is widely available, and presents no thermal compatibility problems with its use. Many mixes use Type III cement and an accelerator to improve setting times and reduce shrinkage. Sand and an aggregate with 0.375 in. maximum size are commonly used to extend the yield of a mix. It is important to maintain the proper w/cm ratio for mixtures and to recognize that high cement factors can lead to excessive shrinkage.

Rapid-Setting Proprietary Materials

Rapid-setting proprietary materials are the predominant patching material used by state transportation departments for DBR projects and feature a range of products, including rapid-hardening concretes, calcium sulfoaluminate, and polyester concrete. The main advantage of these types of materials is that they are quick setting, thereby allowing earlier opening times to traffic. It is critical that all manufacturer's instructions be followed when working with these proprietary materials (ACPA 2006).

Epoxy-Resin Adhesives

Epoxy-resin adhesives have been used to improve the bond between the existing concrete and the patching materials. If used, epoxy-resin adhesives should meet the requirements of AASHTO M 235, and the manufacturer's recommendations should be closely followed for application and placement. It is also critical that the adhesive not be allowed to dry prior to the placement of the repair material, since this could lead to premature debonding of the patch.

Table 8.1. Recommended properties for patching materials

Property	Test procedure	Recommended value
Compressive strength	AASHTO T 160 ASTM C109	>3,000 lbf/in² @ 3 hours >5,000 lbf/in² @ 24 hours
Scaling	ASTM C672	Visual rating of 2 or less
Shrinkage	ASTM C157	<0.13% @ 4 days
Durability factor	AASHTO T 161 ASTM C666A	>90% @ 300 cycles
Bond strength	ASTM C882	>1,000 lbf/in² @ 24 hours

Source: IGGA 2013

A summary of the repair materials used by selected state transportation departments is presented in Table 8.2, illustrating the common use of proprietary materials with fast-setting characteristics.

Table 8.2. Summary of selected state transportation department repair materials

State DOT	Patching material
California	Polyester concrete
Iowa	Rapid-hardening concrete
Minnesota	Rapid-hardening concrete
Missouri	Rapid-hardening concrete
Utah	Rapid-hardening concrete
Washington	Prepackaged mortar + aggregate

Source: After Darter 2017

Dowel Bar Design and Layout

Round, solid steel dowels conforming to AASHTO M 31 or ASTM A615 are commonly used for load transfer in concrete pavements. These bars characteristically have a fusion-bonded epoxy coating (typically between 0.008 to 0.012 in. thick) that provides corrosion protection by acting as a barrier against moisture and chloride intrusion, although other materials (e.g., stainless steel, low-carbon chromium, and zinc) are also used by some agencies for specific applications and exposure conditions (WSDOT 2018). In addition, the dowels must be coated with a bond breaker to allow the joint to open and close in response to changing temperatures.

The diameter of the dowel bar for the DBR project is an important design consideration. Larger diameter bars are extremely effective in preventing or minimizing faulting, and for most high-type, heavy truck–trafficked roadways, the use of 1.5 in. diameter dowel bars is suggested. Thinner pavements can employ smaller-sized bars, as shown in Table 8.3.

The embedment length of the dowel (i.e., the length of bar on either side of the joint) is generally desired to be 6 in., which, when accommodating the expansion caps on either end of the dowel and acceptable placement tolerances, leads to a minimum dowel bar length of 14 in. (ACPA 2006). However, some agencies continue to use standard 18 in. long dowel bars, particularly on thicker slabs or when the DBR is performed on transverse cracks.

For retrofitted dowel bars to be effective, they must not only be of sufficient size but also placed in a suitable configuration. Over the years, agencies have used between three and five dowel bars in each wheel path, with many agencies now specifying three dowel bars per wheel path (Darter 2017). A typical dowel bar configuration is presented in Figure 8.6, showing three dowel bars in each wheel path spaced 12 in. apart. For the outside dowel, both the Missouri (MoDOT 2020) and Washington State (WSDOT 2019) DOTs specify that it be located 18 in. from the outside slab edge; this helps to eliminate the potential for random cracking in the area of the dowel (Darter 2017).

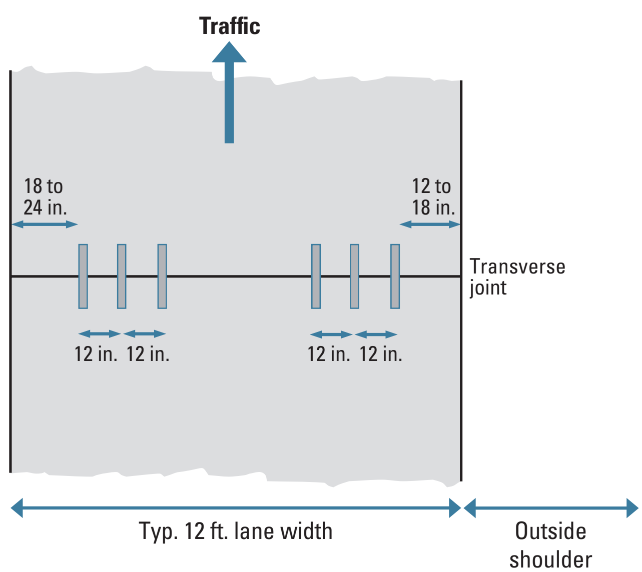


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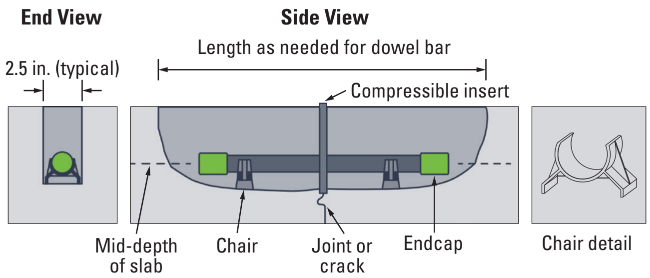
Figure 8.6. Typical DBR configuration

Table 8.3. Recommended dowel dimensions for DBR

Pavement thickness (in.)	Diameter (in.)	Minimum length (in.)	Spacing (in.)
≥7 to <8	1.0	14	12
≥8 to <10	1.25	14	12
≥10	1.5	14	12

Source: Adapted from ACPA 2006



Adapted from IGGA, used with permission

Figure 8.7. Dowel bar retrofit slot details

A final design consideration is the dimensions of the DBR slots themselves. The slots must be sufficiently long to accommodate the full length of the dowel bar (such that each dowel bar lies flat across the bottom of its respective slot without hitting the end of the slot) and to account for the curvature of the sawcut used to create each slot (see Figure 8.7). At the same time, the created slot should be deep enough so that the dowel is positioned at the mid-depth of the slab, allowing a clearance of approximately 0.5 in. beneath the dowel bar to accommodate its placement on chairs. Each DBR slot is typically 2.5 in. wide and its bottom should be flat and uniform across the joint. When DBR is used as a preservation treatment for thinner slabs, it may be necessary to move the bar slightly above mid-depth to minimize the potential for punching out the bottom of the existing slab while at the same time still maintaining a minimum concrete cover of 2 in.

5. Construction Considerations

The completion of a DBR project involves the steps listed below, which are described in more detail in the following sections:

- Step 1: Test section construction and evaluation
- Step 2: Slot creation
- Step 3: Slot preparation
- Step 4: Dowel bar placement
- Step 5: Patching material placement
- Step 6: Diamond grinding (optional)
- Step 7: Joint sealing

Step 1: Test Section Construction and Evaluation

Agencies should consider requiring the contractor to construct a test section to demonstrate their capabilities in constructing a DBR project. The test section should incorporate all phases of the DBR construction process, from concrete sawing and removal to dowel bar placement, and from patching material placement to consolidation, finishing, and curing. Details of the test section may include the following (IGGA 2013):

- **Test section layout and dimensions**—The test section construction should be performed in one lane and should include at least 20 joints or cracks using the prescribed slot layout pattern and dimensions, patching materials, and procedures.
- **Evaluation**—After construction of the test section, the contractor should extract three full-depth cores (minimum of 4 in. diameter) to assess the completeness of slot removal, the effectiveness of the dowel bar installation, and the level of consolidation achieved with the patching material. In addition, agencies may require that FWD testing be performed to verify the effectiveness of the DBR installation. If the test section does not conform to the plans and specifications, the contractor will be required to construct an additional test section.

Step 2: Slot Creation

The recommended method of creating slots for DBR projects is with a diamond-bladed slot-cutting machine (see Figure 8.8).

WSDOT, used with permission

Figure 8.8. Diamond-bladed slot-cutting machine

Modified milling machines are not recommended to create the slots as they produce excessive spalling and do not provide consistent slot dimensions (ACPA 2001a). Slots should not be cut within 15 in. of an existing crack, as this could lead to additional cracking and deterioration.

Diamond-bladed slot-cutting machines make a series of parallel cuts in the pavement for each dowel slot; the “fin” area between the cuts is then broken up with a light jackhammer. Examples of one-slot and three-slot cutting machines are shown in Figures 8.9 and 8.10, respectively. Production rates for the multiple-slot equipment can exceed 2,500 slots per day.

Since misaligned dowels can potentially cause joint/crack lockup that will lead to slab cracking, it is important that dowel bar slots be parallel to the centerline of the pavement. It is also important that dowel bar slots be cut to the prescribed depth, width, and length at the required spacing. Figure 8.11 shows sawcuts for dowel bar slots.

Step 3: Slot Preparation

After the sawcuts have been made, lightweight jackhammers (less than 30 lb) or hand tools are used to remove the concrete in each slot. Jackhammers should be operated at a 45-degree angle or less to decrease the chance of the jackhammer punching through the bottom of the slot (see Figure 8.12).

WSDOT, used with permission

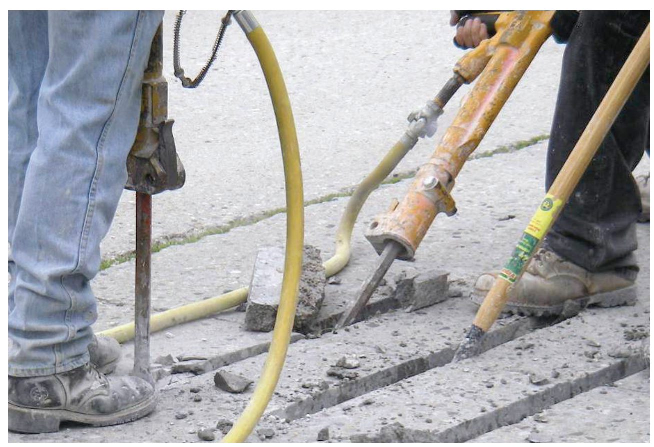
Figure 8.9. Equipment to cut a single dowel bar slot

WSDOT, used with permission

Figure 8.10. Equipment capable of cutting three dowel bar slots at once

WSDOT, used with permission

Figure 8.11. Sawcuts for dowel bar slots across transverse joints



WSDOT, used with permission

Figure 8.12. Operating jackhammers at no more than a 45-degree angle

After removing the concrete wedge, the bottom of the slot should be flattened with a small hammerhead mounted on a small jackhammer, as shown in Figure 8.13.

Once the jackhammering operations are completed, media blasting is performed on the slots to remove the dust and sawing slurry and to provide a slightly roughened surface to promote bonding. When performing media blasting, workers should follow all safety requirements as outlined by OSHA in 29 C.F.R. § 1926.1153 (2016).

Final cleaning through air blasting is performed on the slot immediately before the dowel and patching material are placed. Figure 8.14 shows the media-blasting and air-blasting operations, with a prepared slot ready to receive the dowel and repair material shown in Figure 8.15.

The side of the dowel bar slot is considered clean when wiping the sides of the slot with a clean towel reveals no residue (Pierce et al. 2009).

After cleaning and prior to dowel bar placement, the joint or crack in the slot is caulked with an approved sealant material (see Figure 8.16) to prevent intrusion of any patching material into the joint that might cause a compression failure.

The sealant should not excessively cover the slot sidewalls away from the joint, as this could prevent the patching material from effectively bonding to the existing slab.

WSDOT, used with permission

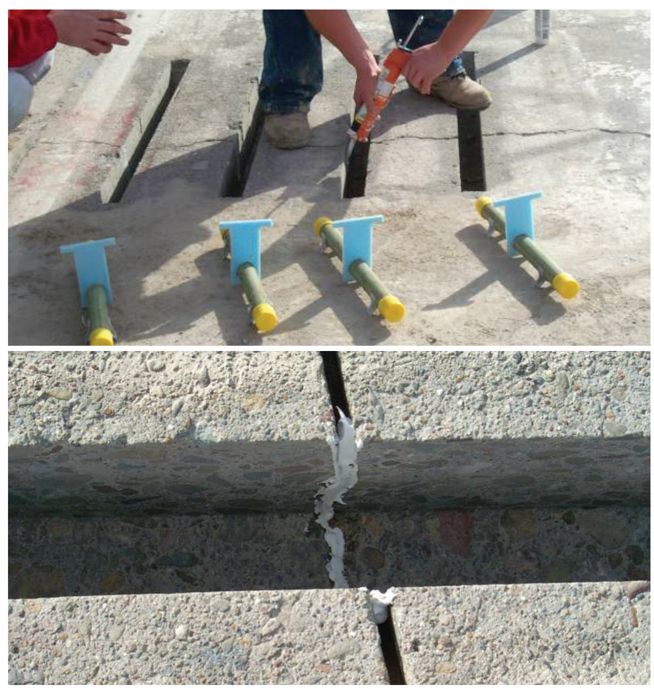
Figure 8.13. Leveling the bottom of the dowel bar slot

WSDOT, used with permission

Figure 8.14. Media blasting to remove residue (left) and air blasting immediately prior to dowel placement (right)

WSDOT, used with permission

Figure 8.15. Media-blasted and cleaned slot



WSDOT, used with permission

Figure 8.16. Applying caulk to dowel bar slot sides and bottom (top) and caulked pavement joint (bottom)

Step 4: Dowel Bar Placement

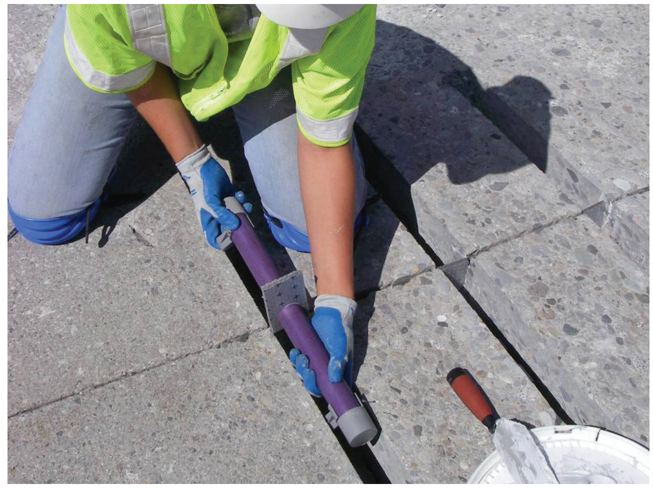
The dowel bars should be coated with a bond-breaking material along their full length to facilitate joint movement. Expansion caps are placed at both ends of each dowel to allow for any joint closure after installation of the dowel. Dowels are typically placed on support chairs (nonmetallic or coated to prevent corrosion) for positioning. Figure 8.17 shows an end expansion cap and a nonmetallic support chair, as well as the inside of a cap revealing a plastic stop.

Excessive pressure should not be used to force the dowel past the stop as otherwise there will no room for expansion.

Resting on the support chairs, the dowel bar should be positioned in the slot so that it rests horizontally and parallel to the centerline of the pavement and at mid-depth of the slab. The proper alignment of the dowel bar is critical to its effectiveness, with the following placement tolerances commonly used (Darter 2017):

- +/- 1 in. of the middle of the concrete slab depth
- +/- 1 in. of being centered over the transverse joint
- +/- 0.5 in. from parallel to the centerline
- +/- 0.5 in. from parallel to the roadway surface

As previously described, a filler board or expanded polystyrene foam material must be placed at the mid-length of the dowel to allow for expansion and contraction, as well as to help form the continuation of the joint or crack within the dowel bar slot (ACPA 2006). Figure 8.18 shows a dowel bar being placed in a slot, and Figure 8.19 shows dowel bars that have been placed in slots.



WSDOT, used with permission

Figure 8.18. Placing a dowel bar assembly into a slot

WSDOT, used with permission

Figure 8.19. Dowel bars placed in slots

Mark B. Snyder, PERC, used with permission (left); WSDOT, used with permission (right)

Figure 8.17. End caps and chairs affixed on dowels (left) and close-up of end caps showing inside stop (right)

Step 5: Patching Material Placement

Once the dowel has been placed and the filler board material is in position, the patching material is then placed in the slot according to the manufacturer's recommendations. It is generally recommended that the patching material be placed in a manner that will not move or jar the dowel bar from its position in the slot (Pierce et al. 2003). That is, instead of dumping the patching material directly onto the dowel bars in the slots, it is recommended that the patching material be placed on the surface adjacent to each slot and then shoveled into the slot, as shown in Figure 8.20.

A small spud vibrator (i.e., <1.0 in.) should be used to consolidate the patching material (see Figure 8.21, left).

After the material is consolidated, it should be finished with a trowel such that it is flush with the existing pavement surface as shown in the top right photo of Figure 8.21. The surface may be left slightly high, however, if it is planned to grind the surface upon completion of the DBR.

During finishing, the patching material in the dowel bar slots should not be overworked, as this could cause migration of the fine material to the surface (Pierce et al. 2003). After consolidation and finishing, a curing compound should be placed on the patching material as shown in the bottom right photo of Figure 8.21.

After the patching material has gained sufficient strength, the transverse joint or crack should be reestablished using saws (as shown in Figure 8.22), usually within 24 hours after placement. The cut width is nominally between 0.19 to 0.31 in. wide and 1.5 in. deep (Darter 2017).

Reestablishing the transverse joint directly over the compressible insert will minimize the potential for spalling of the patching material and create a reservoir for the joint to be sealed.

Opening strength requirements vary by agency from about 1,600 to 3,000 lbf/in² (Darter 2017). This often translates into opening times of about 2 to 4 hours, but this will vary depending on the patching material. Agencies will specify the number of cylinders to be fabricated for measuring and monitoring strength gain.

It is not uncommon for some patching materials to exhibit minor shrinkage cracking after placement, but such shrinkage cracks characteristically remain tight and do not detract from the performance of DBR installations.

WSDOT, used with permission

Figure 8.20. Placing patching material into dowel bar slots

WSDOT, used with permission

Figure 8.21. Patching material placement: consolidation (left), finishing (top right), and curing compound application (bottom right)

WSDOT, used with permission

Figure 8.22. Sawcutting to reestablish the transverse joint through the patching material

Step 6: Diamond Grinding (Optional)

DBR installations may result in increased pavement surface roughness if not finished properly. This is typically due to differences in elevation between the finished dowel bar slots and the existing pavement or, in some cases, due to shrinkage or settlement of the patching material. Consequently, after the installation of retrofitted dowel bars, the entire pavement project is often diamond ground to restore a smooth riding surface. Chapter 9 provides detailed information on diamond grinding.

Step 7: Joint Sealing

After diamond grinding (or patching material placement if diamond grinding is not performed), the transverse joints should be prepared and sealed in accordance with agency policies. Chapter 10 provides detailed information on joint sealing.

6. Quality Assurance

As with any pavement project, the performance of DBR projects is greatly dependent on the quality of the materials and overall construction workmanship. Paying close attention to quality throughout the construction process is therefore key to reducing the likelihood of premature failures for DBR projects. To this end, this section summarizes some of the critical recommendations for successful DBR projects provided in the CP Tech Center's *Dowel-Bar Retrofit for Portland Cement Concrete Pavements* checklist (FHWA 2019b).

Preliminary Responsibilities

Agency and contractor personnel should collectively conduct a review of the project documentation, project scope, intended construction procedures, materials usage, and associated specifications. Such a collective review is intended to minimize any misunderstandings in the field between agency designers, construction personnel, and inspectors. Specific items for this review are summarized in the next sections.

Project Review

An updated review of the project's current condition is warranted to ensure that the project is still a viable candidate for DBR. Specifically, the following items should be verified as part of the project review process:

- Verify that the pavement conditions have not significantly changed since the project was designed.
- Verify that the pavement is structurally sound—a significant amount of slab cracking and/or corner breaks are indicators of structural deficiencies.
- Verify estimated quantities for the planned DBR project.

Document Review

The key project documents should be reviewed prior to the start of any construction activities. Some of the critical project documents include the following:

- Bid/project specifications and design
- Special provisions
- Traffic control plan
- Manufacturers' installation instructions for patching material(s)
- Manufacturers' MSDSs

Materials Checks

In preparation for the construction project, the following list summarizes many of the materials-related checklist items that should be verified:

- Verify that dowel slot patching material meets specification requirements.
- Verify that the dowel slot patching material is from an approved source or is listed on the agency's QPL, as required by the specification.
- Verify that the patching material components for the dowel slot cementing grout have been sampled, tested, and approved prior to installation, as required by contract documents.
- Verify that the additional or extender aggregates have been properly produced to an acceptable quality.
- If a mix design is required, ensure that it has been approved with all the materials/additives to be incorporated into the mix.
- Verify that all materials' packaging is not damaged (e.g., leaking, torn, or pierced).
- Verify that caulking filler meets specification requirements.
- Verify that dowels, dowel bar chairs, and end caps meet specification requirements.
- Verify that dowel bars are properly coated with epoxy (or other approved material) and are free of any minor surface damage, in accordance with contract documents.

- Verify that the curing compound meets specification requirements.
- Verify that the joint/crack reformer material (compressible insert) meets specification requirements (typically polystyrene foam board, 0.38 to 0.5 in. thick).
- Verify that the joint sealant material meets specification requirements.
- Verify that all required quantities of materials are on hand in sufficient quantities to complete the project.
- Ensure that all material certifications required by contract documents have been provided to the agency prior to construction.

Equipment Inspections

Prior to the start of construction, all construction equipment must be examined. The following are equipment-related items that should be checked:

- Verify that the slot-sawing machine is of sufficient weight and horsepower and is configured as required to cut the specified number of slots per wheel path to the depth shown on the plans.
- Verify that vacuum equipment used in conjunction with gang-sawing operations to remove slurry is functioning properly.
- Verify that jackhammers for removing concrete are limited to a maximum rated weight of 30 lb.
- Verify that tools such as bush hammers are available in case they are needed to produce a flat, level bottom.
- Verify that the abrasive cleaning unit is adjusted for the correct abrasive feed rate and has a properly functioning oil and moisture trap.
- Verify that the abrasive cleaning unit uses environmentally acceptable media.
- Ensure that abrasive cleaning operators use appropriate air purification systems as required and that OSHA requirements are met (including that only OSHA-approved equipment types are used and are functioning properly).
- Verify that air compressors have sufficient pressure and volume to adequately remove all dust and debris from slots to meet agency requirements.
- Verify that the airstream contains no water or oil prior to use by passing the stream over a board and examining for contaminants.
- For auger-type mixing equipment used to mix patching materials, ensure that auger flights or paddles are kept free of material buildup, which can cause inefficient mixing operations.
- Ensure that volumetric mixing equipment (such as a mobile mixer) is kept in good condition and calibrated on a regular basis to properly proportion mixes and record the w/cm ratio.
- Ensure that all material test equipment required by the specifications is available on site and is in proper working condition (e.g., slump cone, pressure-type air meter, cylinder molds and lids, rod, mallet, and ruler).
- Verify that vibrators are the size specified in the contract documents (typically 1 in. diameter or less) and are operating correctly.
- Verify that the concrete testing technician meets the requirements of the contract document for training/certification.
- Ensure that sufficient storage area specifically designated for the storage of concrete cylinders is available on the project site.

Weather Requirements

The weather conditions at the time of construction can have a large impact on DBR performance. Specifically, the following weather-related items should be checked immediately prior to construction and on a daily basis thereafter:

- Review manufacturer installation instructions for requirements specific to the patching material used.
- Confirm that the air and surface temperatures meet manufacturer and all agency requirements (typically 40°F and rising but no more than 90°F) for concrete patching material placement.
- Emphasize that neither dowel bar installation nor patching should proceed if rain is imminent.

Traffic Control

To manage the flow of traffic through the work zone, the following traffic-related items should be verified:

- Verify that the signs and devices conform to the traffic control plan stipulated in the contract documents.
- Verify that the traffic control setup complies with the Federal *Manual on Uniform Traffic Control Devices (MUTCD)* or local agency traffic control procedures.

- Verify that flaggers are trained/qualified according to the contract documents and agency requirements.
- Verify all workers are wearing the required PPE.
- Verify that unsafe conditions, if any, are reported to a supervisor.
- Ensure that, if slot sawing is allowed to proceed faster than concrete removal and dowel installation, traffic is not allowed to drive on the sawcuts for a period greater than that specified in the contract (typically five days).
- Verify that temporary dowel slot patching material (e.g., asphalt concrete) is available should the backfilling operation break down.
- Ensure that the repaired pavement is not opened to traffic until the patching material has attained the specified strength or curing time required by the contract documents.
- Verify that signs are removed or covered when they are no longer needed.

Project Inspection Responsibilities

During the construction process, careful project inspection by construction inspectors helps ensure well-performing DBR installations. Specifically, the following checklist items (organized by construction activity) summarize the recommended project inspection items.

Slot Cutting and Removal

During the slot cutting and removal operation, construction inspections should ensure the following:

- Verify that all slots are cut parallel to each other and to the centerline of the roadway within the maximum tolerance permitted by the contract documents, typically 0.25 in. per 12 in. of dowel bar length.
- For projects with skewed joints, ensure the dowels are installed parallel to the centerline and not perpendicular to the joint.
- Verify that the number of slots per wheel path (typically three or four) agrees with the contract documents.
- Verify that the cut slot length extends the proper distance on each side of the joint as required by the contract documents.
- Verify that the concrete between the sawcuts is removed using jackhammers not exceeding a maximum weight of 30 lb and/or with prying or breaking bars.
- Verify that the bottoms of slots are smoothed and leveled using a lightweight bush hammer.

Slot Cleaning and Preparation

The following should be closely inspected in relation to the cleaning of the slots and the adjacent area when preparing the slots prior to the placement of the dowels:

- Verify that after concrete removal, slots are prepared by media blasting, ensuring that all saw slurry is removed from the slot. (It is good practice to clean to a distance of 3 to 4 ft away from the perimeter of the slot.)
- Verify that air blasting is utilized to clean slots. A second air blasting may be required immediately before placement of the dowel slot patching material if slots are left open and become dirty.
- Verify that the existing joint/crack is sealed with approved caulking along the bottom and sides of each slot to prevent concrete patch material from entering the joint/crack. Ensure sealant does not extend more than ½ in. away from the joint.

Placement of Dowels

During the placement of the dowels into the cut slots, construction inspections should ensure the following:

- Verify that plastic end caps are placed on each end of the dowel bar to account for pavement expansion as required by the contract documents.
- Verify that dowels have been coated with an approved bond release compound to prevent the bonding of the dowel slot patch material to the dowels.
- Verify that proper clearance is maintained between the supported dowel bar and the sidewalls, ends, and bottom of the cut slot in accordance with the contract documents.
- Verify that chairs are used to align the dowel correctly in the slot, to support it, and to permit the dowel slot patching material to completely encapsulate the dowel bar (typically with a 0.5 in. clearance between the bottom of the dowel and the bottom of the slot).
- Verify that the joint re-former material (e.g., foam core insert) is placed at the midpoint of each bar and in line with the joint/crack to allow for expansion and to re-form the joint/crack.
- Verify that dowels are centered across the joint/crack such that at least 6 in. of the dowel extends on each side in accordance with the contract documents.

Mixing, Placing, Finishing, and Curing of Patching Material

To achieve a well-performing DBR project, it is imperative that good methods and procedures be used when mixing, placing, finishing, and curing the chosen patching material. Specifically, the following should be ensured during construction:

- Verify whether test strips are required in the contract and, if so, that they are conducted and accepted prior to full-scale production (typically, 20 joints/cracks are used for a test section length).
- Verify that dowel slot patching materials are mixed in accordance with manufacturer recommendations in small enough quantities to prevent premature set.
- Verify that concrete surfaces, including the bottoms of the slots, are dry before the placement of patching materials.
- Verify that material is consolidated using small, handheld vibrators that do not touch the dowel bar assembly during consolidation.
- Verify that the concrete patch material is finished flush with the surrounding concrete, using a finishing motion that proceeds from the center of the patch to the outside of the patch to prevent pulling material away from the patch boundaries; the surface of the concrete patch material should be finished slightly "humped" if diamond grinding will be done.
- Verify that all transverse joints are reestablished in the patching material within 24 hours of placement.
- Verify that adequate curing compound is applied immediately following finishing and texturing in accordance with the contract documents.
- Verify that the DBR operation is proceeding satisfactorily by retrieving cores, if required by the contract, to ensure proper dowel positioning and consolidation of the patching material (typically, 1 core is retrieved either every 600 bars or per day's production, whichever is less).

Cleanup Responsibilities

After the DBR construction procedures are complete, all remaining concrete pieces and loose debris on the pavement should be removed. Old concrete should be disposed of in accordance with agency specifications. Material mixing, placement, and finishing equipment should be properly cleaned in preparation for their next use.

Diamond Grinding

Diamond grinding, if required, should be conducted after completion of the repair work and prior to installation of joint sealant.

Resealing Joints/Cracks

Verify that joints/cracks are resealed (after all required diamond grinding) in accordance with the contract documents.

7. Troubleshooting

Some of the more common problems that a contractor or inspector may encounter in the field during construction as well as recommended solutions are summarized in Table 8.4.

Table 8.5 summarizes potential performance problems that may be observed shortly after the project is completed and opened to traffic along with their respective recommended solutions.

Table 8.4. DBR-related construction problems and associated solutions

Problem	Typical cause(s)	Typical solution(s) and explanation
Slots are not cut parallel to the roadway centerline	• Improper alignment of slot-cutting machine	Misaligned dowels can cause joint/crack lockup that will lead to slab cracking. Fill the original slots with concrete and recut at different locations. (Note: If the material between the sawcuts has not yet been removed, fill the sawcuts with an epoxy resin and recut at different locations.) Multiple-saw slot-cutting machines can ensure that slots are parallel to one another.
Lower slab deterioration is uncovered during slot cutting	• Subsurface deterioration, such as from materials-related distress	If lower slab deterioration is significant, an FDR will be required. Additional cores from other joints may be required to determine the extent of deterioration.
Dowel bar slots are too shallow	• Improper slot-cutting techniques	If a slot is too shallow, the dowel cannot be placed in its proper place in relation to the center of the slab. The solution is to saw the slots more deeply, remove the concrete to the proper depth, and complete as specified.
Dowel bar slots are too deep	• Improper slot-cutting techniques • Improper jackhammer weight • Improper jackhammering techniques	If dowels are placed in slots that are too deep, corner cracks may develop when traffic loads are applied. Follow these suggestions to minimize the probability of creating slots that are too deep: • Use a lightweight jackhammer (generally 30 lb maximum) • Do not lean on the jackhammer • Do not orient the jackhammer vertically; use no more than a 45° angle and push the tip of the hammer along the bottom of the slot • Stop chipping after a little more than mid-depth of the slab
Concrete fin is not easily removed	• Concrete containing mesh reinforcement	If mesh reinforcement is observed in the concrete, sever the steel at each end before attempting to remove the fin of concrete.
Jackhammer is punching through the bottom of the slot	• Improper jackhammering technique • Extremely deteriorated concrete	Make an FDR across the entire lane width at the joint/crack.
Factory-applied dowel coating is missing from one or more areas on the dowel	• Nonuniform application of the factory-applied dowel coating • Mishandling of dowels in the field	Areas of exposed steel can become concentrated points for corrosion that can eventually lead to the lockup of the dowel. If observed, recoat the dowel with a manufacturer-approved coating substance prior to the placing of the dowel in the slot. (Do not coat dowels in the slots because the sides and bottoms of the slots may become contaminated.)
Dowel cannot be centered over joint/crack because slot does not extend far enough	• Improper slot preparation	Chip out additional slot length with a jackhammer to facilitate proper placement of the dowel in accordance with the contract documents. Typically, at least a 6 in. segment of dowel is desired on each side of the joint/crack. Properly sized chairs will fit snugly into the slot.
Joint/crack caulking filler material does not extend all the way to the edge of the slot	• Improper caulking installation	Improperly placed caulking in a joint can allow incompressible patching material to enter the joint, thereby increasing the probability of a compression failure. Extend the caulking to the edge of the slot prior to the placement of patching material. If patching material does enter the joint adjacent to the slot, it must be removed using a technique agreed upon by the agency and the contractor.
Caulking material in a joint or crack extrudes onto a sidewall of the slot by more than 0.5 in.	• Improper caulking installation	Excessive caulking will not allow the patching material to bond to the sides of the slot. Therefore, remove excess caulking before placing patching material.
Dowels are misaligned after vibration	• Vibrator contacting the dowel assembly • Overvibration of the patching material • Improper width of the slots	Do not allow the vibrator to touch the dowel assembly. Check for overvibration; each slot should require only two to four short, vertical penetrations of a small-diameter spud vibrator. Ensure that the slots are sized the exact width of the plastic dowel bar chairs.

Sources: Adapted from ACPA 2006, FHWA 2019b

Table 8.5. Potential DBR-related performance problems and prevention techniques

Problem	Typical cause(s)	Typical solution(s) and explanation
In-place patching material cracking	• Joint is not well isolated • Dowels are not all properly aligned • Patching material is too strong • Patch was opened to traffic too soon • Material susceptible to excessive shrinkage	Confirm that proper construction practices are followed and patching material used is resistant to cracking. (Some tight shrinkage cracks can occur, but these typically will not detract from the performance of the repair material.)
In-place patching material popping out	• Slot was not properly cleaned or prepared • Repair material was not properly cured (causing unexpected material shrinkage during curing)	Verify that proper construction procedures are followed.
In-place patching material wearing off	• Nondurable repair material was used • Repair material was improperly mixed/handled • Patch is exposed to studded tires	Check material specifications, material preparation, and placement conditions to be sure that repair material is being handled properly.

8. Cross-Stitching

Introduction

Cross-stitching is a preservation method designed to strengthen nonworking longitudinal joints and cracks that are in relatively good condition (IGGA 2010). The construction process consists of inserting and bonding tie bars into holes drilled across the joint or crack at angles of 35 to 45 degrees relative to the pavement surface. This process is effective at preventing vertical and horizontal movement or widening of the crack or joint, thereby keeping the crack or joint tight, maintaining good load transfer, and slowing the rate of deterioration.

Cross-stitching was first used on a US highway by the Utah Department of Transportation (UDOT) in 1985 (ACPA 2001b). UDOT engineers used cross-stitching to strengthen uncontrolled cracks on a new 9 in. JPCP design on I-70 in central Utah. Considerable reflection cracking from the 4 in. lean concrete base occurred soon after construction. The cracks of major concern were the longitudinal cracks in or near the wheel paths of the driving lanes. After 15 years of service, a review of this cross-stitching project found the pavement to be in generally good condition, with some faulting across nondoweled transverse contraction joints (IGGA 2010).

Another early cross-stitching project was conducted in 2002 on the longitudinal joint of a portion of I-70 in Kansas in which the tie bars had been placed too deep in the 12 in. slab; after 15 years of service, this cross-stitching treatment was also found to be performing well with no spalling, cracking, or joint opening (Darter 2017). Overall, agency expectations for cross-stitching range from at least 10 to more than 20 years (Darter 2017).

Purpose and Application

Cross-stitching is applicable to several situations where strengthening joints or cracks is required, including the following (ACPA 2001b, Darter 2017):

- Strengthening longitudinal cracks in concrete pavements to prevent slab migration and to maintain aggregate interlock to help prevent further deterioration
- Mitigating the issue of tie bars not effectively holding longitudinal contraction joints together, either due to their excessively deep placement in the slab or, in some cases, to their omission altogether
- Tying roadway lanes or concrete shoulders that are separating and causing a maintenance problem
- Tying centerline longitudinal joints that are starting to fault

Longitudinal cracks that exist in the wheel path can be cross-stitched if they are relatively tight and not deteriorated; for example, MnDOT requires cracks to be less than 0.38 in. wide (Darter 2017). Faulted longitudinal joints or longitudinal cracks can also be cross-stitched if they are relatively tight.

Cross-stitching is not recommended for use on transverse cracks (especially those that are opening and closing in response to temperature changes) because cross-stitching does not allow movement. If used on working transverse cracks, a new crack may develop near the cross-stitched crack, or the concrete may spall over the reinforcing bars (ACPA 1995). Also, experience has demonstrated that cross-stitching is not a substitute for slab replacement if the degree of cracking is too severe, such as when slabs have multiple cracks or are shattered into more than four to five pieces (ACPA 2006).

In cases where drifted slabs are to be tied together, it is not necessary to attempt to move the drifted slabs together before cross-stitching. The primary concern in this case is preventing the backfill material (either epoxy or grout) from flowing into the space between the slabs; for these cases, a sand-cement grout is a suitable backfill for this purpose (ACPA 2006).

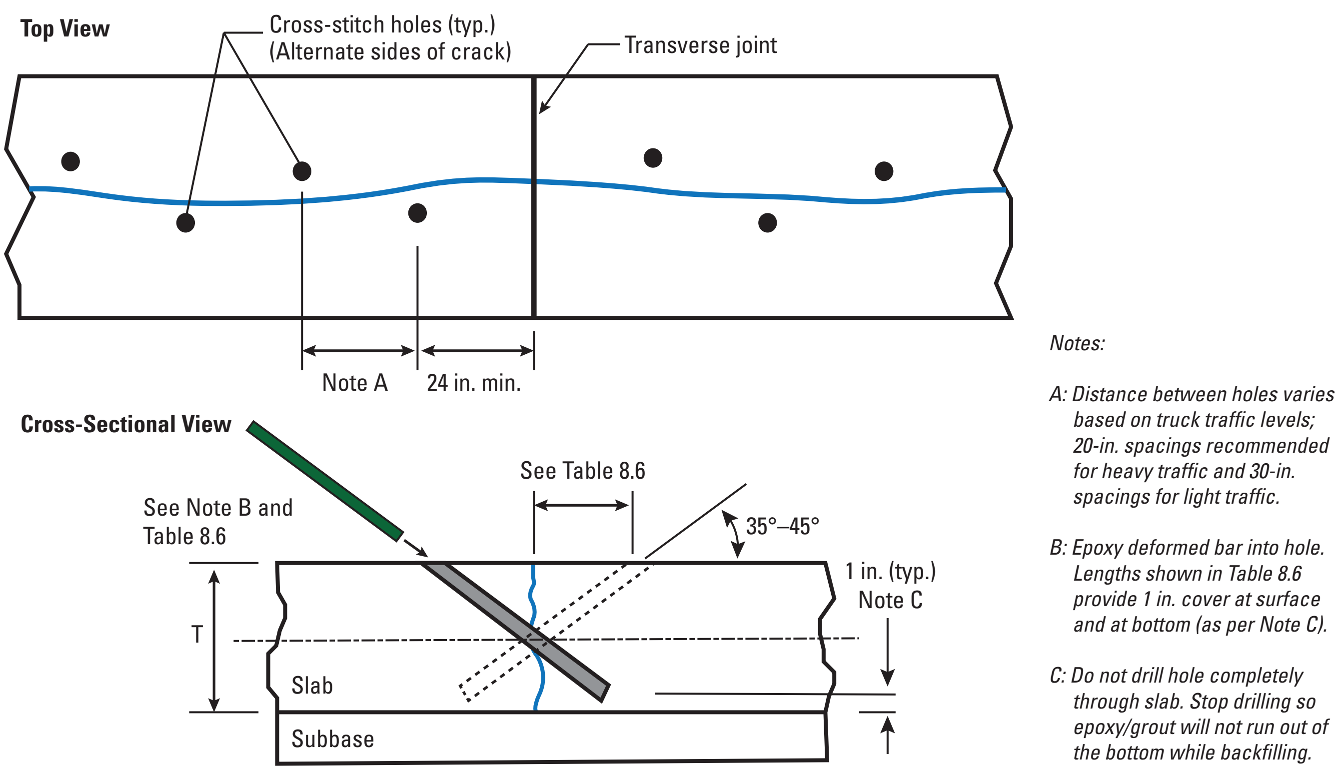
For CRCP, TxDOT recommends the only application for cross-stitching is when the lanes are separating at longitudinal construction joints (TxDOT 2021).

Construction Considerations

Cross-stitching generally uses a 0.75 in. diameter deformed tie bar to hold the joint or crack tightly together and enhance aggregate interlock (ACPA 2001b). The bars are typically spaced at intervals of 20 to 30 in. along the joint or crack and alternated on each side of the joint or crack (see Figure 8.23).

The recommended spacing of the tie bars used for cross-stitching varies by truck-traffic levels, with a 20 in. spacing recommended for heavy truck traffic and a 30 in. spacing recommended for light-traffic roadways and interior highway lanes (IGGA 2010). Another consideration is to provide greater levels of reinforcement to the more critical scenario, which is why KDOT specifies 30 in. spacings for longitudinal joints but 24 in. spacings for longitudinal cracking (Darter 2017).

An important element in the cross-stitching activity is to ensure that the hole intersects the joint or crack at mid-depth, as shown in the bottom cross-sectional view of Figure 8.23. This requires knowing the slab thickness and the required drilling angle. One set of overall recommendations on cross-stitching bar dimensions and angles/locations of the drilled holes is presented in Table 8.6.



Adapted from IGGA/ACPA 2001, 2010, used with permission

Figure 8.23. Top and cross-sectional views of cross-stitching

Table 8.6. KDOT cross-stitching bar dimensions and angles/locations of drilled holes

Slab thickness (in.)	Angle with pavement surface	Offset to drill hole (in.)	Rebar diameter (in.)	Depth of hole (in.)*
8.0	35°	5.75	0.75	12.50
8.5	35°	6.00	0.75	13.00
9.0	35°	6.50	0.75	14.00
9.5	35°	6.75	0.75	15.00
10.0	40°	6.00	0.75	14.00
10.5	40°	6.25	0.75	15.00
11.0	40°	6.50	0.75	15.50
11.5	40°	6.75	0.75	16.50
12.0	45°	6.00	0.75	15.75
12.5	45°	6.25	0.75	16.25
13.0	45°	6.50	1.00	17.00
13.5	45°	6.75	1.00	17.50
14.0	45°	7.00	1.00	18.50
14.5	45°	7.25	1.00	19.00
15.0	45°	7.50	1.00	20.00

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

Source: KDOT

The cross-stitching process requires the following steps and considerations (IGGA 2010, Darter 2017):

- Drill holes at an angle to the pavement so that they intersect the joint or crack at mid-depth (see Figures 8.23 and 8.24). Depending on the drill angle, a shallow pilot hole may be required at the prescribed location to help the drill get a "bite," as shown in Figure 8.25. It is important that the drill be equipped to lock in at the prescribed angle to ensure that the hole intersects the crack at about mid-depth of the slab. Both hydraulic and pneumatic drills have been used but it is important to minimize damage at the concrete surface and to select a drill diameter no more than 0.375 in. larger than the tie bar diameter. In addition, the drilling should not go through the bottom of the slab but instead should terminate about 1 in. from the bottom of the slab; this can be achieved by placing a cap on the drill.
- After drilling—and after verifying the air stream is oil-free and moisture-free—blow air into the drill holes to remove dust and debris.
- Inject epoxy into the hole, leaving some volume for the bar to occupy the hole (see Figure 8.26).
- Insert the tie bar in a rotating fashion to help distribute the epoxy around the bar and to remove air out of the hole (see Figure 8.27).
- Remove any excess epoxy from the top of the hole and finish the epoxy flush with the pavement surface, filling in any chipped areas. The pavement may be reopened to traffic as soon as the epoxy has fully set. A completed project is shown in Figure 8.28.

IGGA/ACPA, used with permission

Figure 8.24. Drilling holes for cross-stitching

John Donahue, MoDOT, used with permission

Figure 8.25. Pilot hole drilled in slab

John Donahue, MoDOT, used with permission

Figure 8.26. Injecting epoxy into a drilled hole used for cross-stitching

John Donahue, MoDOT, used with permission

Figure 8.27. Inserting a tie bar into a drilled hole

IGGA/ACPA, used with permission

Figure 8.28. Completed cross-stitching

Quality Assurance

Critical recommendations for successful cross-stitching projects are presented in the following sections, based largely on the *Cross-Stitching for Portland Cement Concrete Pavements* checklist that the CP Tech Center developed for the FHWA (FHWA 2019c).

Preliminary Responsibilities

Agency and contractor personnel should collectively conduct a review of the project documentation, project scope, intended construction procedures, materials usage, and associated specifications. Such a collective review is intended to minimize any misunderstandings in the field between agency designers, construction personnel, and inspectors. Specific items for this review are summarized in the next sections.

Project Review

An updated review of the current project's condition is warranted to ensure that the project is still a viable candidate for cross-stitching. Specifically, the following items should be verified as part of the project review process:

- Verify that pavement conditions have not significantly changed since the project was designed.
- Check estimated quantities for cross-stitching materials.
- Verify actual pavement thickness and compare plan's layout against industry recommendations (e.g., ACPA 2001b, IGGA 2010).

Document Review

The key project documents should be reviewed prior to the start of any construction activities. Some of the critical project documents include the following:

- Bid/project specifications and design
- Special provisions
- Traffic control plan
- Manufacturer's installation instructions for epoxy material
- Manufacturer's MSDSs

Materials Checks

A number of materials-related checks are recommended prior to the start of a cross-stitching project. Specifically, agency and contractor personnel should collectively verify the following:

- Verify that repair materials meet specification requirements.
- Verify repair materials are being obtained from an approved source or are listed on the agency's QPL, if required by the specification.
- Verify that repair material components have been sampled, tested, and approved prior to installation as required by the contract documents.
- Verify that material packaging is not damaged (e.g., packages leaking, torn, or pierced), preventing proper use.
- Verify that tie bars meet specification requirements: size, strength, and coating.
- Verify that tie bars are properly coated with epoxy (or other approved material) and are free of any minor surface damage in accordance with contract documents.
- Verify that all required materials are on hand in sufficient quantities to complete the project.
- Ensure that all material certifications required by contract documents have been provided to the agency prior to construction.
- Ensure the epoxy materials for tie bar insertion have not exceeded their shelf life.

Equipment Inspections

Prior to the start of construction, all construction equipment must be examined. The following are equipment-related items that should be checked:

- Verify that vacuum equipment, if used in conjunction with drilling operations to remove dust, is functioning properly.
- Ensure OSHA requirements are being met for worker safety during drilling operations.
- Verify that an appropriate fixture or system is in place to correctly align the drill holes at the designated angle, size, and depth.
- Ensure the proper drill bit size is used to allow tie bar and epoxy placement (typically 0.25 to 0.38 in. larger than the tie bar diameter).
- Ensure hydraulic drills are correctly set up to minimize damage to the pavement.
- Verify that air compressors have sufficient pressure and volume to adequately remove all dust and debris from drill holes.
- Verify the airstream contains no water or oil prior to use by passing the stream over a board and examining for contaminants.
- Ensure epoxy injection equipment is in proper working order and has sufficient capacity to provide the required volume of material.

Weather Requirements

Immediately prior to the start of the construction project and on a daily basis thereafter, the following weather-related concerns should be checked:

- Review manufacturer's installation instructions for temperature requirements specific to the epoxy injection material.
- Ensure air and surface temperatures meet agency requirements (typically 40°F and rising but no more than 90°F).

Traffic Control

To manage the flow of traffic through the work zone, the following traffic-related items should be verified:

- Verify that signs and devices conform to the traffic control plan stipulated in the contract documents.
- Verify that the traffic control setup complies with the Federal *Manual on Uniform Traffic Control Devices (MUTCD)* or local agency traffic control procedures.
- Verify that flaggers are trained/qualified according to contract documents and agency requirements.
- Verify that all workers are wearing the required PPE.
- Verify that unsafe conditions, if any, are reported to a supervisor.
- Ensure that the repaired pavement is not opened to traffic until the epoxy injection material has attained the specified strength or curing time as required by the contract documents.
- Verify that signs are removed or covered when they are no longer needed.

Project Inspection Responsibilities

During the construction process, careful project inspection by construction inspectors helps ensure well-performing cross-stitching installations. Specifically, the following checklist items (organized by construction activity) summarize the recommended project inspection items.

Drilling Holes for Tie Bars

- Verify that the plan's layout is appropriate for the thickness of the existing pavement and traffic conditions.
- Verify that drill holes are marked out according to project plans, ensuring correct spacing and offset to crack or joint. (Offsets should be at right angles to joint or crack and are typically spaced 20 to 30 in. apart.)
- Ensure the drill fixture or system maintains the correct drill depth and angle and does not punch through the bottom of the pavement. (Drilling for cross-stitching is typically accomplished at a 35- to 45-degree angle.)
- Verify that OSHA air quality requirements are maintained during drilling operations.
- Verify that drill holes are the correct size for tie bar and epoxy placement. (Note: Design charts typically allow for the tie bar to be located 1 in. from the top and bottom of the slab.)
- Verify that no transverse joints or cracks are being cross-stitched.

Placement of Tie Bars and Epoxy

- Verify that air blasting is utilized to clean drill holes. A second air blasting may be required immediately before placement of the epoxy if the holes are left open and become dirty or wet.
- Verify the correct tie bar lengths and diameter are used.
- Verify tie bars have appropriate epoxy coating thickness that is undamaged or satisfactorily repaired.
- Ensure the correct amount of epoxy is placed in the drill holes to fill the cavities once the tie bars are inserted.
- Ensure installed tie bar remains below the pavement surface and epoxy is flush with surface.

Optional Crack Sealing

- Use a crack chasing saw to create a reservoir for sealant if required by contract documents.
- Perform abrasive blasting on the crack.
- Seal crack with required sealant.

Cleanup Responsibilities

- Verify that all loose debris is removed from the pavement surface and disposed of in accordance with contract documents.
- Verify that epoxy placement equipment is properly cleaned for the next use.

Troubleshooting

A few troubleshooting items that may occasionally be encountered on a cross-stitching project are listed in Table 8.7 along with possible solutions.

Table 8.7. Cross-stitching-related construction problems and associated solutions

Problem	Typical solution(s)
Drill wanders or dances when initiating the drilling operation and may cause shallow spalling	Drill a shallow pilot hole at the prescribed location to establish a "bite."
Drill breaks through the bottom of the slab	Verify the actual pavement thickness and check the angle and depth requirements; adjust as necessary for future holes.
Epoxy coating on tie bars is damaged	Recoat the tie bar with a manufacturer-approved coating.
Tie bar extends above the pavement surface	Remove the tie bar before the epoxy sets and redrill the hole; if the tie bar is still too long, cut the tie bar to accommodate the hole depth.
Epoxy not setting properly	Consult the manufacturer's instructions and verify the shelf life of the epoxy.

9. Slot-Stitching

Introduction

Slot-stitching is a preservation treatment/repair technique for longitudinal cracks and joints that grew out of the DBR technique. The process and technique for slot-stitching is like DBR except slot-stitching uses deformed tie bars and it is applied at longitudinal joints and cracks. Generally, wider cracks can be addressed with slot-stitching as compared to cross-stitching.

Purpose and Application

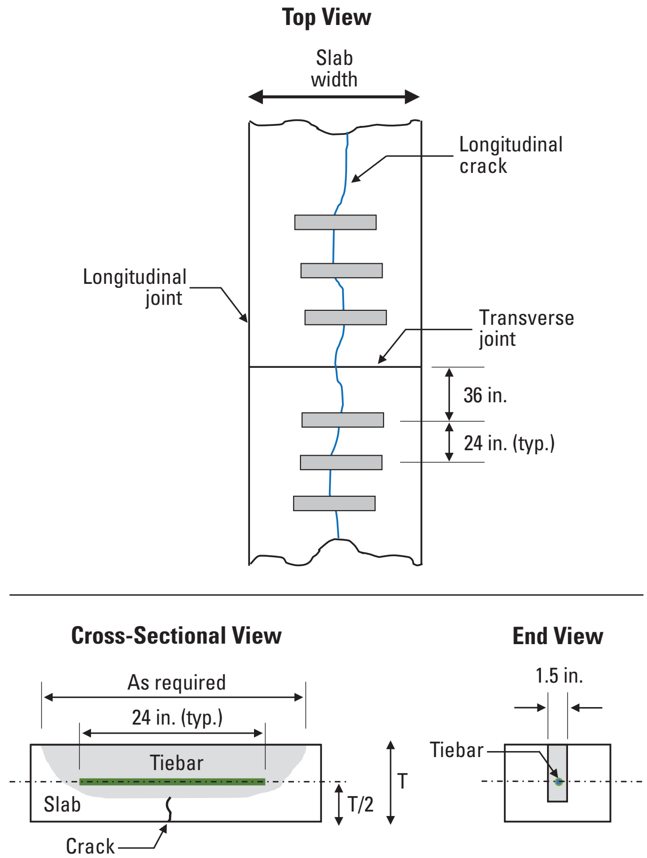
The purpose of slot-stitching is to hold together adjoining concrete slabs or segments through the use of deformed tie bars (typically 1 in. in diameter or larger) placed in slots cut into existing concrete pavement (IGGA 2010). The goal is to maintain some degree of aggregate interlock and to prevent cracks from further opening and deteriorating. Slot-stitching should be used with caution on CRCP as the depth of its slots is necessarily shallower, which makes the repair material more susceptible to failure if there are significant vertical movements.

Construction Considerations

The slot-stitching process is like the DBR process, consisting of the following steps (IGGA 2010):

- Cut slots approximately perpendicular to the longitudinal joint or crack using a slot-cutting machine or walk-behind saw. Unlike in the case of DBR, alignment for slot-stitching is not critical since the deformed bars will hold the joint tightly together, preventing the slabs from separating.
- Prepare the slots by removing the concrete and cleaning the slot. If the slabs have separated, consider using a joint reformer and caulking the joint or crack to prevent the precision backfill materials from flowing into the area between the slabs.
- Place the deformed bars into the slot.
- Place the backfill material into the slot and vibrate it so it thoroughly encases the bar. (Select a backfill material that has very low shrinkage characteristics.)
- Finish flush with the surface and cure.

Figure 8.29 illustrates the slot-stitching preservation treatment.



Adapted from IGGA/ACPA 2001, 2010, used with permission

Figure 8.29. Top, cross-sectional, and end views of slot-stitching

John Donahue, MoDOT, used with permission

Figure 8.30. Completed slot-stitching of a longitudinal crack

Note that the slots in slot-stitching can be cut merely approximately perpendicular to the longitudinal joint or crack, and if the joint or crack width is particularly wide, consideration should be given to reforming and caulking prior to the placement of the patching material. Figure 8.30 shows slot-stitching that has been performed on a longitudinal crack.

Slot-Stitching versus Cross-Stitching

To date, there have been no studies that have documented the comparative benefits and costs between cross-stitching and slot-stitching. In general, however, cross-stitching can be constructed more quickly and less expensively than slot-stitching, and the resulting repair is less aesthetically offensive. Moreover, cross-stitching is more suitable for use on CRCP. Slot-stitching, on the other hand, can generally be applied to more severe cracks and cracks that are wider but potentially could lead to an increase in roughness. In any case, as described previously, the cause of any longitudinal cracking should be carefully evaluated in order for it to be fixed correctly; working cracks that are stitched by either cross-stitching or slot-stitching could lead to the development of additional cracking in other locations.

10. Summary

This chapter provided guidance for properly designing and installing retrofitted dowel bars in concrete pavements. DBR is intended to restore load transfer across joints or cracks that exhibit poor load transfer from one side of the joint or crack to the other. Dowel bar retrofit provides a number of benefits, including a reduction in faulting rates, improvements in pavement performance, and extensions to pavement life. Pavements most suited for DBR are those that are in relatively good condition (i.e., display little or no distress) but are exhibiting poor joint load transfer. The optimum time for the application of this strategy is when the pavement is just beginning to exhibit signs of distress, such as pumping or the onset of faulting. Although this chapter primarily focused on the details of the DBR technique, it also contained a brief discussion on the pavement cross-stitching and slot-stitching techniques, which are used primarily to strengthen nonworking longitudinal cracks and longitudinal joints that are in relatively good condition (ACPA 2001b).

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Chapter 9

Diamond Grinding and Grooving

1. Introduction

Conventional diamond grinding (CDG) and diamond grooving are two different surface restoration procedures that are used to correct concrete pavement surface distresses or deficiencies. Each technique addresses a specific pavement shortcoming and is often used in conjunction with other pavement preservation techniques (e.g., DBR, PDRs, and FDRs) as part of a comprehensive pavement preservation program. In some situations, it may be justified to use diamond grinding or grooving as the sole preservation technique, although this will depend on the conditions and characteristics of the specific project.

This chapter describes the use of both diamond grinding and diamond grooving and discusses important design considerations and construction procedures for successful projects using each treatment. Although these two treatments are the focus of this chapter, two other surface-texturing processes are also briefly described:

- Next Generation Concrete Surface (NGCS), a manufactured, low-noise texture developed for both new and existing concrete pavements
- Cold milling, which has some application for concrete pavement removal (such as for PDRs, as described in Chapter 5) and in preparation of an existing bituminous pavement for a concrete overlay but is not recommended as a final riding surface

2. Diamond Grinding

Purpose

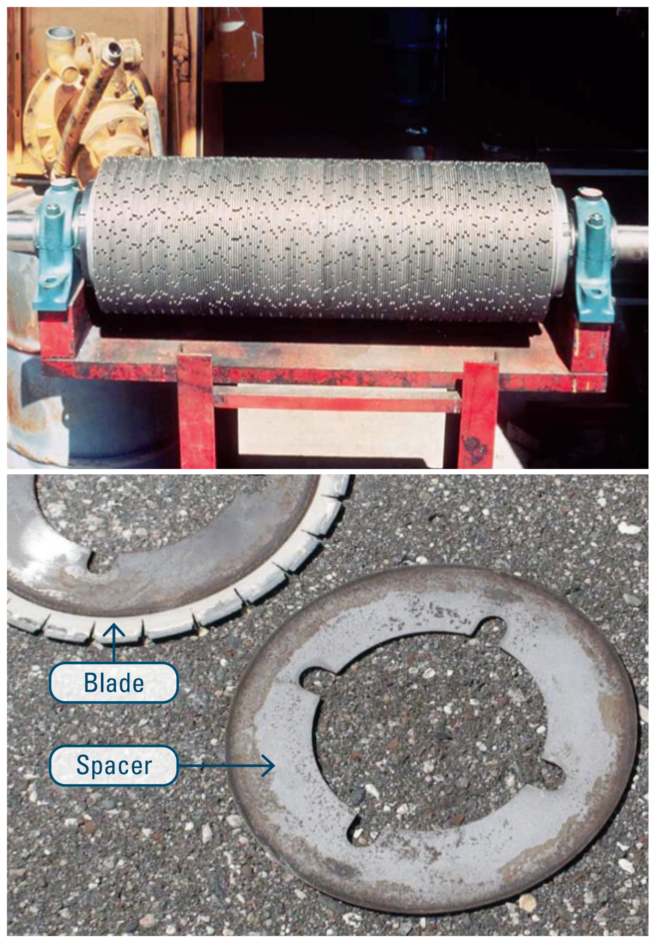
Diamond grinding is the removal of a thin layer of hardened concrete pavement surface (typically about 0.25 in.) using a self-propelled machine outfitted with a series of closely spaced diamond saw blades mounted on a rotating shaft. Diamond grinding is primarily conducted to restore or improve ride quality, but it also provides improvements in surface texture (and therefore safety) as well as reductions in noise levels. The focus herein is on the use of diamond grinding for preservation, but it is also noted that diamond grinding can be used as the final surface texturing for new concrete pavement construction or may be used intermittently on new paving projects to help meet smoothness specifications.

One of the earliest highway projects to use diamond grinding was in 1965 on a then-19-year-old section of Interstate 10 in California to eliminate significant faulting (Neal and Woodstrom 1976). Since that first diamond grinding operation, this section of pavement has been periodically reground at least an additional three times and continues to demonstrate outstanding performance.

Figure 9.1 shows images of a diamond grinding head along with the individual diamond blades and spacers that comprise the grinding head.

The blades and spacers are placed in alternating fashion on the shaft to produce the desired corduroy-type surface texture shown in Figure 9.2.

The surface texture produced by the diamond grinding operation is shown in Figure 9.3.

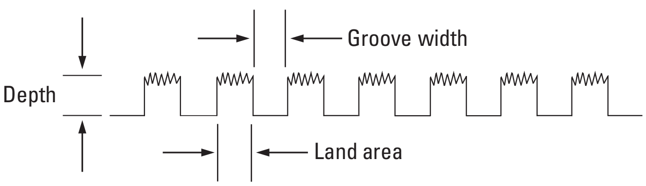


IGGA (top) and ACPA/IGGA (bottom), used with permission

Figure 9.1. Grinding head (top) and saw blades and spacers (bottom)

Matt Ross, CTS Cement, used with permission

Figure 9.2. Surface texture provided by diamond grinding



Adapted from IGGA, used with permission

Figure 9.3. Surface texture produced by diamond grinding

Since its first use on a highway project in 1965, diamond grinding has grown to become a major element of concrete pavement preservation projects. Diamond grinding has been employed on concrete pavement surfaces to address a number of different distresses and conditions such as the following:

- Removal of transverse joint and crack faulting
- Removal of wheel path "rutting" caused by studded tire wear
- Removal of "built-in" slab curling and/or warping
- Texturing of a polished pavement surface to improve surface friction
- Improvement of transverse slope to improve surface drainage
- Reduction in tire-pavement noise levels

By improving the overall smoothness of the pavement and by increasing surface texture, diamond grinding also makes a positive contribution to the overall sustainability of the pavement structure. Smoother pavements reduce fuel consumption and, by extension, reduce greenhouse gas emissions, while the provision of adequate surface texture promotes safety and reduces wet-weather crashes (Van Dam et al. 2015, Van Dam 2021). These benefits are even more pronounced on high-traffic roadways. An additional sustainability advantage of diamond grinding is that it achieves these smoothness and safety benefits without the preparation, transport, and placement of new paving materials.

One final sustainability advantage of diamond grinding is its potential for increasing the sequestration of carbon dioxide through carbonation (Van Dam 2021). Carbonation is the process by which a concrete surface absorbs carbon dioxide from the atmosphere over its service life (PCA 2021), but the amount of absorption diminishes with time. Periodic diamond grinding creates a fresh surface that increases the rate of carbonation and the subsequent amount of carbon dioxide that can be sequestered (Van Dam 2021).

Project Selection

Existing Concrete Pavements

Although most state and local highway agencies have their own criteria for determining when to use diamond grinding, some general considerations for its use on a specific concrete pavement project include the following:

- Average transverse joint faulting in excess of 0.10 in.
- IRI values in excess of 80 to 170 in./mi, but this will depend on traffic levels and posted speeds, as well other factors such as social (e.g., user) and environmental (e.g., fuel consumption and emissions) considerations
- Wheel path wear from studded tires greater than 0.25 to 0.50 in.
- Surface friction values below agency standards for the roadway facility and location
- Tire-pavement noise issues in noise-sensitive areas

The above information notwithstanding, it is important to recognize that diamond grinding is not appropriate for all pavement conditions. When selecting candidate projects for diamond grinding, many pavement-related characteristics (such as structural condition, pavement materials, traffic level, past rehabilitation history, underlying joint conditions, and current distress types, severities, and extents) must be considered (Darter 2017). Some of these additional considerations are described below to help agencies determine the feasibility of diamond grinding for a given project:

- Pavements with high levels of roughness may be beyond the window of opportunity for cost-effective diamond grinding, particularly if the pavement is exhibiting structural deterioration.
- Faulting of transverse joints suggests load transfer and slab support issues, so agencies should consider the installation of retrofitted dowel bars and slab stabilization (and possibly retrofitted edgedrains) prior to the diamond grinding operation to address the root cause of the faulting. (If the underlying cause of the faulting is not addressed, then it can quickly redevelop.)
- Structural distresses such as corner breaks, working transverse cracks, and shattered slabs will require repair before grinding. The presence of significant slab cracking in a project (often taken as more than 10% of the slabs exhibiting cracks) suggests a structural problem that diamond grinding cannot address. Similarly, the presence of significant slab replacement and repair may be indicative of continuing progressive structural deterioration that grinding would not remedy.
- The hardness of the aggregate affects the cost of the diamond grinding operation. Grinding a pavement with extremely hard aggregate (such as chert, traprock, or quartzite) takes more time and effort than grinding a pavement with a softer aggregate (such as limestone). These hard aggregates, however, hold the diamond-ground texture longer and can therefore provide an extended service life.
- Diamond grinding is generally not suitable as a long-term preservation treatment for concrete pavements suffering from durability problems (such as D-cracking or ASR) but may be used for such pavements as a stopgap solution when only a few years of additional service are desired.
- Jointed reinforced concrete pavements may have wire mesh located near the surface of the pavement, which could create localized spalling of the concrete surface if it is diamond ground.

If a pavement project contains few structural or materials-related problems, the decision to diamond grind a pavement often comes down to an assessment of its overall roughness and faulting levels, along with the overall economics of the operation (which will depend on the type of aggregate, the depth of removal, size of the project, availability of contractors, and so on). Furthermore, agencies should be aware that there are a number of factors that contribute to roughness besides faulting (such as settlements, heaves, and joint deterioration), and these contributions to roughness may not be fully addressed through diamond grinding. Each agency is encouraged to develop guidelines to determine the appropriateness of performing diamond grinding based on their local experience and on the specific project conditions.

Concrete Pavements with Asphalt Overlays

Many agencies employ asphalt overlays as a patterned response to the loss of serviceability in concrete pavements, regardless of the types and extent of deterioration or the structural condition of the pavement. In many cases, these overlaid pavements may have been good candidates for the application of concrete pavement preservation treatments (including diamond grinding) to restore overall smoothness.

Even if a concrete pavement has an asphalt overlay, there may still be opportunities to implement a concrete pavement preservation strategy, provided that the overlay was placed for functional reasons and that the underlying concrete pavement is structurally sound and in relatively good condition (Frentress 2009, IGGA 2020a). To help determine if a particular concrete pavement is a good candidate for preservation, a review of performance records and bid documents is useful to help determine its pre-overlay pavement conditions; however, an examination of the existing pavement will still be needed to confirm the viability of the proposed concrete pavement preservation approach. Overall, the process should include the following activities:

- Conduct an engineering analysis of the existing pavement in accordance with the procedures described in Chapter 3 (records review, nondestructive testing using an FWD or GPR device, coring and subsurface boring, etc.) to confirm the structural integrity of the underlying concrete pavement.
- If determined suitable, remove the asphalt overlay through milling. It is critical that the milling head not be allowed to mill into the concrete surface as this will result in damage to the transverse and longitudinal joints.

- Implement restorative treatments to return the pavement to a more robust condition; treatments may include the following:
  - Slab stabilization (to restore slab support)
  - Partial-depth repairs
  - Full-depth repairs and/or slab replacement
  - Dowel bar retrofit
  - Cross-stitching
- Diamond grind the surface to remove faulting and other surface irregularities as well as to restore surface friction.
- Seal the joints (as specified) and open the project to traffic.

This "buried treasure" strategy offers a number of benefits, including reduced cost, improved performance and longevity, and reduced environmental impacts (as no new materials are being introduced). The strategy has been used successfully by a number of state transportation departments, including Arizona, Iowa, Minnesota, New Jersey, and Washington. A 2009 New Jersey project on a section of NJ 21 near Newark was one of the first documented uses of this approach (Frentress 2009). The Iowa Department of Transportation (Iowa DOT) also implemented this concept in the eastbound direction of US 20 in 2014, removing the asphalt overlay and performing patching, DBR, and diamond grinding. Figure 9.4 shows the current US 20 eastbound pavement after 4 years of service.

More recently, the Arizona Department of Transportation (ADOT) has performed over 1 million yd² of diamond grinding on previously asphalt-overlaid concrete freeway pavements in the Phoenix area (Everett 2020).

Limitations and Effectiveness

A number of studies on the effectiveness of diamond grinding have indicated excellent long-term performance when diamond grinding is conducted in conjunction with other required CPR activities (Rao et al. 1999a, Stubstad et al. 2005, Chen and Hong 2015, Darter 2017). One possible explanation for this positive impact on pavement service life is the long-standing theory that eliminating faulting and restoring smoothness reduces the dynamic effects of traffic loadings on the performance of the pavement.

Immediate Effectiveness

The immediate effect of diamond grinding is a noticeable reduction in the pavement roughness, the magnitude of which depends on the pre-grinding roughness of the pavement and the form of the roughness, among other things. An evaluation of 11 projects in Texas, for example, showed an average reduction of 60 in./mi immediately after grinding (Chen and Hong 2015). It is common to express this reduction in terms of a percentage from the pre-grinding roughness using the following equation:

$$\text{\% reduction in roughness} = [(R_b - R_a)/R_b] \times 100 \qquad \textbf{(9.1)}$$

Where:

R_b = Smoothness before grinding (typically expressed in terms of IRI but could be any smoothness statistic)

R_a = Smoothness after grinding

Figure 9.4. Previously overlaid concrete pavement on US 20 in Iowa restored via patching, DBR, and diamond grinding

For example, an original pavement with an IRI of 150 in./mi that was ground to an IRI of 60 in./mi would show a 60% reduction in roughness (i.e., 150 minus 60 divided by 150). For the 11 projects in Texas, an average reduction in roughness of about 40% was achieved (Chen and Hong 2015). Many agencies offer incentives that are tied to the amount of the reduction in roughness or to the final smoothness that is achieved by the contractor.

Performance

Field studies of diamond-ground pavement have indicated that diamond grinding can be an effective, long-term treatment. For example, a 1999 study of 76 projects in 9 states showed that the average longevity of diamond-ground projects (i.e., the time until a second grinding or rehabilitation was needed) was 14 years, whereas the expected longevity at an 80% reliability level was 11 years (i.e., 80% of the sections lasted at least 11 years) (Rao et al. 1999a, Rao et al. 1999b). A 2005 study of diamond-ground projects in California revealed that, on average, diamond-ground pavements have an expected longevity of nearly 17 years (at a 50% level of reliability) and a longevity of 14.5 years at an 80% level of reliability (Stubstad et al. 2005). More recently, case study investigations of diamond grinding effectiveness in several states have revealed the following (Darter 2017):

- Utah has found that for its nondoweled JPCP designs diamond grinding projects exhibit about 15 years of performance. On projects where DBR was performed, the faulting has remained low and even longer service lives are expected.
- Georgia has observed service lives of at least 20 years for its diamond grinding projects, depending on the pavement design and materials durability.
- In Minnesota, the service life of diamond-ground pavement has been found to be long but also to depend on the adequacy of other repairs. If DBR is not performed, Minnesota has determined that pavements will last about 10 years before faulting reoccurs, whereas with DBR, pavement performance can be 20 years or more.
- Missouri has seen diamond grinding extend pavement service life by 7 to 9 years.
- Washington State sees variable performance among its diamond grinding projects, depending on their exposure to studded tires. In the eastern half of the state, diamond grinding texture life is on the order of 15 years whereas on the western side of the state diamond grinding lasts 25 to 30 years.

Diamond grinding can also be repeated several times during the life of the pavement to help maintain the functional qualities of the pavement. As previously mentioned, the early diamond grinding project on I-10 in California has been ground multiple times and still exhibits good performance, and an adjacent section on the same freeway has been ground five times and continues to carry traffic (Ram et al. 2020).

Friction and Safety

In addition to addressing pavement roughness, diamond grinding also produces a pavement surface with ample macrotexture, which contributes to surface friction. An Arizona study showed that the increase in friction values associated with different grinding configurations ranged between 15% and 41%, with an overall average improvement of 27% (Scofield 2003). In Wisconsin, Drakopoulos et al. (1998) found that the overall crash rate for diamond-ground surfaces was 60% of the crash rate for the unground surfaces. A Texas study of 11 diamond grinding projects showed a 30% increase in skid number and reductions in crash occurrences by approximately 62% and 46% for fatalities and incapacitating injuries, respectively (Chen and Hong 2015).

Tire-Pavement Noise

Another documented benefit of diamond grinding is its ability to reduce tire-pavement noise. An unwanted characteristic of pavements with faulted transverse joints or cracks is the thumping or slapping created by tires as they pass over the joints or cracks. Because diamond grinding removes faulting, the result is not only a smoother pavement, but a quieter one as well. Some state transportation departments are also allowing contractors to use diamond grinding as the final surface texture, since diamond grinding can produce a more consistent, smoother, and quieter surface than many conventional textures (Rasmussen et al. 2012). In addition, diamond grinding has been shown to reduce exterior noise levels by 2 to 6 dBA by eliminating the "whine" commonly associated with transverse tining (Snyder 2006).

Clearances and Curb Lines

Diamond grinding also offers the distinct advantages of not affecting overhead clearances at bridges, curb lines, or the hydraulic capacities of gutters in urban areas.

Limitations

Although diamond grinding is highly effective in removing faulting and restoring smoothness, the underlying mechanism of the faulting distress must be treated in order to prevent its redevelopment (ACPA 2000). One study indicated that following diamond grinding, if nothing is done to control load transfer, faulting redevelops at a fast rate initially but then stabilizes to a faulting rate comparable to that just prior to the diamond grinding (Rao et al. 1999a). This was observed in an evaluation of the performance of diamond grinding in Utah, in which the pavements that had undergone DBR were found to provide longer performance than those that had not (Darter 2017). Therefore, to stop faulting from rapidly returning in nondoweled JPCP sections after grinding, other CPR work (such as DBR and perhaps slab stabilization) must be conducted in conjunction with the grinding operation.

Also, as previously mentioned, diamond grinding does nothing to add to the structure of a pavement, so alternative structural solutions must be sought if significant structural issues or deficiencies exist. In addition, the hardness of the aggregate in the concrete will affect the longevity of the pavement's surface texture in that softer aggregates will tend to polish, losing their surface texture more quickly.

Because diamond grinding is removing a portion of the slab thickness, a common concern is whether that reduces the load-carrying capacity of the pavement, which could potentially result in increased cracking. Studies have indicated, however, that the slight reduction in slab thickness resulting from diamond grinding in fact does not significantly compromise the fatigue life of the slab, largely because the continued long-term strength gain of the concrete offsets any slight reduction in slab thickness (Rao et al. 1999a).

Design Considerations

When considering a diamond grinding operation, information on the degree of joint faulting at transverse joints (and cracks, if applicable) is needed. Concurrent restoration techniques, such as DBR, slab stabilization, and retrofitted edgedrains, should be considered to help minimize the recurrence of joint faulting after grinding. Plans and specifications should clearly define the areas for diamond grinding and which concurrent restoration activities are required.

A schematic of the surface texture produced by the diamond grinding operation was illustrated previously in Figure 9.3; the major components include the groove width, the land area, and the groove depth. These dimensions will vary depending on the blade spacing that is selected for a particular project, which is a function of the aggregate hardness. Pavements with harder aggregates (such as granite) require closer blade spacing to cut through the harder rock and to ensure that the fins break off under traffic, whereas pavements with softer aggregates (such as limestone or dolomite) can accommodate slightly wider blade spacings. Table 9.1 summarizes the ranges of typical dimensions for diamond grinding operations, while Figure 9.5 illustrates the importance of selecting the appropriate blade spacing for the final ground surface. In Figure 9.5, the harder aggregate in the pavement required closer blade spacings (i.e., more blades per foot of width) to produce the desired result.

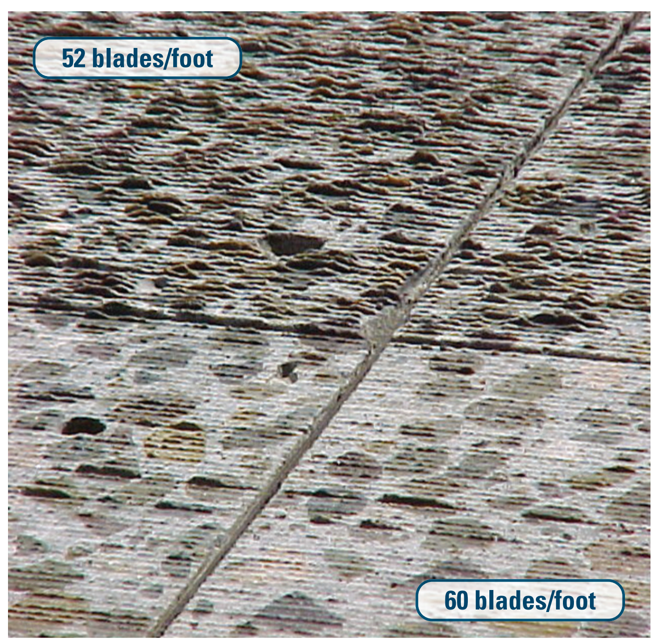
Table 9.1. Range of typical dimensions for diamond grinding operations

Characteristic	Range	Hard aggregate[a]	Soft aggregate[b]
Groove width	0.090–0.150 in.	0.090–0.150 in.	0.090–0.150 in.
Land area	0.070–0.130 in.	0.070–0.110 in.	0.090–0.130 in.
Depth	0.040–0.12 in.	0.040–0.12 in.	0.040–0.12in.
No. of blades	50–60/ft	53–60/ft	50–54/ft

[a] *Such as granite, quartz, or some river gravels*

[b] *Such as limestone or dolomite*

Source: ACPA 2006



Photograph provided by IGGA, used with permission

Figure 9.5. Varying effects of blade spacing on finished diamond-ground surface texture

Even though the land area is conceptually easy to visualize, its use in specifications is problematic because its dimensions depend on many other factors, not all of which are defined in specifications. For example, the width of the saw blade core, the width of the diamond saw blade segments affixed on the periphery of the blade, and the width of the spacers between the blades all affect the land area width. Moreover, blade irregularities and wear, variability in machine setup and operator control, as well as difficulties in obtaining in-field measurements further complicate the use of land area as a specification item.

Figure 9.6 presents a schematic close-up of the saw blade core and spacer arrangement and shows how the saw blade segments extend out beyond the width of the saw blade core. In this example, the 0.105 in. saw blade core and the 0.110 in. spacer produce a land area of 0.090 in.

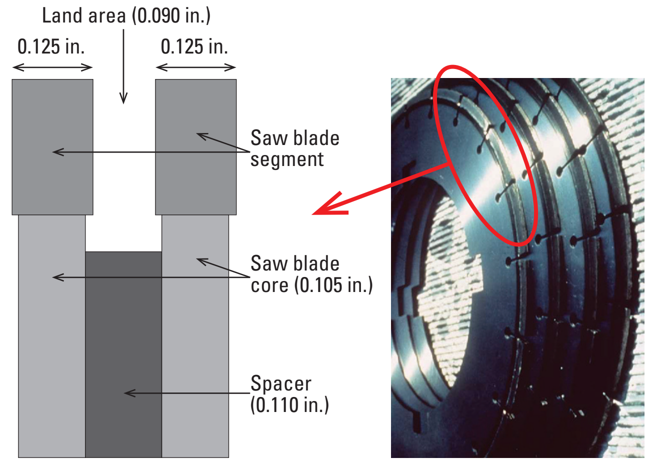


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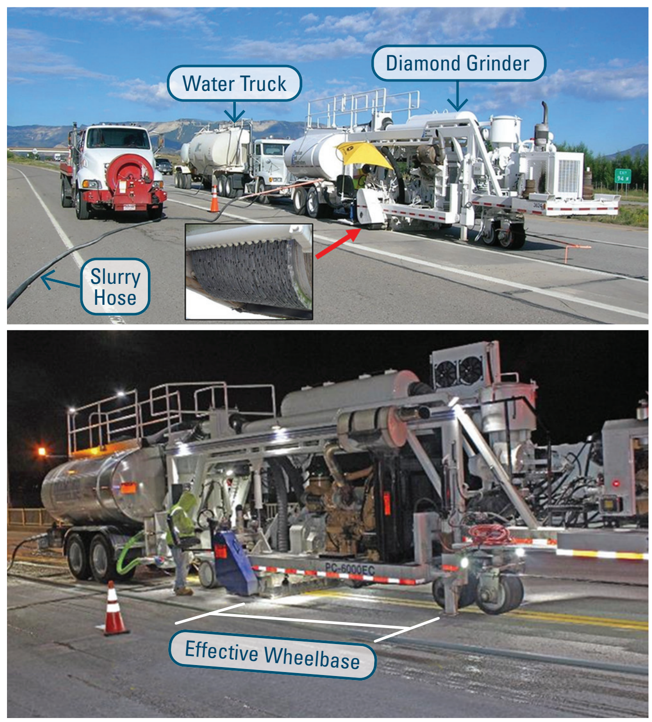
Figure 9.6. Typical configuration of saw blade and spacer pairings

Because of the issues associated with specifying land area dimensions, the number of blades per unit width is recommended for use in grinding specifications (see bottom row in Table 9.1). In general, more blades will be used per unit width for projects with harder aggregates (in order to produce a thinner land area), and fewer blades will be used per unit width for projects with softer aggregates (in order to produce a wider land area). The contractor then works with the blade manufacturer to select the appropriate number of blades needed for a given project.

Construction Considerations

Equipment

Diamond grinding operations use self-propelled machines equipped with diamond blades and spacers mounted on a spindle to provide the desired pattern. Various sizes are available, but typical diamond grinding equipment for production grinding operations have a 4 ft wide grinding head, an effective wheelbase of between 10 and 14 ft (as measured from the leading bogie wheels to the depth-control wheels), and a weight of between 53,000 and 62,000 lb, which includes the grinding head (Scofield 2020). The grinding machines are also equipped with a vacuuming system for removing grinding residue from the pavement surface. The production rates vary considerably from about 2 to 40 ft per minute. Figure 9.7 shows characteristics of typical diamond grinding equipment.



Recreated from IGGA, used with permission

Figure 9.7. Diamond grinding equipment (top) and effective wheelbase (bottom)

The cutting head affixed on the equipment typically is 4 ft wide. As previously shown in Figure 9.1, the diamond blades are typically spaced in the range of 50 to 60 blades per ft, depending on the hardness of the aggregate. Figure 9.8 shows the stacking of the spacers and blades to assemble the cutting head.

IGGA, used with permission

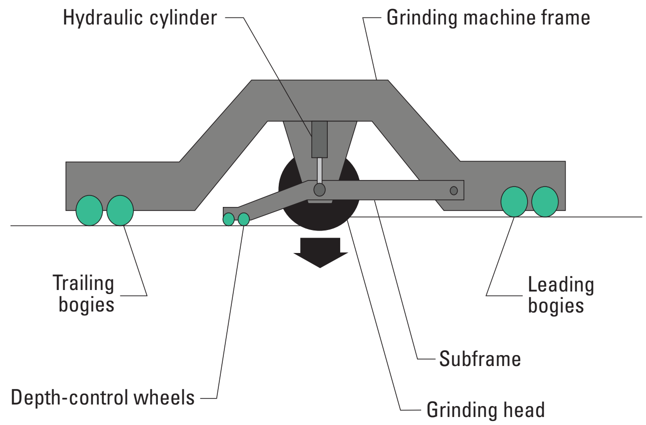
Figure 9.8. Stacking spacers and blades on a diamond grinding machine's cutting head

Procedures

Figure 9.9 shows a simplified conceptual illustration of a diamond grinding machine.

The length of the equipment serves as a reference plane, and the grinding head located in the central part of the diamond grinding machine removes the high spots in the pavement. By blending the highs and lows, excellent riding quality can be obtained with a minimum depth of removal. Low spots will likely be encountered, and specifications should recognize this. Generally, it is required that a minimum of 95% of the area within any 3 by 100 ft test area be textured by the grinding operation. However, this requirement should be considered in light of the existing pavement conditions, as on older or badly faulted pavements it can be difficult to achieve 95% coverage at a removal depth of 0.25 in. Isolated low spots of less than 2 ft^2 should not require texturing if lowering the cutting head would be required (ACPA 2000).

Grinding should be performed continuously along a traffic lane for best results. Grinding should always be started and ended from lines that run perpendicular to the pavement centerline and should also be consistently maintained parallel to the centerline. Grinding can be conducted on multilane facilities using a mobile single



Recreated from ACPA 2000, used with permission

Figure 9.9. Diamond grinding machine

lane closure, allowing traffic to be carried on any adjacent lanes. The traffic control plan must comply with the Federal or local agency traffic management procedures. Multiple grinding machines working together can be used to help expedite the grinding operation.

Several passes of the diamond grinding equipment will be needed to grind an entire traffic lane. It is recommended that the amount of overlap between adjacent passes of the diamond grinding equipment be maintained to no more than 1 to 2 in. Occasionally an adjacent pass may deviate slightly and leave a short unground area called a dog tail, but this generally is not an issue unless it is large. Vertical deviations between adjacent passes should be less than 0.12 in.

Prior to performing any grinding work, obtaining a profile of the existing surface as the control profile is recommended. Profile measurements may be obtained by the agency or by the contractor using either lightweight or high-speed profiler equipment. Because of inaccuracies with the use of single-point (i.e., spot) lasers on a textured surface, the profiler should be equipped with a wide-footprint (i.e., line) laser (see discussion in Chapter 3).

The control profile can be used to identify the target value for the required project smoothness. Upon completion of the diamond grinding process, the profile should be rerun and evaluated to determine whether or not the diamond-ground surface now meets the smoothness requirements.

It is important to note that diamond grinding is most effective at removing short-wavelength roughness, such as that caused by faulted joints. Roughness caused by long wavelengths can be more difficult to remove but can still be accomplished.

"Holidays" refer to unground areas of the pavement that remain after the grinding operation. While it is intended for the entire surface to be textured during a diamond grinding operation, most specifications provide for a small amount of holidays within a project (for example, 95% diamond-ground texture coverage for any 3 by 100 ft test area is commonly specified). Figure 9.10 shows a typical holiday on a diamond-ground concrete pavement.

Most production grinding equipment can grind within about 10 to 24 in. of a lateral obstruction, depending on the machine type. Several options are available to enable grinding closer to any obstructions:

- Use a nonconforming grinder attachment, typically allowing grinding within about 9.5 in. of the obstruction.
- Use specialty grinding equipment, which is typically capable of grinding within about 4 in. of the obstruction.
- Use smaller, walk-behind grinding equipment.

Additional feathering passes with the diamond grinding machine are often needed to assimilate the surface elevation of the ground pavement with the surface elevation of any adjacent shoulders, through lanes, or entrance/exit ramps that are not ground. This is to ensure a uniform cross slope across the pavement, to prevent the ponding of water, and to eliminate abrupt vertical deviations between the two adjacent surfaces. Similarly, a feathering pass will also be needed when grinding adjacent to a curb and gutter to maintain a uniform cross slope, but this will often require the use of a smaller grinding machine. Figure 9.11 shows a gutter apron before and after a feathering pass.

As described in Chapter 5, the presence of certain flexible polymer-based repair materials on the pavement may create some issues for the diamond grinding operation. Some of the key considerations for the diamond grinding of a concrete pavement with large areas repaired with flexible polymer-based repair materials include the following (Ram et al. 2019):

- Limit the loading and time of grinding operations as much as possible, as heavy downward forces can cause the diamond-head blades to sink too deeply into the repair material, gumming up the blades and potentially injecting the repair material into the vacuum pumps.
- Avoid grinding operations at higher ambient temperatures (say, above 90°F) and work to keep the grinding head as cool as possible.

IGGA, used with permission

Figure 9.10. Holiday on a diamond-ground surface

Dan Frentress, IGGA, used with permission

Figure 9.11. Gutter apron before (top) and after (bottom) a feathering pass

- Restrict grinding to only small polymer-based repairs, as larger areas may be more apt to gum up the diamond-bladed grinding heads.
- Alternatively, either
  - add coarse aggregates into the polymer repair material to reduce the potential for these issues or
  - install the flexible repair materials after the grinding operation (if they are not already in place).

Quality Assurance

This section summarizes the recommended quality control activities for diamond grinding as presented in the CP Tech Center's *Diamond Grinding of Portland Cement Concrete Pavements* checklist (FHWA 2019a).

Preliminary Responsibilities

Agency and contractor personnel should collectively conduct a review of the project documentation, project scope, intended construction procedures, materials usage, and associated specifications. Such a collective review is intended to minimize any misunderstandings in the field between agency designers, construction personnel, and inspectors. Specific items for this review are summarized below.

Project Review

An updated review of the pavement condition is warranted to ensure that the project bid quantities are sufficient and that the project is still a viable candidate for diamond grinding. The following items should be evaluated as part of the review process:

- Verify that the pavement conditions have not significantly changed since the project was designed.
- Ensure broken or rocking slabs are repaired/replaced prior to diamond grinding.
- Verify that other pavement repairs are conducted prior to diamond grinding, except for joint sealing and PDRs using elastomer-based concrete. (If elastomeric material is to be used for PDRs, it may be necessary to install the repairs after diamond grinding.)

Document Review

The key project documents should be reviewed prior to the start of any construction activities. Some of the critical project documents include the following:

- Bid/project specifications and design
- Special provisions
- Traffic control plan
- Slurry disposal requirements

Equipment Inspections

Prior to the start of construction, all construction equipment must be examined. The following are items that should be checked on the diamond grinding machine:

- Verify that the diamond grinding machine meets the requirements of the contract documents for weight, horsepower, blade configuration, and effective wheelbase.
- Verify that the equipment has an effective means of vacuuming the grinding residue from the pavement, leaving the surface in a clean, near-dry condition.
- Verify that the blade spacing on the diamond grinding cutting head meets the requirements of the contract documents and can produce the desired corduroy texture.

The following are items that should be checked on the inertial profiling equipment:

- Verify that the profile-measuring equipment is acceptable for measuring diamond-ground textures (i.e., uses a wide-footprint/line laser versus single-point/spot laser).
- Verify who will be conducting profile measurements and when they will be conducted.
- Verify that the inertial profiling unit has been calibrated in accordance with its manufacturer's recommendations and contract documents; calibration should be conducted on a similarly diamond-ground texture.
- Verify that the operator meets the requirements of the contract documents for training/certification.

Weather Requirements

The following weather-related items should be checked immediately prior to construction and daily throughout the diamond grinding process:

- Air and/or surface temperature should meet minimum agency requirements (typically 35°F and rising) for diamond grinding operations in accordance with the contract documents.
- Diamond grinding should not proceed if icy weather conditions are imminent.

Traffic Control

To manage the flow of traffic through the work zone, the following traffic-related items should be verified:

- Verify that the signs and devices conform to the traffic control plan stipulated in the contract documents.
- Verify that the traffic control setup complies with the Federal *Manual on Uniform Traffic Control Devices (MUTCD)* or local agency traffic control procedures.
- Verify that all construction personnel are wearing the required PPE.
- Verify that the repaired pavement is not opened to traffic until all equipment and personnel have been removed from the work zone.
- Verify that signs are removed or covered when they are no longer needed.
- Verify that any unsafe conditions are reported to a (contractor or agency) supervisor.

Project Inspection Responsibilities

During the construction process, careful project inspection by construction inspectors helps ensure well-performing diamond grinding projects. Specifically, the following checklist items (organized by construction activity) summarize the recommended project inspection items:

- Verify that the diamond grinding operation conforms to project requirements.
- Ensure that diamond grinding proceeds in a direction maintained consistently parallel with the pavement centerline, beginning and ending at lines perpendicular to the pavement centerline.
- Verify that diamond grinding results in a corduroy texture extending across the full lane width and that this texture is in accordance with the contract requirements.
- Verify that the grinding equipment does not cause raveling, aggregate fractures, or disturbance to the joints.
- Verify that the construction operation proceeds in a manner that produces a neat, uniform finished surface.
- Verify that the shoulder, auxiliary, or ramp lane grinding transitions from the edge of the mainline pavement as required to provide drainage, leaving no more than a 0.19 in. ridge and an acceptable riding surface.
- Verify that each application of the diamond-ground texture overlaps the previous application by no more than the amount designated in the contract documents.
- Verify that each application of the diamond-ground texture does not vary from the depth of the previous application by more than the amount permitted in the contract documents, typically 0.12 in.
- Verify that the finished cross slope mirrors the pre-grind cross slope and has no depressions or misalignment in slope greater than 0.25 in. per 12 ft when measured with a 12 ft straightedge placed perpendicularly to the centerline.
- Verify that lateral drainage is achieved by maintaining a constant cross slope between grinding passes in each lane.
- Verify that the project's concrete slurry is adequately vacuumed from the pavement surface and is not allowed to flow into adjacent traffic lanes.
- Verify that the grinding residue disposal operation is in accordance with agency specifications and local environmental regulations.

3. Diamond Grooving

Purpose and Project Selection

Diamond grooving is a process in which parallel grooves are cut into the pavement surface using diamond saw blades with a typical center-to-center blade spacing of 0.75 in. The principal objective of diamond grooving is to provide escape channels for surface water, thereby reducing the incidence of hydroplaning, which can otherwise contribute to wet-weather crashes. Diamond grooving should only be applied to pavements that are structurally and functionally adequate. Figure 9.12 shows the recommended groove dimensions, whereas Figure 9.13 shows a longitudinally grooved surface.

Grooving on concrete pavements has been performed since the 1950s to reduce the potential for wet-weather crashes, and it may be performed either transversely or longitudinally. The main advantage of transverse grooving is that it provides a shorter path for the drainage of water from the pavement and helps reduce hydroplaning (IGGA 2020b); in addition, it produces a surface that provides considerable braking traction.

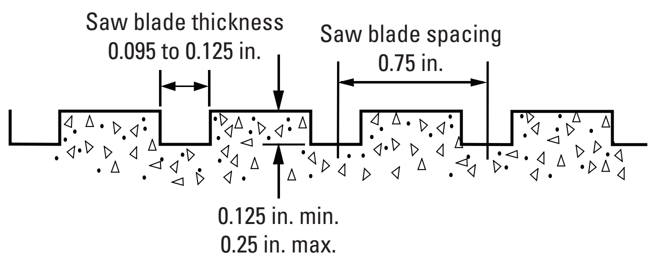


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Figure 9.12. Diamond-grooved concrete pavement showing typical grooving dimensions

IGGA, used with permission

Figure 9.13. Longitudinally diamond-grooved surface

Although common on bridge decks, transverse grooving is not often used on roadway pavements due in part to difficulties encountered during construction in maintaining traffic on the adjacent lane and in part to the higher noise levels that transverse grooving can generate.

Longitudinal grooving is more commonly used on roadways. Longitudinal grooving is often done in localized areas where wet-weather crashes have been a problem, such as curves, exit ramps, bridges, and intersection approaches. As with transverse grooving, longitudinal grooving has been shown to be effective in reducing hydroplaning on pavements while offering the additional benefit of producing a tracking effect for vehicles, particularly on horizontal curves (IGGA 2020b).

Limitations and Effectiveness

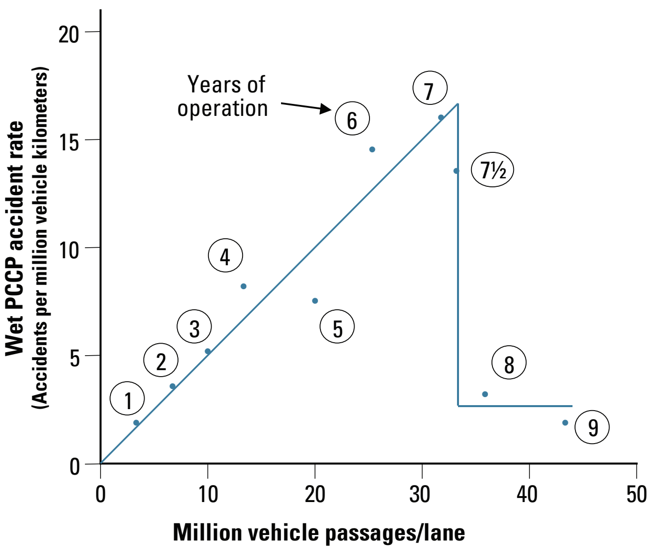
As previously described, diamond grooving increases the macrotexture of the pavement and provides channels for water to escape, thereby decreasing the potential for hydroplaning. This was profoundly demonstrated in an early diamond grooving project in California; Figure 9.14 shows how the number of wet-weather crashes increased with time until 7.5 years after construction, at which point diamond grooving was performed and the number of crashes was reduced by nearly 80% (Ames 1981).

Across multiple projects, average reductions in wet-weather accident rates after grooving are about 70% (IGGA 2020c).

One interesting behavioral attribute of diamond grooving is its strong effect in reducing wet-weather accidents while often producing little or no change in the surface friction of the pavement as measured when using a ribbed tire (Scofield 2016). To investigate this phenomenon, a study was performed using the California CT-342 test device, which has the ability to measure friction at any angle on a pavement surface, as shown in Figure 9.15.

The study's friction results measured at each angle were recorded and expressed as a ratio of the friction measured in the longitudinal direction; thus, ratios greater than 1 indicate that the friction at the angle of measurement is greater than that of the standard longitudinal direction and ratios less than 1 indicate that the friction at the angle of measurement is less than that of the standard longitudinal direction (Scofield 2016).

The overall results of the study are presented in Figure 9.16 and indicate that the friction ratio for grooved surfaces increases as the measured angle increases.



Adapted from Ames 1981

Figure 9.14. Wet-weather crashes on California concrete pavement for years 1 through 7 before longitudinal grooving and years 7.5 through 9 after longitudinal grooving

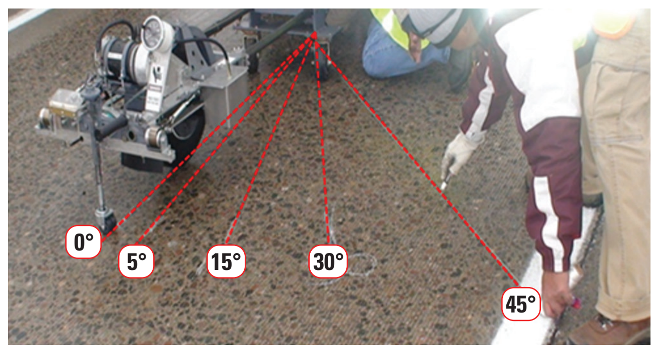


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Figure 9.15. Friction measurements via the California CT-342 device at various angles relative to the centerline of the pavement

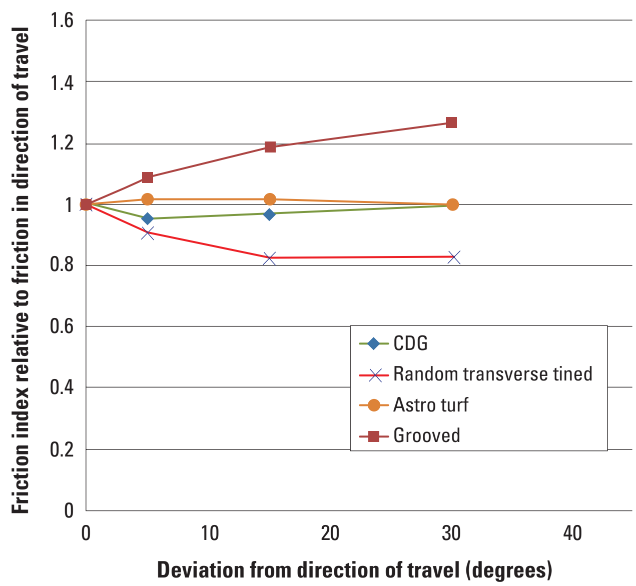

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Figure 9.16. Ratio of friction measured at varying angles relative to the standard longitudinal friction measurement for various concrete pavement surface textures

This may suggest that, as drivers begin to lose control of their vehicles, the friction that the tires encounter as they deviate from a true longitudinal direction increases, which aids in the ability to recover control (Scofield 2016). One caveat on this study is that the CT-342 test procedures require testing when the ambient temperature is above 40°F and, during this study's testing, the temperatures fell below that threshold. However, earlier simulation modeling had shown similar results and suggested that it was such skewed frictional effects that were helping keep skidding vehicles within the roadway and reducing wet pavement accidents (Ong and Fwa 2007).

Historically, a stated disadvantage of longitudinal grooving has been the perception by motorcyclists and drivers of small vehicles that longitudinal grooving impairs their ability to control their vehicles. This subject was studied at length by the California Division of Highways in the 1960s and 1970s (Zube et al. 1968, Sherman et al. 1969, Karr 1972). Although some small lateral movement was still noted by these vehicles on longitudinally grooved pavements, the agency did adopt a 0.095 in. groove width and a groove spacing of 0.75 in. to help minimize these effects.

In 2007, several of California's original longitudinally grooved pavements were reevaluated to determine their noise emission characteristics. While the study confirmed the effectiveness of longitudinal grooving in providing lateral stability and improved friction, it was determined that longitudinal grooving was not an effective treatment for noise mitigation (ACPA 2007).

Design Considerations

Grooving operations are intended to reduce hydroplaning and accompanying wet-weather crashes. Information regarding an area with a high number of wet-weather crashes, as well as surface friction data for the relevant pavement section, should be reviewed prior to considering grooving operations. The typical dimensions for grooving operations were presented previously in Figure 9.12 and are summarized in Table 9.2.

Table 9.2. Typical dimensions for diamond grooving operations

Characteristic	Typical dimension
Groove spacing	0.75 in.
Groove width (blade width)	0.095 to 0.125 in.
Groove depth	0.125 to 0.25 in.

Areas to be grooved should be clearly indicated on project plans. Typically, the length of an entire project is not grooved, but the operation is focused on localized areas where wet-weather crashes have been an issue (e.g., curves, ramps, and intersections).

The entire lane width should be grooved, but allowance should be made for small areas that remain ungrooved because of pavement surface irregularities. If the existing pavement profile exhibits excessive roughness, it may be necessary to employ diamond grinding prior to the grooving operation to improve smoothness. Otherwise, pavements exhibiting dips and bumps may make it extremely difficult to maintain a consistent groove depth and stay within specification.

Construction Considerations

Equipment

The equipment used to groove pavements is similar to that used for diamond grinding but is specifically designed for the grooving activity. Like diamond grinding, the cutting head for diamond grooving uses a series of blades that are stacked on a drum mounted on a self-propelled machine. However, because fewer diamond blades are required on the cutting head, the cutting head can be wider than that used in diamond grinding; cutting head widths up to 6 ft are commonly used. Figure 9.17 shows a diamond grooving cutting head.

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Figure 9.17. Diamond grooving cutting head

Procedures

Construction procedures for diamond grooving typically follow those described previously for diamond grinding. Most commonly, grooving is performed longitudinally along a pavement, in which case it should begin and end from lines that run perpendicular to the pavement centerline and be maintained consistently parallel to the centerline. If multiple passes are required, the additional passes should be performed to maintain the same groove spacing across the adjacent passes. Grooves are typically not cut closer than about 3 in. to a parallel longitudinal joint.

Typically, only localized areas (such as curves or intersection approaches) are grooved, instead of the entire project length. Wet-weather crash data can be examined to help identify those specific areas where grooving may be required.

The traffic control plan must comply with Federal or local agency traffic control standards to ensure the safety of the construction personnel and traveling public.

Quality Assurance

This section summarizes the recommended quality control activities for diamond grooving as presented in the CP Tech Center's *Longitudinal Diamond Grooving of Portland Cement Concrete Pavements* checklist (FHWA 2019b).

Preliminary Responsibilities

Agency and contractor personnel should collectively conduct a review of the project documentation, project scope, intended construction procedures, materials usage, and associated specifications. Such a collective review is intended to minimize any misunderstandings in the field between agency designers, construction personnel, and inspectors. Specific items for this review are summarized below.

Project Review

An updated review of the pavement condition is warranted to ensure that the project bid quantities are sufficient and that the project is still a viable candidate for diamond grooving. The following items should be evaluated as part of the review process:

- Verify that pavement conditions have not significantly changed since project design.
- Ensure that broken or rocking slabs are repaired/replaced prior to diamond grooving.
- Verify that other pavement repairs except joint sealing are conducted prior to diamond grooving.
- Verify that construction phasing/staging allows for grooving at all required locations.

Document Review

The key project documents should be reviewed prior to the start of any construction activities. Some of the critical project documents include the following:

- Bid/project specifications and design
- Special provisions
- Staging requirements
- Traffic control plan
- Slurry disposal requirements

Equipment Inspections

Prior to the start of construction, all construction equipment must be examined. The following are equipment-related items that should be checked:

- Verify that diamond grooving equipment meets contract requirements and uses multiple diamond blades mounted on a self-propelled machine designed for diamond grooving concrete pavement and bridge decks.
- Verify that grooving equipment has a depth-control device enabling the adjustment of the cutting head height to maintain the specified groove depth.
- Verify that grooving equipment has the capability to maintain alignment with the center of the roadway.
- Verify that grooving equipment can install grooves at the dimensions and spacing designated in the plans.
- Verify that grooving equipment has an effective means of vacuuming the grinding residue from the pavement surface, leaving the surface in a clean, near-dry condition.
- Verify that blade spacing on the diamond grooving head meets contract requirements and can produce the desired groove width, spacing, and depth.

Weather Requirements

The following weather-related items should be checked immediately prior to construction and on a daily basis thereafter:

- Air and/or surface temperature should meet minimum agency requirements (typically 35°F and rising) for diamond grooving operations.
- Diamond grooving should not proceed if icy weather conditions are imminent.

Traffic Control

To manage the flow of traffic through the work zone, the following traffic-related items should be verified:

- Verify that signs and devices match the traffic control plan stipulated in the contract documents.
- Verify that the traffic control setup complies with the Federal *Manual on Uniform Traffic Control Devices (MUTCD)* or local agency traffic control procedures.
- Verify that all construction personnel are wearing the required PPE.
- Verify that the grooved pavement is not opened to traffic until all equipment and personnel have been removed from the work zone and the pavement is clean and safe for traffic.
- Verify that signs are removed or covered when they are no longer needed.
- Verify that any unsafe conditions are reported to a (contractor or agency) supervisor.

Project Inspection Responsibilities

During the construction process, careful project inspection by construction inspectors helps ensure well-performing diamond grooving projects. Specifically, the following checklist items (organized by construction activity) summarize the recommended project inspection items:

- Ensure that diamond grooving proceeds in a direction parallel to the pavement centerline, beginning and ending at lines perpendicular to the pavement centerline.
- Verify that the grooving equipment does not cause raveling and produces neat vertical sawcut groove faces.
- Verify that the construction operation proceeds in a manner that produces a neat, uniformly grooved surface across the full roadway width. (Note: Grooving often terminates at the shoulder stripe.)
- Grooves should not be allowed to overlap an existing longitudinal joint. (Note: Grooves are typically not cut closer than 3 in. and no more than 6 in. away from longitudinal joints.)
- Verify that each application of diamond grooving does not overlap the previous application and maintains the specified groove spacing, typically 0.75 in.
- Verify that proper groove width is obtained (typically 0.095 to 0.125 in.) and is uniform throughout the project.
- Verify that groove depth conforms to project specifications (typically 0.25 in.) and is uniform throughout the project.
- Verify that concrete grooving slurry is adequately vacuumed from the pavement surface and is not allowed to flow into adjacent traffic lanes.
- Verify that grooving residue is not discharged into a waterway, onto a roadway slope within 100 ft of any natural stream or lake, or within 3 ft of a water-filled ditch. Concrete grooving slurry must be collected and discharged at the disposal area designated in the contract documents.

4. Next Generation Concrete Surface

Purpose and Project Selection

With the emergence of tire-pavement noise emissions as an issue in many parts of the country, the concrete industry launched a research program to evaluate the tire-pavement interaction phenomenon and the effects of diamond grinding on noise levels (Scofield 2016). Research determined that the noise levels for diamond-ground textures were more a function of the fin (land) profile than of blade or spacer widths (Dare et al. 2009). From that research, NGCS was developed as a manufactured surface that controls the resulting fin profile and minimizes positive (upward) texture (Scofield 2010). A comparison of the surface textures produced by CDG and NGCS is provided in Figure 9.18.

It is important to note that while the flush grind process does create an extremely smooth riding surface, this process was not intended to reduce roughness by any significant degree. Consequently, on new pavements, the required level of smoothness should be achieved prior to constructing NGCS, whereas on existing pavements any significant roughness should first be removed through conventional diamond grinding before performing the NGCS operation.

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Figure 9.18. Comparison of surface textures produced by CDG (top) and NGCS (bottom)

NGCS uses conventional diamond grinding equipment and blades, but in such a way that no positive texture is produced (Scofield 2016). It consists of a two-pass operation in which an initial flush grinding is first performed and then followed by a separate grooving operation. NGCS can be used on both new pavement construction and in the rehabilitation of existing concrete pavements (Scofield 2016). So far, NGCS projects have been constructed in 15 states and by three international transportation authorities (IGGA 2019).

Limitations and Effectiveness

NGCS has been determined to be the quietest nonporous concrete texture developed to date (Scofield 2016). Typical noise levels (as measured using AASHTO TP 76, Standard Method of Test for Measurement of Tire/Pavement Noise Using the On-Board Sound Intensity [OBSI] Method) at the time of construction are commonly about 99 dBA and may range up to 103 dBA over time (Scofield 2016). Table 9.3 summarizes the difference in OBSI results for several projects in which NGCS and conventional diamond grinding operations were performed at the same time and then monitored at several later points in time.

For all projects, the section with NGCS was quieter than the counterpart section with conventional diamond-ground surface at the time of construction, and NGCS has maintained a quieter level for all but two projects after from 1 to 6 years of service.

Table 9.3. Summary of differences in on-board sound intensity (OBSI) between projects constructed using conventional diamond grinding and NGCS

Agency	OBSI difference at time of construction, dBA*	Most recent OBSI-tested difference, dBA*(age when tested)
Arizona	-2.9	-1.6 (1 year)
California	?	-2 (≤4 years)
Illinois	-0.2	+0.4 (7 years)
Iowa	-1.3	-0.5 (1 year)
Kansas	-2.3	-0.3 (6 years)
Minnesota	-4.2	-0.8 (6 years)
Virginia	-3	+2.4 (2 years)
Average Difference	-2.3	-1.4

** Negative value indicates NGCS measured as quieter than its conventionally diamond-ground counterpart*

Source: Scofield 2016

Design Considerations

NGCS can be constructed on existing concrete pavements as well as on new concrete pavements. Although the reasons for considering NGCS for an existing pavement may be similar to those for performing diamond grinding (namely, fault removal, surface friction, and noise reduction), the most compelling reason for constructing NGCS is its impact on reducing noise emissions; therefore, an agency would be most likely to consider it for an existing pavement that exhibited critical noise issues. Additional benefits provided by NGCS, however, are the reduction in hydroplaning potential and the increased lateral stability compared to a transversely tined or CDG surface.

As previously described, NGCS is constructed using a two-part operation. The first operation creates the flush ground surface and is produced by a 4 ft minimum grinding head stacked with 0.125 in. wide blades separated by 0.035 in. (±0.005 in.) wide spacers; this results in 92 to 100 blades per foot (IGGA 2020d). The blades used to produce the flush ground surface should be flat across their contact surface and in the same plane with other flush grind blades when mounted. Prior to the second operation, the required smoothness levels from the first operation must be achieved. The second operation provides the longitudinal grooves using blades 0.095 in. (±0.05 in.) wide and cutting 0.125 to 0.1875 in. deep into the slab (IGGA 2020d). The longitudinal grooves are spaced 0.5 to 0.625 in. apart.

It should be noted that the recommended blade width for NGCS was recently reduced to a nominal dimension of 0.095 in. (from 0.125 in.) to further minimize NGCS skittering effects on vehicles (IGGA 2020d).

Construction Considerations

Equipment

For the two-pass NGCS operation, two separate pieces of equipment will be needed, one for the flush grinding and one for the longitudinal grooving. Each should be self-propelled and should be outfitted with the blade and spacer configurations presented previously.

Procedures

Prior to the start of an NGCS project, the construction of a short test section is recommended in order to demonstrate that the equipment and procedures used are capable of attaining the desired surface texture and smoothness requirements. As with conventional diamond grinding, NGCS texturing begins and ends at the project boundaries on lines perpendicular to the pavement centerline. The overlap of passes of the flush grinding head should not be more than 1 to 2 in. and no unground surface area should be permitted between passes. The passes of the grooving head should not overlap with the previous cuts.

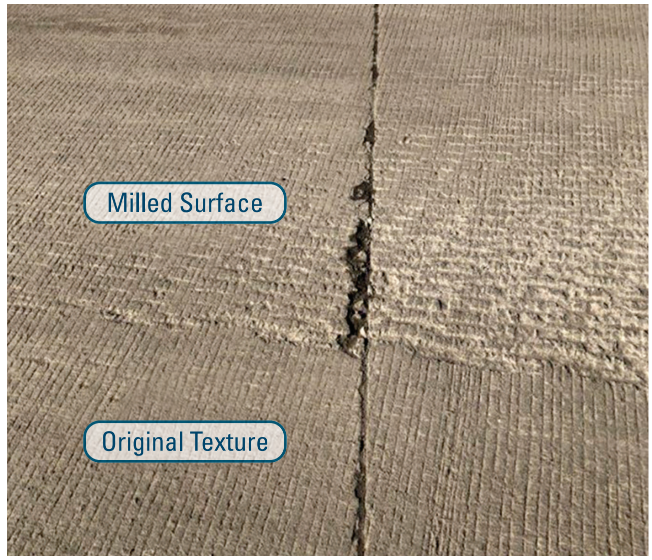
Quality Assurance

The recommended quality control activities for each aspect of the NGCS process are similar to those presented for diamond grinding and diamond grooving.

5. Cold Milling

Cold milling is an operation substantially different from diamond grinding and diamond grooving, and it is more commonly performed on bituminous pavements as a means of pavement removal. Cold milling equipment uses carbide bits mounted on a revolving drum to break up and remove the surface material, and the drums can range in size depending on the type of project. A recent innovation in the cold milling field is the use of micromilling, sometimes referred to as fine milling. Micromilling uses a greater number of carbide bits and a closer spacing of these bits on the drum to produce a smoother surface than that produced by conventional cold milling.

The primary difference between diamond grinding and cold milling is the way the concrete layer is removed. With diamond grinding, the diamond blades abrade away the concrete surface, while the cold milling head chips away at the pavement, leaving a rough surface and fractured joint faces (ACPA and IGGA 2001). While cold milling has been shown to be an effective and productive method of preparing small surface areas for PDRs (as described in Chapter 5), it is not recommended for restoring concrete pavement smoothness because it leaves a rough pavement surface and damages transverse and longitudinal joints (ACPA and IGGA 2001). This is illustrated in Figure 9.19.



Photograph provided by IGGA, used with permission

Figure 9.19. Joint damage caused by cold milling

6. Slurry Handling

The grinding and grooving operations described in this chapter produce a slurry that consists of the water used to cool the blades, hardened cement paste, and aggregate particulates (IGGA 2013). The composition of this slurry (sometimes referred to as concrete grinding residue [CGR]) can vary depending on the source concrete characteristics, pavement exposure conditions during service, and runoff. However, research has shown CGR to be an inert, nonhazardous byproduct of the grinding operation (IGGA 2013). In fact, several recent studies have demonstrated that the slurry resulting from grinding and grooving operations not only is safe but in many instances actually provides benefits to soil in terms of increasing its pH for enhanced plant growth and contributing minerals with nutritive benefits (Mamo et al. 2015, Townsend et al. 2016, Ceylan et al. 2019).

During the grinding and grooving operations, the slurry is picked up by onboard wet vacuums to be disposed of in accordance with local environmental regulations; this may include discharge along the roadside or transport to a settlement pond or processing plant.

An investigation of state transportation department requirements for dealing with slurry revealed substantial variability in management procedures, and in some cases the item is not addressed at all (Tymvios et al. 2019). When allowed, roadside discharge is most cost-effective, but slurry recovery provides the opportunity for potential reuse of the recovered material. A guide on diamond grinding slurry management best practices is available and provides the following recommendations regarding slurry disposal (IGGA 2013):

- **Slurry Spreading Disposal**—In rural areas with vegetated slopes, the slurry can be deposited on the slopes as the diamond grinding process progresses along the road, as shown in Figure 9.20. Wetlands and other environmentally sensitive areas where slurry discharge is not permitted should be clearly identified in the contract documents.
    - The engineer and the contractor should jointly conduct a site inspection prior to the start of the grinding to identify all sensitive areas.
    - All spreading start and stop points should be clearly marked on the shoulder of the road.
    - Slurry generated in wetland and other environmentally sensitive areas should be picked up and hauled for disposal to nonsensitive areas of the job.
    - The diamond grinding equipment should be equipped with a well-maintained vacuum system that can remove all standing slurry, leaving the roadway in a clean, damp condition after the grinder passes.
    - The slurry should not be allowed to flow across the roadway into adjacent lanes.
    - The vacuumed material should be spread evenly on the adjacent slopes by dragging a flexible hose or other approved device along the slope (see Figure 9.20).
    - The slurry material should not be spread on the shoulder.
    - The spreading of the slurry material should begin a minimum of 1 ft from the shoulder, with each pass of the grinder moving the spreading operation farther down the slope to ensure no buildup of grinding residue.
    - The slurry should not be spread within 100 ft of any natural stream or lake or within 3 ft of a water-filled ditch. Efforts should be made to restrict the spreading operation to the space above the high-water line of the ditch.
    - At no time should the grinding residue be allowed to enter a closed drainage system. The contractor is responsible for providing suitable means to prevent the infiltration of the grinding residue into all closed drainage systems.
- **Slurry Collection and Pond Decanting**—In urban and other areas with closed drainage systems, the slurry should be vacuumed and collected in watertight haul units and then transported to settlement ponds constructed by the contractor (see Figure 9.21).
    - Slurry settlement ponds may be constructed within or outside of the right-of-way. All pond locations should be approved by the engineer.
    - The ponds should be constructed to allow for the settlement of the solids and decanting of the water for reuse in the grinding operation.
    - At the completion of the grinding operation, the remaining water can be allowed to evaporate or may be used in a commercially useful manner (e.g., for dust control).
    - After drying, the remaining solids may be used as a fill material, a component in recycled aggregate, or any other commercially useful application.
    - The pond area should be reclaimed to its original condition and vegetated to protect against erosion.

John Roberts, IGGA, used with permission

Figure 9.20. Depositing diamond grinding slurry on vegetated slopes in rural area

John Roberts, IGGA, used with permission

Figure 9.21. Vacuum system (top), slurry haul truck tied to a diamond grinding machine (middle), and slurry settlement pond (bottom)

- **Slurry Collection and Plant Processing**—The slurry should be collected and hauled in a manner like that for pond processing.
  - Various plant designs are available that may be used, such as centrifuge and belt press designs.
  - The plant site should be prepared to control any storm water runoff in accordance with state regulations.
  - The site should be restored and vegetated at the completion of the operations.
  - Processed water and solids at plant sites should be handled in the same fashion as at settlement ponds.
  - The plant site may be within or outside of the right-of-way. Site locations are to be approved by the engineer.

Monitoring the pH of the slurry is an important part of the slurry management process and the contractor should have in place a pH control plan with both the spreading and pickup operation (IGGA 2013):

- The slurry should be managed to maintain a pH below 12.5 and greater than 2.
- The contractor should test the pH at least once per hour to ensure it is within the acceptable limits.
- The pH testing equipment should be calibrated daily and approved by the engineer.
- Once the pH control plan is operational and producing consistent results, the testing frequency may be reduced to 4 tests per day.
- The contractor shall log all test results and deliver a signed copy to the engineer on a weekly basis.
- At no time shall slurry containing a pH outside of the acceptable limits be allowed to be deposited on the ground. The contractor should determine the procedure to be used to maintain the pH within the acceptable range. This procedure should be approved by the engineer.

7. Troubleshooting

Potential construction problems that may be encountered with diamond grinding and diamond grooving are presented in Tables 9.4 and 9.5, respectively. Typical causes and recommended solutions are also provided in these tables.

Table 9.4. Potential diamond grinding construction/performance problems and associated solutions

Problem	Typical cause(s)	Typical solution(s)
"Dogtails" (pavement areas that are not ground due to a lack of horizontal overlap)	These are primarily caused by weaving during the grinding operation.	Maintaining the required horizontal overlap between passes (i.e., 1 to 2 in.) and steady steering by the diamond grinding machine operator will avoid the occurrence of dogtails.
"Holidays" (areas that are not ground)	These are isolated low spots in the pavement surface.	Lower the grinding head and complete another pass. Typical specifications require 95% coverage for the diamond-ground texture, but the required coverage can vary and will depend on the age and condition of the existing pavement.
Poor vertical match between passes	There is inconsistent downward pressure. This is often obtained when unnecessary adjustments to the down-pressure are made.	A constant down-pressure should be maintained between passes to maintain a similar cut depth.
Too much or too little material removed near joints	Expansion joints or other wide gaps in the pavement can cause the cutting head to dip if the leading wheels drop into these openings. Slabs deflecting from the weight of the grinding equipment can cause insufficient material to be removed.	Wide gaps can be temporarily grouted to provide a smooth surface. If the slabs deflect excessively from the weight of the grinding equipment, stabilizing the slab or retrofitting dowel bars may be needed.
Fins that remain after grinding not quickly breaking free	This could be an indication of excessive wear on the grinding head, but most likely it is the result of incorrect blade spacing.	The grinding head should be checked for wear before or after each day of operation. If the fins remaining after grinding are not quickly breaking free when the cutting blades are not worn, the blade spacing should be reduced.
Large amounts of slurry left on the pavement during or after grinding	Most likely this indicates a problem with the vacuum unit or skirt surrounding the cutting head.	If large amounts of slurry are left on the pavement or if slurry flows into adjacent traffic lanes or drainage structures, the diamond grinding operations should be stopped, the equipment inspected, and all necessary repairs made.
Vehicle tracking experienced by motorcycles and other lightweight vehicles	This indicates a problem with the spacing between the blades.	Reduce the spacing between the blades.

Sources: Adapted from ACPA 2000, ACPA 2006, FHWA 2019a

Table 9.5. Potential diamond grooving construction problems and associated solutions

Problem	Typical cause(s)	Typical solution(s)
Lack of horizontal overlap	This is primarily caused by weaving during the grooving operation.	Lack of horizontal overlap or weaving during grooving operations may cause lighter vehicles and motorcycles to experience increased vehicle tracking. Maintaining the required horizontal overlap between passes and steady steering by the diamond grooving equipment operator will minimize the occurrence of this problem.
Isolated areas with inconsistent groove depth	There are isolated low spots in the pavement surface.	Although the effects of variable-depth grooves are less readily apparent to traffic since no dip in the pavement surface is created, a uniform depth is desirable to ensure the intended drainage characteristics. The grooving head may need to be lowered in areas known to contain isolated low spots.
Inconsistent groove depth near joints	Expansion joints or other wide gaps in the pavement can cause the cutting head to dip if the leading wheels drop into these openings. Slabs deflecting from the weight of the grooving equipment can cause insufficient material to be removed.	Wide gaps can be temporarily grouted to provide a smooth surface. If the slabs deflect excessively from the weight of the grinding equipment, stabilizing the slab or retrofitting dowel bars may be needed.
Large amounts of slurry left on the pavement during or after grooving	This indicates a problem with the vacuum unit or skirt surrounding the cutting head.	If large amounts of slurry are left on the pavement or if slurry flows into adjacent traffic lanes or drainage structures, the diamond grooving operations should be stopped, the equipment inspected, and the necessary repairs made.

Sources: Adapted from ACPA 2000, FHWA 2019b

8. Summary

Diamond grinding and grooving are surface restoration techniques that have been used successfully to correct a variety of surface distresses on concrete pavements. The appropriate application of these techniques can result in a cost-effective extension of pavement life, particularly when these techniques are used in conjunction with other pavement preservation activities.

Diamond grinding uses closely spaced diamond saw blades to remove a thin layer of material from a concrete pavement surface. Although diamond grinding is primarily used to restore or improve ride quality by removing transverse joint faulting and other surface irregularities, other common usages of diamond grinding include improving skid resistance (by increasing macrotexture) and reducing tire-pavement interaction noise.

Diamond grooving is the use of diamond saw blades to cut longitudinal or transverse grooves into a pavement surface. The purpose of grooving is to provide channels on the pavement that collect water and drain it from the surface. A reduction in surface water translates into a reduction in the potential for the wet-weather crashes that are associated with hydroplaning as well as decreased splash and spray. Longitudinal grooving is commonly employed in local areas such as curves, where the grooves provide a tracking effect that helps hold vehicles on the road. For areas where increased braking resistance is required, transverse grooving is often used. Grooving is usually done on pavements that show little or no structural distress.

One other surface texturing process was also introduced in this chapter. NGCS was noted to be a manufactured, low-noise surface texture that can be applied to both new and existing concrete pavements. Information was presented on its use and application, along with general design considerations and construction guidelines.

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Chapter 10
Joint Resealing and Crack Sealing

1. Introduction

Joint resealing and crack sealing are concrete pavement preservation activities that serve two primary purposes. One purpose is to reduce the amount of moisture and deicing chemicals that can infiltrate a pavement structure, while the second is to prevent the intrusion of incompressible materials (sand, pebbles, and other solid debris) into the joint. In addition, keeping joints and cracks sealed also has the beneficial effect of reducing the noise emissions caused by "tire slap" or "joint slap" (ACPA 2007, Donavan 2010), which are a result of the vibration in the tire tread and carcass created by the impact with the pavement joint (SNS 2011a). Joint resealing and crack sealing operations are common preservation activities and are routinely performed by many state and local roadway agencies on their concrete pavement network.

This chapter presents information on the appropriate use and recommended installation procedures for joint resealing and crack sealing operations. It also provides information on quality assurance, construction inspection responsibilities, and troubleshooting and includes a brief description of surface sealers as a means of reducing the penetration of water into the surrounding concrete.

Some agencies differentiate between joint/crack sealing and joint/crack filling. In this case, sealing is defined as employing more rigorous preparation of the joint/crack channel, including the provision of a designed reservoir, along with the use of generally higher quality materials. Filling, on the other hand, is defined as involving minimal preparation and generally using lower quality materials. The focus of this chapter is on sealing as applied to either a joint or crack application.

Figure 10.1. Pumping (top), faulting (center), and corner break (bottom) distresses

2. Purpose and Project Selection

As described previously, one of the goals of joint and crack sealing is to minimize the amount of fluids that enter a joint. Free water and deicing chemicals that enter joints or cracks can accumulate beneath the slab or within the joint, contributing to the development of such distresses as pumping, loss of support, faulting, corner breaks, and concrete deterioration. Pumping, faulting, and corner breaks (see Figure 10.1) are three related distresses that often occur when concrete pavements are constructed on erodible bases and exposed to high truck-traffic levels.

A recent analytical study of moisture infiltration into concrete pavements confirmed the role of base erodibility, moisture levels (measured in terms of the number of wet days), and traffic loadings as key parameters affecting erosion of the base and subsequent joint faulting (Neshvadian et al. 2017).

The infiltration of deicing chemicals into the joints and cracks of concrete pavements has been a critical issue contributing to premature concrete deterioration in many northern snowbelt states (Weiss and Farnam 2015, Weiss et al. 2016). The deterioration primarily shows up as a degradation of the joints (see Figure 10.2) and has been attributed to two primary mechanisms: (1) the use of certain deicing chemicals—notably calcium chloride and magnesium chloride—that react with cement paste to form the expansive calcium oxychloride and (2) freeze-thaw damage when a critical degree of saturation is reached within a joint (Weiss et al. 2016).

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Figure 10.2. Joint deterioration of concrete pavement

Reducing the infiltration of water and deicing chemicals into the joints through joint sealing or preventing the ingress of deicing chemicals into the concrete itself through surface sealers (discussed later) are two ways of addressing this phenomenon.

The second primary goal of joint resealing is to reduce the infiltration of incompressible materials into joints or cracks; such materials can interfere with the normal opening and closing movements of joints, causing compressive stresses in the slab and increasing its potential for spalling (Figure 10.3 top). In the extreme case, if these compressive stresses exceed the compressive strength of the deteriorated pavement, blowups or buckling may occur (Figure 10.3 bottom).

Even if blowups do not occur, the continued intrusion of incompressibles such as dirt, debris, and sand may cause the pavement to "grow." This "growth" can force the movement of nearby bridge abutments or other pavement structures that may, over time, cause serious damage and necessitate major rehabilitation.

Application of Joint Resealing

Joint resealing should be performed when the existing sealant material is no longer performing its intended functions. As shown in Figure 10.4, this is indicated by missing sealant, sealant that is in place but not bonded to the adjoining joint faces, or sealed joints that contain incompressible materials.

CP Tech Center (top) and WSDOT, used with permission (bottom)

Figure 10.3. Concrete pavement joint spalling (top) and blowup (bottom)

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Figure 10.4. Examples of joint sealant failures: missing and debonded sealant (left) and incompressible materials in joint (right)

Some agencies specify that joints be resealed when a certain amount of sealant material (typically 25% to 50% of the length) has failed to perform one or both of its primary functions, whereas other agencies base their decision on pavement type, pavement and sealant condition, and available funding (Evans et al. 1999).

Although a visual evaluation can provide a good indication of whether a sealant is performing its functions, research by Bakhsh and Zollinger (2016) showed that the amount of water that infiltrates concrete pavements through the joints depends not only on the presence of sealant but also on the sealant type, bonding condition, and the amount of joint opening. Specifically, substantially higher rates of water inflow occur not only when there is no seal but also when there are higher levels of debonded sealant together with larger joint openings. That is, while the amount of sealant adhesive failure is a good general performance indicator, joints with wider openings will allow more water to enter the pavement for the same amount of debonded sealant length.

For concrete pavements with transverse joints that were initially sealed at the time of construction, the general recommendation is to continue to regularly reseal those joints over the life of the pavement. It is noted, however, that some agencies choose not to reseal transverse joints, but only when design factors (e.g., narrow joints and drainable bases), climatic conditions (e.g., low annual rainfall), or their local experience favor such a decision.

Application of Crack Sealing

Crack sealing is a comprehensive operation involving thorough crack preparation and the placement of high-quality materials into candidate cracks to significantly reduce moisture infiltration and to retard the rate of crack deterioration. Crack sealing is most effective on concrete pavements that exhibit minimal structural deterioration and can be performed on random transverse and longitudinal cracks up to 0.5 in. or more, provided that there is minimal spalling or faulting; cracks less than 0.12 in. wide are generally not sealed. Full-depth working transverse cracks can experience about the same range of movement as transverse joints, which is why it is recommended that these cracks be sealed to reduce the potential for infiltration by incompressibles and water. If the potential exists for full-depth working cracks to fault or spall, then retrofitted dowel bars should be installed across the cracks prior to the application of crack sealing (ACPA 1995).

3. Limitations and Effectiveness

The performance of joint and crack sealing treatments (i.e., how long they effectively perform their primary functions) varies considerably with the type of sealant material, the reservoir design, prevailing climatic conditions, and the quality of the installation process. For example, a Texas study of joint sealant performance revealed an average life of about 10 years for primarily silicone sealants (Choi et al. 2017). This is not to say, however, that longer performance lives are not possible. For instance, using nearly 7 years of performance data, the Strategic Highway Research Program (SHRP) H-106 joint resealing experiment extrapolated the performance life of several silicone sealants to be between 12 and 16 years (Evans et al. 1999). In addition, several follow-up pavement evaluations of earlier joint sealant studies documented sealant performance lives of more than 20 years for both silicone and hot-applied sealants on a concrete airfield pavement in Washington State and a similar 20-year service life for silicone sealants on a concrete highway pavement in Arizona (Scofield 2013).

The optimum time of the year to perform joint resealing is generally during the spring or fall when moderate installation temperatures are prevalent and the joint width is near the middle of its working range; however, it is also important that the prevailing conditions are dry and that the threat of condensation is low.

The greatest benefits from resealing can be expected when the pavement is not severely deteriorated and is still performing well and when the existing joint sealant warrants replacement. Joint resealing is often performed in conjunction with other pavement restoration activities, such as PDR, FDR, DBR, and diamond grinding, although none of these activities are a prerequisite for resealing.

Most joint sealing and resealing operations focus on the transverse joints, where it is more challenging to achieve an effective seal than with longitudinal joints because of the greater movement associated with transverse joints. However, the sealing of longitudinal joints is often done at the same time as that of transverse joints, and this has the potential for significant benefits as one study has indicated that as much as 80% of the water infiltrating a pavement structure does so through the longitudinal lane-shoulder joint (Barksdale and Hicks 1979). The importance of sealing the longitudinal lane-shoulder joint was demonstrated in a preventive maintenance study conducted at the Minnesota road research test facility, which showed that the amount of water entering a pavement system can be reduced by as much as 85% merely by sealing the joint between a concrete mainline pavement and the associated asphalt shoulder (Olson and Roberson 2003).

4. Sealant Material Selection

When planning a joint resealing project, one of the primary design activities is the selection of an appropriate sealant material. Sealant material selection is dependent on a number of factors, which include the following:

- Climate conditions (at the time of installation and during the life of the sealant)
- Joint/crack characteristics (widths and movements)
- Sealant material availability and cost

The first two factors govern the range of movement that the joints/cracks—and the installed sealant material—will experience. Because sealant materials have different extension properties, a sealant material must be selected that will be able to accommodate the maximum anticipated joint opening movement. A tool for estimating joint and sealant movement is available that can be used to assist in the sealant material selection process (ACPA 2020a). In addition, some sealant manufacturers indicate the applicability of their suite of products for ranges of expected pavement temperatures within selected climatic categories.

Available Types of Sealant Materials

Joint resealing and crack sealing operations generally employ either hot-applied asphalt sealant materials or cold-applied silicone sealant materials, as listed in Table 10.1.

Table 10.1. Common material types and related specifications for concrete pavement joint sealing and resealing

Material type	Specification(s)	Description
Hot-applied, formed-in-place, thermoplastic materials		
Polymerized/rubberized asphalts	ASTM D6690, Type I[1, 2]	Self-leveling, moderate climates, 50% extension at 0°F
	ASTM D6690, Type II[1, 3]	Self-leveling, most climates, 50% extension at -20°F
	ASTM D6690, Type III[1]	Self-leveling, most climates, 50% extension at -20°F, water-immersed bond testing, and aged resilience testing
	ASTM D6690, Type IV[1]	Self-leveling, very cold climates, 200% extension at -20°F
Cold-applied, formed-in-place, thermosetting materials		
Single-component silicone	ASTM D5893, Type NS	Non-sag, toolable, low modulus
	ASTM D5893, Type SL	Self-leveling, no tooling, low modulus
Elastomeric polymer (polyurethanes, polysulfides)	ASTM C920, Type S, Grade NS	Non-sag, toolable
	ASTM C920, Type S, Grade P	Self-leveling, no tooling
Preformed polychloroprene elastomeric materials (compression joint seals)		
Preformed compression seals[4] • Polychloroprene elastomeric (neoprene) seals • Lubricant	ASTM D2628 ASTM D2835	Jet-fuel-resistant preformed compression seals Used in the installation of preformed compression seals
Preformed expansion/isolation joint filler materials		
Preformed filler material	ASTM D1751 (AASHTO M 213)	Closed-cell polypropylene foam Asphalt-saturated fiberboard (nonextruding)
	ASTM D1752, Types I–IV (AASHTO M 153)	Sponge rubber, cork, and recycled PVC
	ASTM D994 (AASHTO M 33)	Bituminous
Backer rod materials[5]		
Closed cell	ASTM C1330, Type C ASTM D5249	Standard polyethylene foam Cross-linked polyethylene foam
Open cell	ASTM D5249	***Not recommended***
Bicellular	ASTM D5249	Outer: Cross-linked; Inner: Open-cell foam For cold-applied sealants only

[1]*AASHTO equivalent of ASTM D6690 (AASHTO M 324) was discontinued in 2013.*
[2]*ASTM D1190 was withdrawn in 2002 and replaced with ASTM D6690 (Type I).*
[3]*ASTM D3405 was withdrawn in 2002 and replaced with ASTM D6690 (Type II).*
[4]*The use of preformed compression seals in a particular resealing operation will depend on the condition of the joints being resealed.*
[5]*A few agencies no longer use backer rods because of concerns that they trap moisture within joints.*

Sources: Adapted from ACPA 2006, ACPA 2018

Note that even though preformed sealant types are included in Table 10.1, these materials see greater use in new pavement construction, particularly where long-term performance is sought (ACPA 2006). Their use in resealing operations may be precluded by various challenges, including uneven joint widths along individual joints, the presence of minor spalling along a joint, and nonuniform joint widths throughout a project. Also note that, while one- and two-component polyurethanes and polysulfides are available and listed in Table 10.1, they are more commonly used for other infrastructure (bridges, parking decks, buildings, etc.).

Hot-Applied Asphalt Sealant Materials

Hot-applied asphalt sealants are bitumen-based materials that typically soften upon heating and harden upon cooling, usually without a change in chemical composition. These sealants vary in their elastic and thermal properties and are affected by weathering to some degree. The materials are heated prior to installation, typically between 350°F and 400°F.

Polymer-modified/ground-tire-rubber-modified asphalt is the sealing industry standard. This material is produced by incorporating various types and amounts of polymers and/or melted rubber into asphalt cement. The resulting sealants possess a large working range with respect to low-temperature extensibility and resistance to high-temperature softening and tracking.

Softer grades of asphalt cement can be used to further improve low temperature extensibility. These low-modulus sealants are used for sealing operations in many northern states because of their increased extensibility.

Most of the high-quality hot-applied sealant materials are governed by ASTM D6690, which includes four classes of sealants to better match low-temperature performance with climate. The left photo of Figure 10.5 shows a transverse joint sealed with a hot-applied asphalt material.

Cold-Applied Silicone Sealant Materials

Cold-applied silicone sealants are one-component materials that cure through a chemical reaction. These sealants have demonstrated long-term performance capabilities as a result of their high extensibility, good bonding strength, and strong resistance to weathering. Although their material costs are typically higher than the standard hot-applied asphalt sealants, silicone sealants are often placed in a thinner application (due to their low modulus) than are hot-applied asphalt sealants. They may therefore have slightly lower labor and equipment costs because silicone sealants require less time for daily preparation and cleanup (e.g., they require no initial heating and no purging of lines and pump). The right photograph in Figure 10.5 shows a project with both the transverse and longitudinal joints sealed with a silicone material.

Silicone sealants are governed by ASTM D5893, which includes two classes of material—non-sag and self-leveling. The non-sag silicone sealants require a separate tooling operation to press the sealant against the sidewall and to form a uniform recessed surface. The self-leveling silicone sealants can be placed in one step because they flow freely and can fill the joint reservoir without tooling.

The performance of silicone sealants is typically tied to joint cleanliness, the presence of moisture, and tooling effectiveness. The type of aggregate in the existing concrete pavement, however, may also affect performance. For example, the adhesion of silicone sealant to concrete containing certain dolomitic limestones has been observed as an issue (McGhee 1995, ACPA 2018, FHWA 2019a).

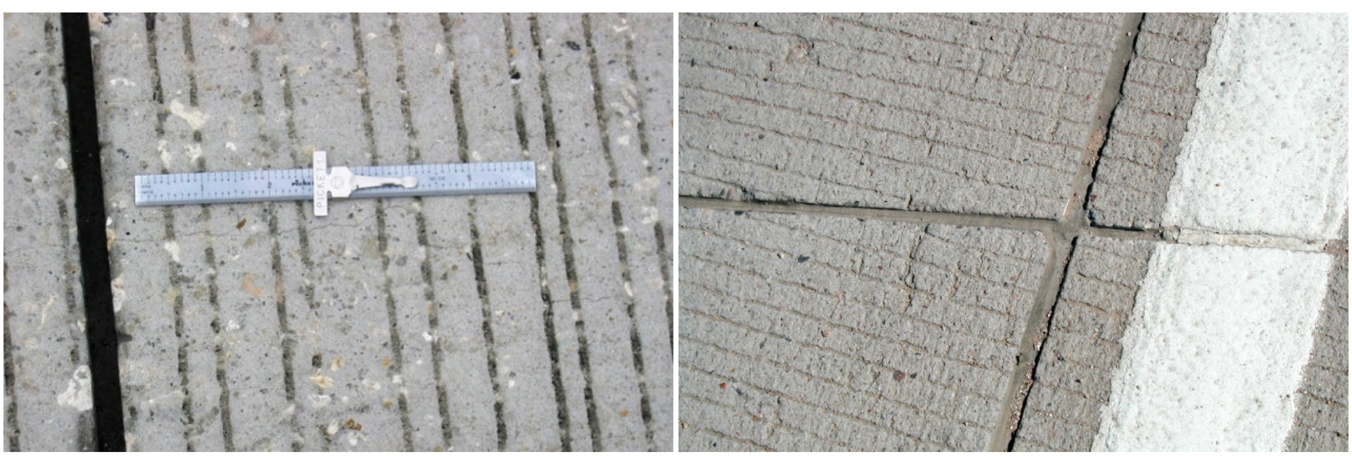


Figure 10.5. Hot-applied sealant (left) and silicone sealant (right)

Sealant Properties

Critical sealant properties that significantly affect the performance of sealant materials include:

- **Durability**—Durability refers to the ability of the sealant to resist deterioration when exposed to the effects of climatic factors such as moisture, ultraviolet rays, and ozone effects. A sealant that is not durable will blister, harden, and crack in a relatively short period of time.
- **Extensibility**—The extensibility of a sealant refers to the ability of the sealant to stretch or deform without rupturing. The more extensible the sealant, the lower the internal stresses that might cause rupture within the sealant or at the sealant-sidewall interface. Sealant extensibility is most important under cold conditions because maximum joint and crack openings occur in colder months. Softer, lower modulus sealants tend to be more extensible, but they may not be stiff enough to resist the intrusion of incompressible materials during warmer temperatures or provide the necessary bond to the joint face.
- **Resilience**—Resilience refers to the sealant's ability to fully recover from deformation and to resist stone intrusion. In the case of hot-applied asphalt sealants, however, resilience and resistance to stone intrusion are often sacrificed in order to obtain extensibility. This compromise is generally warranted, taking into consideration the expected joint or crack movement and the presence of incompressible materials in specific climatic regions.
- **Adhesiveness and Cohesiveness**—As sealant material in a joint or crack is elongated, high stress levels can develop, such that the sealant material is separated from the sidewall (adhesive failure) or the material internally fails or ruptures (cohesive failure). Sealant adhesiveness is one of the most important properties of a good sealant, and often the cleanliness of the joint or crack sidewalls is what determines a sealant's bonding ability. Cohesive failures are most common in sealants whose depth of placement is too thin and/or sealants that have hardened significantly over time, losing their elasticity.

Other sealant properties that may warrant consideration include jet fuel resistance and the compatibility of the sealant with the existing concrete or with the accompanying backer rod (ACPA 2018).

Cost Considerations

Hot-applied asphalt sealants are generally less expensive than cold-applied silicone sealants, but these costs can vary considerably geographically and by the size of the project and the design of the joint reservoir. In addition, the life expectancy should also be considered as some of the longer-lasting sealant materials may have a higher initial cost, but because of their extended life may incur a lower life-cycle cost.

5. Design Considerations

After the selection of a suitable sealant material, a joint resealing or crack sealing project requires decisions to be made regarding the design of the sealant reservoir. The design must consider the primary resealing objectives of reducing the infiltration of moisture and preventing the intrusion of incompressible materials. In addition, in locations where noise emissions may be an issue, the design should also consider the tire slap noise generated by vehicle tires as they pass over transverse joints in the pavement. In general, wider and deeper joint openings and closer joint spacings increase the overall tire-pavement noise. A tool is available that can be used to evaluate the impact of joint geometry (sealed or unsealed) on existing tire-pavement noise levels (ACPA 2020b).

Transverse Joints

In new concrete pavement design, the selection of appropriate joint sealant reservoir dimensions is primarily dependent on the expected joint movement due to climatic conditions, moisture conditions, and traffic loads, combined with the specific properties of the selected sealant material. In a joint resealing operation, however, the width of the joint is already determined, and it is generally desirable to limit the amount of widening that is done to minimize material requirements and the potential tire slap that is created by excessively wide joints. Consequently, the primary consideration in joint resealing is the selection of an appropriate joint shape factor for the sealant in order to accommodate the anticipated joint opening movement.

As previously noted, a tool is available for estimating joint openings and corresponding sealant elongations (ACPA 2020a). One additional item to consider when estimating joint movements is the possibility that some joints along a project experience little or no movement (Bakhsh and Zollinger 2016). The theoretical movements at these joints are generally absorbed by adjacent free-moving joints, and thus the additional movements experienced by the adjacent joints should be considered as part of the joint seal reservoir design.

Joint Shape Factor

Sealant Stresses

The performance of both hot-applied asphalt and cold-applied silicone materials depends on the stresses that develop in the sealant. Pioneering research dating back to the 1950s (Tons 1959) showed that the stresses that occur in a given sealant material are primarily a function of the shape of the sealant at the time it is poured. Figure 10.6 illustrates the stresses produced in sealants placed to different depths in the joint.

As each sealant material is elongated (simulating the opening of the joint), the sealant placed to a greater depth experiences much greater stresses than the sealant placed to a shallower depth. These higher stresses result from the "necking down" effect that occurs as the sealant is stretched. The material attempts to maintain a constant volume, but it is restrained at the reservoir faces by adhesion to the pavement.

The dimensions of the in-place sealant are described in terms of a "shape factor." The shape factor is defined as the ratio of the sealant width (W) to the sealant depth (D), as illustrated in Figure 10.7.

A proper shape factor minimizes the stresses that develop within the sealant and at the sealant/pavement interface as the joint opens. The backer rod, as shown in Figure 10.7, helps to achieve the desired shape factor and also inhibits the sealant from bonding to the bottom of the reservoir and from entering the crack at the bottom of the reservoir.

The backer rod must be flexible, compressible, nonabsorbent, and compatible with the selected sealant material. The diameter of the backer rod should be selected such that its uncompressed width is about 25% larger than the width of the joint or crack in which it will be placed. Backer rods are commonly manufactured from polyethylene, polyurethane, polychloroprene, or polystyrene; materials such as paper, rope, or cord should not be used (ACPA 2006).

Various types of backer rods are available, as shown in Figure 10.8.

The use of closed-cell, cross-linked, and bicellular backer rod products are recommended because they are more resistant to moisture absorption than open-cell materials (SNS 2011b). Open-cell backer rods can retain moisture in a way that contributes to detrimental joint sealant performance and are therefore not recommended (ACPA 2018, Tompkins and Khazanovich 2019).

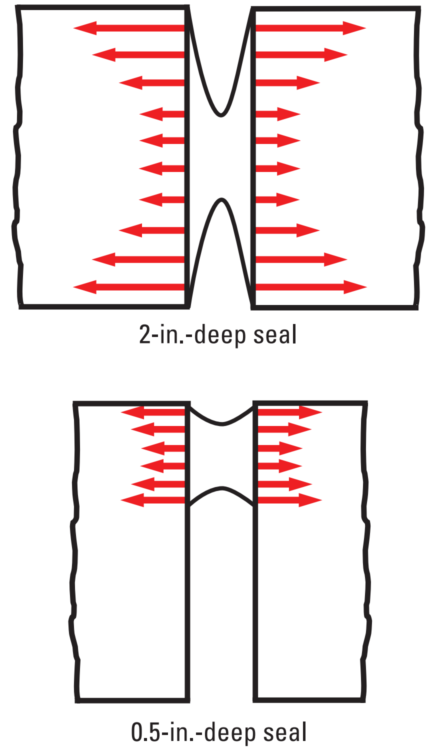


Figure 10.6. Relative effect of sealant depth on sealant stresses

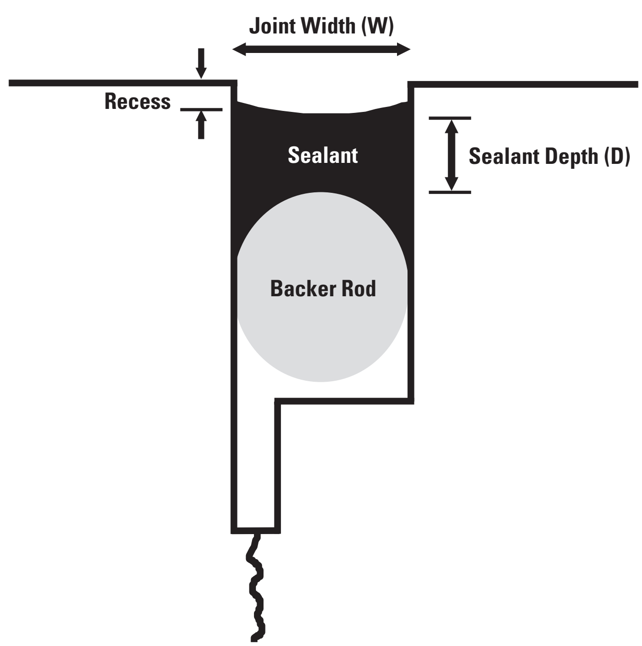
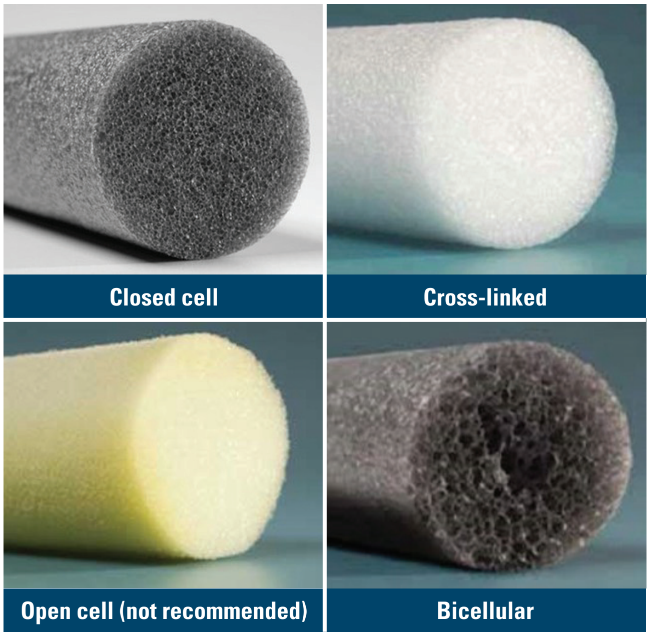


Figure 10.7. Schematic of joint sealant reservoir



Jerry Voigt, from ACPA members

Figure 10.8. Types of backer rod materials

Purdue University from Taylor et al. 2012

Figure 10.9. Deterioration below the joint sealant

In some cases, the use of a backer rod in joint sealing operations should be considered with caution. For example, one study suggests that a backer rod may trap water beneath joint sealant, leading to a critically saturated condition that can contribute to the deterioration of the concrete at or below the joint, as shown in Figure 10.9 (Taylor et al. 2012).

The issue of backer rods trapping water beneath joint sealant may be particularly problematic when the concrete is of marginal durability. Because of this potential problem, a few agencies no longer use backer rods but instead fill the entirety of joint reservoirs with hot-applied sealant. Ideally, the sealant in this case should penetrate to the bottom of a reservoir, but this can be difficult to achieve with narrow joints. As a general rule, a joint opening should be at least 0.19 in. wide to allow sufficient penetration of the hot-applied sealant into the joint reservoir (SNS 2020).

Recommended Shape Factors

The design of a sealant reservoir (i.e., determining how wide to saw the joint and how deep to place the sealant) should take into consideration the amount of strain or deformation from stretching that the sealant will experience. Most hot-applied sealants are designed to withstand strains of roughly 25% to 35% of their original width, whereas silicone sealants are designed to tolerate strains from 50% to 100%. For example, a hot-applied sealant placed in a 0.5 in. wide joint can withstand an opening of 0.125 in. (0.5 in. x 25%) before exceeding a strain of 25%. A silicone material placed in a 0.5 in. wide joint can withstand an opening of 0.25 in. (0.5 in. x 50%) before exceeding a strain of 50%. Recommended shape factors for the various sealant types are presented in Table 10.2, but note that many products also have minimum thickness (depth) requirements, as indicated by the manufacturer.

Table 10.2. Typical recommended shape factors

Sealant material type	Typical shape factor (W:D)
Hot-applied asphalt	1:1
Cold-applied silicone	2:1
Polysulfide and polyurethane	1:1

Source: Evans et al. 1999

Longitudinal Joints

Because of their limited amount of movement, concrete-to-concrete longitudinal joints rarely have a designed reservoir. These joints are typically very narrow (around 0.25 in. wide) and are commonly sealed or filled with a hot-applied material. A backer rod is often not used.

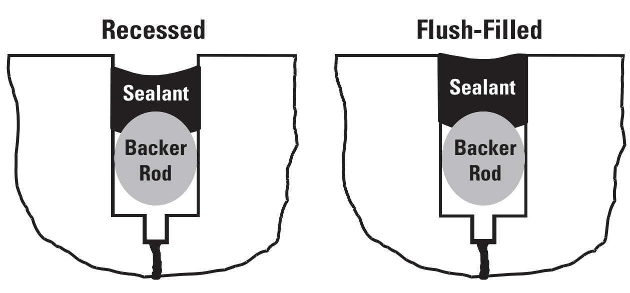
Longitudinal joints between a mainline concrete pavement and an HMA shoulder are particularly difficult joints to seal because of the differential vertical movement that occurs between the two materials (Barksdale and Hicks 1979). The differential vertical movements for the two materials are due both to the structural differences between their cross sections and to the differences in their thermal properties. Settlements or heaving of the shoulder are quite common along such joints, so they often will require a wider reservoir to withstand that vertical movement. A reservoir configuration of either 0.75 in. by 0.75 in. or 1 in. by 1 in. for the longitudinal concrete mainline–HMA shoulder joint is suggested in order to accommodate the anticipated movements.

Sealant Configurations

For joint resealing, the sealant material can be placed in two basic configurations, as shown in Figure 10.10.

Silicone sealants are always placed in the recessed configuration, with a typical recess of about 0.25 to 0.38 in. for most roadway joint widths. Silicone sealant manufacturers will provide recommendations on the amount of recess for their product as applicable to the given joint design.

Hot-applied sealants can be placed in either the recessed or flush fill configuration. Some manufacturers of these materials recommend the flush fill configuration, which keeps the sealant material ductile under the kneading action of passing tires and eliminates space that could otherwise accumulate sand, pebbles, and other debris. Also, with their reduced exposure to standing water, sealants placed in the flush fill configuration experience less age-hardening damage. This was demonstrated in a long-term evaluation of sealants placed on an airfield, where the flush fill configuration was found to increase the life of hot-applied sealants by more than 50% when compared to those placed in the standard recessed configuration (Lynch et al. 2013).



Adapted from Smith et al. 2014, CP Tech Center

Figure 10.10. Joint sealant configurations

Cracks

Crack seal design should largely follow the same general approach as transverse joint reseal design, particularly if the cracks are full-depth transverse working cracks. A diamond-bladed saw or router can be used to create a reservoir for the crack, the width of which will generally be governed by the upper end of the range of crack widths that exist throughout the crack sealing project to enable the use of a standard width. Many agencies seal cracks between 0.25 to 0.5 in., although some agencies may consider sealing cracks that are more than 0.5 in. wide, provided there is limited spalling or faulting.

6. Construction Considerations

After the sealant material has been selected for a joint resealing and/or crack sealing project, careful attention must be paid to the installation procedure to ensure the successful performance of the sealant. Many projects have performed poorly because of improper or inadequate construction practices. Therefore, this section presents the recommended procedures for an effective sealant installation.

Transverse Joint Resealing

The resealing of transverse joints in concrete pavements consists of the following steps, each of which is described in detail in the subsequent sections:

1. Old sealant removal
2. Joint refacing
3. Joint reservoir cleaning
4. Backer rod installation
5. New sealant installation

Step 1: Old Sealant Removal

The first step of the joint resealing process is to remove the old sealant from the joint, along with any incompressible materials. Initial removal of old sealant from a joint can be done by any procedure that does not damage the joint itself, such as by using a rectangular joint plow or diamond-bladed saw (see Figure 10.11).

Scott Eilken, Quality Saw & Seal, Inc., used with permission

Figure 10.11. Sealant removal through joint sawing

Because of how diamond-bladed sawing combines the sealant removal and joint refacing steps into a single process, the procedure has gained widespread acceptance. Diamond-bladed sawing is most effective at removing existing asphalt-based sealants after they have become hardened enough that they will not melt nor gum up the saw blades or joint faces.

If a joint plow is used, it should be rectangular and fit into the joint without causing any spalling damage at the top of the joint face.

Step 2: Joint Refacing

The purpose of the refacing operation is to provide a clean surface that will bond with the new sealant and to establish a reservoir with the desired shape factor. If a diamond-bladed saw is used for sealant removal, refacing can be performed at the same time. If a joint plow or some other means has been used to remove old joint sealant material, then a separate joint refacing operation must be performed. Ideally, a joint should be widened by no more than 0.08 in. total during the refacing operation to limit the amount of concrete removed, increase production, and keep the width of the joint as narrow as possible.

Refacing is generally done using a water-cooled saw with diamond blades. These saws may use a single sawblade or may use multiple blades ganged together to provide the desired cutting width.

Although skimming the edges of each joint face using a single-bladed saw can serve this purpose, on wider joints the skimming method is more challenging to implement and usually results in a variable joint reservoir width. A gang-bladed saw, on the other hand, will remove more concrete than the edge-skimming approach but also produces a more uniform reservoir width that can lead to better sealant performance.

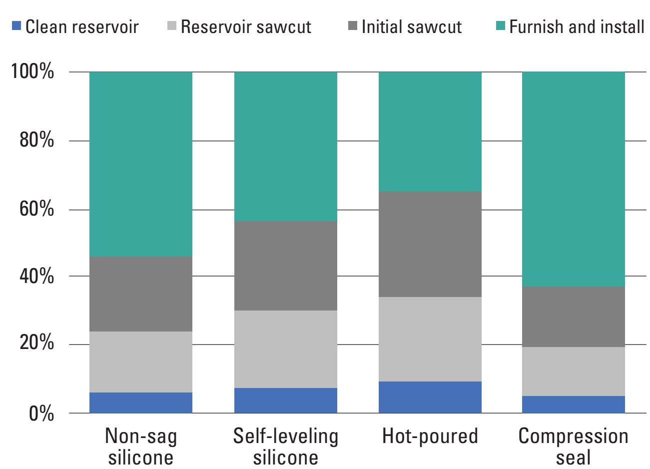
The use of pavement routers is not recommended for joint refacing operations because routers can excessively spall joints and may smear the existing sealant on joint sidewalls. Routers are, however, sometimes used in crack sealing operations.

Step 3: Joint Reservoir Cleaning

The effective cleaning of the joint sidewalls is the most important aspect of the joint sealing process. Dirty or poorly cleaned joint or crack sidewalls can reduce the performance of even the best sealant and the most reliable sealant reservoir design. Several common materials that may contaminate joint sidewalls include the following:

- Old sealant left on the joint or crack sidewalls
- Water-borne dust (laitance) from the sawing operation
- Oil or water introduced by the compressed air stream
- Dust and dirt not removed during the cleaning operation
- Debris entering the joint after cleaning and prior to sealing
- Other contaminants that may inhibit bonding, such as moisture condensation

Effective cleaning is essential to the performance of the sealant material, and the cleaning activity is not a costly endeavor. Figure 10.12 shows the breakout of installation costs for a range of sealing materials, and in all cases the cleaning costs (shown in blue) are less than 10% of the total installation expense (ACPA 2018).



Adapted from ACPA 2018, used with permission

Figure 10.12. Relative cost of joint sealant installation steps

Joint cleaning with high-pressure air or a water wash should commence immediately after joint refacing to remove slurry (ACPA 2018). After the joint has dried, media blasting is performed (see Figure 10.13) to remove laitance (wet-sawing dust) and any other residue. The media-blasting operation should proceed along each side of the joint, and the nozzle should be directed at an angle to the joint faces to clean the top 1 in. of the joint (ACPA 2018). The rate of progression along the joint should be slow enough such that the joint sidewalls are effectively cleaned yet fast enough that spalling of the joint edge or other joint damage does not occur.

The air compressors used with the media blasters must be equipped with working water and oil traps to prevent contamination of the joint bonding faces (ACPA 2018). Compressors should be tested prior to media-blasting operations using a clean white cloth to ensure oil- and water-free operations. Water blasting may occasionally be used for cleaning in applications where media blasting is not permitted. The use of hot-air lances to dry joint reservoirs should be done with caution, as overheating can damage the concrete (ACPA 2004).

For worker protection, the media-blasting equipment should include a remote shutoff valve and protective clothing for the operator (Evans et al. 1999). When performing media blasting, workers should follow all safety requirements as outlined by OSHA in 29 C.F.R. § 1926.1153 (2016).

Following media blasting and immediately prior to backer rod and sealant installation, the joints should be blown again with high-pressure, clean, dry air to remove media particles, sand, dust, and other incompressible materials that remain in the joint. The compressor should deliver air at a minimum of 120 ft^3/min and develop at least 90 lbf/in^2 nozzle pressure to be effective (ACPA 2018). Joints and surrounding surfaces should be air-blown in one direction away from prevailing winds, taking care to not contaminate previously cleaned joints or to blow debris into traffic in adjacent lanes.

Power-driven wire brushes should never be used to remove old sealant or to clean a joint in a concrete pavement. This procedure is ineffective, and it can smear the old sealant across the concrete sidewall and/or leave a sheen on the surface to which the new sealant cannot bond.

Scott Eilken, Quality Saw & Seal, Inc., used with permission

Figure 10.13. Media blasting along joint

Step 4: Backer Rod Installation

As previously described, the backer rod must be a flexible, nonabsorbent material that is compatible with the sealant material in use. Closed-cell products (see Figure 10.8) are recommended because of their nonabsorbent nature. The melting temperature of the backer rod material should be at least 25°F higher than the sealant application temperature to prevent damage during sealant placement (ACPA 2006).

The backer rod should be installed in the joint with a properly fitted backer rod insertion tool as soon as possible after the joints are air blasted. The backer rod material should be about 25% larger in diameter than the joint width to ensure that it fits snugly in the joint and will not move. The backer rod should be installed to the proper depth, and no gaps should exist at the intersections of backer rod strips. The rod should be stretched as little as possible to reduce the likelihood of shrinkage and the resultant formation of gaps.

Because joint widths can be expected to vary over the length of a project, various backer rod sizes should be available.

Figure 10.14 illustrates the insertion of a backer rod that is the proper diameter into the reservoir using a handheld roller.

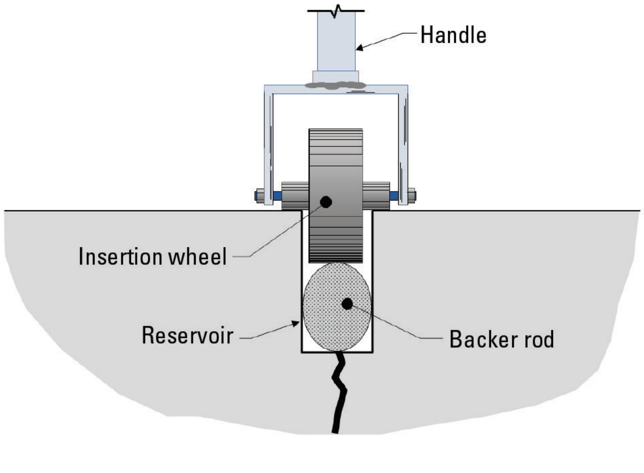


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Figure 10.14. Backer rod insertion with a handheld roller

IGGA, used with permission

Figure 10.15. Installation of hot-applied asphalt joint sealant

Step 5: New Sealant Installation

As soon as possible after backer rod placement, the sealant material should be installed. This helps to avoid the problems that occur when the backer rod is left in place too long before the sealant is placed, such as condensation on the backer rod and debris collecting in the reservoir. An additional check to verify that the reservoirs are clean and dry helps to ensure good long-term performance. Reservoir-sidewall cleanliness can easily be checked with a finger swipe or by using the black cloth wipe test (ACPA 2017).

Hot-Applied Asphalt Sealant Materials

Hot-applied asphalt sealants should be placed only when the air temperature is at least 40°F and rising (FHWA 2019b). The sealant material should be installed in a uniform manner, filling the reservoir from the bottom up to avoid trapping any air bubbles.

For recessed configurations, the joint reservoir should typically be filled no higher than 0.125 to 0.25 in. below the surface of the pavement to allow room for sealant expansion during the summer when the joint closes, thus preventing the sealant from being pulled out by traffic. For flush fill configurations, the joint reservoir should be overfilled and the sealant struck off as needed to form the specified configuration. In each case, to avoid "tracking" of the sealant, traffic should not be allowed on the newly sealed joints for about 30 minutes to 1 hour after sealant placement. The sealant manufacturer should be consulted for recommendations on when the sealant can be exposed to traffic. Figure 10.15 shows the installation of a hot-applied asphalt sealant material in a joint resealing project.

It is important to follow the manufacturer's recommendations with regard to the maximum sealant temperature, the recommended placement temperature, and any prolonged heating limitations. Many hot-applied asphalt sealants break down or degrade when subjected to temperatures above the recommended safe heating temperature. Prolonged heating can cause changes in the viscous properties of some sealant materials (causing them to gel in the heating tank), while for other sealant materials the prolonged heating may lead to changes in their elastic properties. In addition, sealant material that has been overheated tends to burn onto the hot surfaces of the inside of the melter/applicator. This burnt material, if remixed into new sealant, can reduce sealant performance.

In addition to the built-in thermometer on the melter/applicator, the use of a second external thermometer to monitor sealant temperatures can help prevent damage due to sealant overheating.

Cold-Applied Silicone Sealant Materials

Silicone sealants should not be placed at temperatures below 40°F. As with the hot-applied asphalt sealants discussed previously, silicone sealants should be installed in a uniform manner from the bottom to the top of the joint to ensure that no air is entrapped. Low-modulus silicone sealants have properties that allow them to be placed with shape factors of 2. It is recommended that they be placed thinner than half the width of the joint, with a minimum thickness of 0.25 in. For narrow joints (say, 0.25 in. wide), a 1:1 shape factor will be required, in accordance with the manufacturer's recommendations.

Non-sag silicones will be tack-free and may be opened to traffic in 25 to 90 minutes, whereas self-leveling silicones become tack-free and may be opened to traffic in about 3 hours—but it should be noted that opening times for both will depend on the ambient conditions (e.g., temperature and humidity). Again, the sealant manufacturer should be consulted for recommendations on when the sealant can be exposed to traffic.

As mentioned previously, silicone materials come in two varieties: non-sag and self-leveling. The non-sag silicone sealants must be tooled to force the sealant around the backer rod and against the joint sidewalls. This tooling should also form a concave sealant surface with the lowest point being about 0.25 in. below the pavement surface. Successful tooling has been accomplished using such devices as a rubber hose on the end of a fiberglass rod or pieces of a large-diameter backer rod. Tooling should be done within about 10 minutes of sealant installation before the sealant begins to skin over. Figure 10.16 shows a close-up of the silicone sealant installation. Note that the material has yet to be tooled.

Self-leveling silicone sealants do not require this tooling operation. Extra care, however, must be taken with placing the backer rod for self-leveling silicone sealants, because this type of sealant can easily flow around a loose backer rod prior to curing and may flow out at the joint ends if not properly blocked.

When installing both silicone and hot-applied sealants on the same project (such as silicone sealant in the transverse joints and hot-applied sealant in the longitudinal joint), the silicone should be installed first to reduce the potential for contamination of the transverse joint during the longitudinal joint sealing operations.

Expansion Joint Resealing

Expansion joints are special-use joints installed in new pavements to accommodate slab movements and to relieve pressure buildup from thermal changes in the concrete. Typically located near bridge ends and between intersecting concrete roads, these joints are usually 1 to 2 in. wide and include a preformed filler material (such as those listed in the previous Table 10.1) placed 0.75 to 1 in. below the slab surface. A hot-applied joint sealant is then applied as a “cap” to seal the expansion joint (see Figure 10.17), although appropriately sized preformed compression seals may also be employed in lieu of the sealant-and-filler combination.

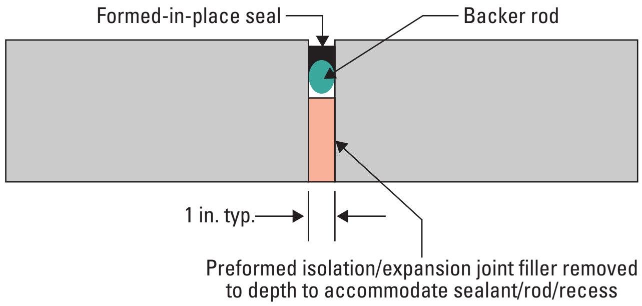
Over time, expansion joints start to close, possibly resulting in extrusion of the sealant and subsequent loss of material as traffic pulls at the exposed material above the pavement surface. Thus, periodic resealing of these joints is required. This resealing process is similar to the process used to reseal transverse contraction joints. After removing the remaining old sealant, a new joint reservoir must be established by sawing the joint slightly wider and sufficiently deep; if specified, the depth must be sufficient to allow for the placement of a backer rod to achieve a proper sealant shape factor (see Figure 10.18).

ACPA, used with permission

Figure 10.16. Close-up of silicone sealant installation

John Donahue, MoDOT, used with permission

Figure 10.17. New expansion joint (top) and older expansion joint with extruded sealant shoulder (bottom)



Adapted from ACPA 2018, ©ACPA 2018, used with permission

Figure 10.18. Expansion joint detail with a backer rod installed to ensure proper joint shape factor

As a result of this sawing operation, the top portion of the preformed filler material may be removed. If the filler material is deteriorated, it may need to be replaced over the entire length of the joint.

Once the new joint reservoir has been media blasted and the backer rod has been inserted to the proper depth, the new sealant can be installed. The sealant material should be installed from the bottom up and should be recessed at the surface to avoid extrusion (and possible damage under traffic) during joint closures.

Longitudinal Joint Resealing

As previously described, two types of longitudinal joints in concrete pavements may also be addressed as part of a resealing operation: longitudinal joints between adjacent concrete pavement slabs and the longitudinal joint between a mainline concrete pavement and an HMA shoulder. Although the procedures are essentially the same as transverse joint resealing, some additional considerations are described below.

Concrete-Concrete Longitudinal Joints

Longitudinal joints between adjacent concrete slabs are found between adjacent traffic lanes or between a concrete mainline pavement and a concrete shoulder (or a concrete curb and gutter). This joint is generally tied together with deformed tie bars so that movements are not excessive and conventional joint sealing operations can be followed. In the resealing operation, typically no reservoir is formed or needed. Because of the limited amount of movement that occurs at these joints, they are often sealed with a hot-applied sealant material. A few agencies may use silicone for longitudinal joints, particularly if they are already using silicone in the transverse joints.

Concrete Mainline–Hot-Mix Asphalt Shoulder Longitudinal Joints

The longitudinal joint between a concrete mainline pavement and an HMA shoulder can be a very difficult joint to seal. The differences in the thermal properties of each material and the differences in the structural cross section often result in large differential horizontal and vertical movements. Also adding to this movement can be the curling and/or warping of the concrete, the inability to tie the concrete mainline and the HMA shoulder, and the potential for frost heave or swelling in the subgrade beneath the shoulder.

Again, the steps required for the sealing of lane-shoulder joints are the same as for transverse joint resealing operations. It is important, however, that a sufficiently wide reservoir be cut in the existing HMA shoulder to allow for the anticipated vertical movements. Common reservoir dimensions range from 0.75 by 0.75 in. to 1 by 1 in. The reservoir can be created using either a router or a diamond-bladed saw. Figure 10.19 shows the preparation and sealing of the longitudinal joint between a concrete mainline and an asphalt shoulder.

Scott Eilken, Quality Saw & Seal, Inc., used with permission

Figure 10.19. Routing (top) and sealing (bottom) of a longitudinal joint between a concrete mainline and asphalt shoulder

The joint reservoir between a concrete mainline and asphalt shoulder should be cleaned prior to the placement of sealant material. A backer rod is generally not needed if proper depth control during the creation of the reservoir has been maintained. Many agencies use hot-applied asphalt materials to seal this joint, but there are also some silicone products that have been specifically developed for the concrete mainline–HMA shoulder application.

Crack Sealing

Except for the sealant removal step, the sealing of cracks in concrete pavements essentially follows the same basic steps as the resealing of joints: refacing, cleaning, backer rod installation, and sealant installation (ACPA 1995). The first step is to reface the crack to the desired width to allow for a proper sealant shape factor that can accommodate the expected crack movement. This can be achieved with a small-diameter, diamond-bladed saw (ACPA 2006). While the cutting blades for crack saws are typically 7 to 8 in. in diameter and 0.25 to 0.5 in. wide, smaller-blade diameters in addition to lightweight two- or three-wheel unit designs may be needed for particularly irregular cracks. These saws can pivot more easily and therefore are able to more closely follow a crack profile, leading to a more uniform sealant reservoir positioned directly over the crack.

Crack routers are not typically recommended for use on concrete pavements because of the chipping and microcracking damage this equipment can cause in concrete (ACPA 2004). However, for cracks that measure around 0.5 in. wide at the pavement surface, routers are generally more effective and more productive than crack saws.

Once a reservoir is created, a crack should be cleaned following the steps prescribed earlier for joint resealing. Media blasting is particularly recommended to remove laitance from the sawing operation. After cleaning, a crack should be blown with high-pressure compressed air, and the backer rod (if specified) and sealant material should be installed. The same precautions that apply to the installation of sealant materials into joints also apply here (ACPA 1995). Figure 10.20 shows a sealed transverse crack on a concrete highway.

7. Quality Assurance

Proper sealant application is a process that relies heavily upon the care and conscientiousness of the contractor. Paying close attention to quality during construction greatly increases the chances of minimizing premature failures on joint resealing and crack sealing projects. The remainder of this section summarizes key quality control recommendations as presented in the CP Tech Center's *Joint and Crack Sealing of Portland Cement Concrete Pavements* checklist (FHWA 2019b).

Preliminary Responsibilities

Agency and contractor personnel should collectively conduct a review of the project documentation, project scope, intended construction procedures, materials usage, and associated specifications. Such a collective review is intended to minimize any misunderstandings in the field between agency designers, construction personnel, and inspectors. Specific items for this review are summarized on the following pages.

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Figure 10.20. Sealed transverse crack

Project Review

An updated review of the current project's condition is warranted to ensure that the project is still a viable candidate for joint resealing. Specifically, the following items should be verified as part of the project review process:

- Review selected joint condition to verify that the specified joint size is appropriate.
- Verify that pavement conditions have not significantly changed since the project was designed and that joint sealing is appropriate for the pavement.
- Verify that joint design and sealant type are appropriate for the project climate and conditions.
- Verify that joint sawing and cleaning methods are appropriate.
- Verify that methods to remove old sealant materials are appropriate and in accordance with resealing project specifications.

Document Review

The key project documents should be reviewed prior to the start of any construction activities. Some of the critical project documents include the following:

- Bid/project specifications and design
- Special provisions
- Traffic control plan
- Manufacturer's sealant installation instructions
- Sealant MSDSs
- Applicable OSHA safety requirements

Materials Checks

In preparation for a construction project involving joint or crack sealing, the following list summarizes many of the materials-related items that should be checked or reviewed prior to construction:

- The sealant conforms to project specification requirements.
- The sealant is from an approved source or is listed on the agency's QPL (if required by the specification).
- The sealant has been sampled and tested prior to installation (if required) or appropriate certification has been submitted and approved.
- The sealant packaging is not damaged in a way that will prevent proper use (e.g., boxes leaking and pail or drums dented or pierced).
- Primer, if used, meets specification requirements.
- The backer rod is of the proper size and type for the selected sealant type and installation requirements.
- The sealants are within the manufacturer's recommended shelf life.
- Sufficient quantities of all materials are available for the completion of the project.

Equipment Inspections

Prior to the start of construction, all construction equipment must be examined. The following sections describe equipment-related items (specific to the different available sealant types) that should be checked prior to construction.

Hot-Pour Sealant Melters

- For hot-applied sealants, an indirectly heated, double-boiler-type melter with effective agitation is being used.
- Melters are in good working order with all heating, agitation, pumping systems, valves, thermostats, and other parts functioning properly.
- Melter heating system is thermostatically controlled and maintains the product at the recommended installation temperature inside the wand.
- Temperature gauges have been calibrated and checked for accuracy. Verify that equipment has automatic high and low temperature controls.
- Properly sized wand tips for the desired application are available.
- Melter is of sufficient size for the project.

Cold-Applied Sealant Pumps (One- and Two-Component Materials)

- The pump is designed for the intended purpose and is in proper working order.
- Hoses and fittings prevent intrusion of moisture into the system and can sustain pumping pressures. Teflon-lined hoses are preferred.
- The follower plate(s) are in good shape and lubricated.
- Hoses are unobstructed (i.e., not clogged).

Joint/Crack Cleaning Equipment

- Abrasive cleaning unit is adjusted for the correct abrasive feed rate and has oil and moisture traps.
- Abrasive cleaning uses environmentally acceptable abrasive media.
- Abrasive cleaning operators use appropriate air purification systems as required by OSHA.
- Air compressors have sufficient pressure and volume to clean joints adequately and meet agency requirements.
- Air compressors are equipped with oil and moisture filters/traps that are properly functioning. Check the airstream for water or oil prior to use by passing the stream over a board and examining for contaminants.
- Joint plows (if used in a resealing project) are of the correct size and configuration to remove the required amount of old sealant without spalling the joint edges. Plows must be rectangular to avoid damaging the joint faces.
- Concrete saws and saw blades are of sufficient size to adequately cut the required joint width and depth and are in good working order.
- Saw blade is mounted in the correct direction for proper saw operation.
- Water-blasting equipment can supply the water volume and pressure required by the specifications.

Other Equipment

- Backer rod insertion tool is adjusted for the correct installation depth and does not have sharp or jagged edges that could cut or abrade backer material.
- Brushes or sprayers for primer application (if used) are available.
- Tooling/leveling devices (if needed) for finishing the sealant to the required dimensions are available.
- Preformed sealant insertion devices function properly and insert seal strips to the correct recess without excessively stretching the sealant material.

Weather Requirements

The weather conditions at the time of construction can have a large impact on the performance of an installed sealant. Specifically, the following weather-related items should be checked prior to construction:

- Review manufacturer installation instructions for requirements specific to the sealant used.
- Air and/or surface temperature should meet manufacturer and all agency requirements (typically 40°F and rising) for sawing and sealing.
- Sealant should not be installed when temperatures are at or below the dew point. Conditions should be closely monitored if temperatures are approaching the dew point.
- Sealing should not proceed if rain is imminent. Operations should cease if rain commences during installation.
- Application should not begin if there is any sign of moisture on the surface or in the joint.

Traffic Control

To manage the flow of traffic through the work zone, the following traffic-related items should be verified:

- The signs and devices used match the traffic control plan.
- The traffic control setup complies with the Federal *Manual on Uniform Traffic Control Devices (MUTCD)* or local agency traffic control procedures.
- Any unsafe conditions are reported to a supervisor.
- The sealed pavement is not opened to traffic until the sealant has adequately cooled or cured so the sealant material is not picked up on vehicle tires.

Project Inspection Responsibilities

During the construction process, careful project inspection by construction inspectors helps ensure well-performing joint resealing projects. Specifically, the following checklist items (organized by construction activity) summarize the recommended project inspection items.

Joint/Crack Preparation

During the joint/crack preparation steps, the inspector should ensure the following:

- During cutting and cleaning operations, all safety mechanisms and guards on equipment are in place and functioning properly, and operators are using the required PPE.
- Old sealant (if present) is removed from the joint.
- Concrete is allowed to cure for the specified time (minimum of seven days of dry weather) prior to sawing joints.
- The joint is sawn or refaced to produce a rectangular reservoir of the specified depth with cut vertical sides.
- After sawing, joints are flushed with high-pressure water to remove all saw slurry and debris.
- Joint surfaces are cleaned using abrasive cleaning or water blasting.
- Abrasive cleaning is accomplished with the nozzle 1 to 2 in. above the joint using two passes, each directed at one of the joint faces.
- The joint is blown clean with clean, dry air.
- Primer, if used, is applied at the correct coverage rate and allowed to cure as required by the manufacturer.
- Joints are inspected prior to sealing by rubbing a finger along the joint walls to ensure that no contaminants (dust, dried sawing residue, dirt, moisture, or oil) are on the joint walls. The "wipe test" may also be performed (ACPA 2018). If dust or other contaminants are present, reclean joints to a satisfactory condition.
- Inspect joints for proper sealant geometry.

Backer Rod Installation

If backer rods are used, the following items should be checked during the backer rod installation process:

- Ensure that the correct backer rod material is being used and is installed properly. The backer rod diameter should be 25% to 50% greater than the reservoir width.
- The backer rod is installed after final joint cleaning, after inspection for cleanliness, and just prior to sealant installation.
- The backer rod is inserted uniformly (without stretching) into the joint to the required depth to provide the specified sealant dimensions.
- The backer rod fits snugly into the joint with no gaps along the joint sides.
- The backer rod is not torn, abraded, ripped, or otherwise damaged during installation.
- Install the backer rod first—and continuously—into longitudinal joints and last into transverse joints. (At the corners, the transverse backer rods will be placed on top of the backer rods in the longitudinal joints.)

Sealant Installation

Formed-in-Place Hot-Pour Sealants

The project inspector should verify the following:

- The operator is aware of the joint configuration to be installed and has appropriate equipment.
- The manufacturer's/owner's installation instructions are being followed.
- The melter's heat transfer medium is heated to the correct temperature range.
- The sealant is heated to the manufacturer's minimum recommended pouring or application temperature, but not exceeding the material's safe heating temperature.
- To ensure uniformity, the sealant is continuously agitated except when adding additional material.
- The operator wears the required PPE.
- The melter is equipped with a heated hose system and, prior to the beginning of sealant application, the hose is heated to operating temperature.
- The sealant temperature is checked periodically to ensure proper temperature.
- The melting vat should be kept at least one-third full to help maintain temperature uniformity.
- Where joint reservoir dimensions permit (typically when a backer rod is used), the joint is filled from the bottom up with no voids in the sealant to produce a uniform surface flush with the pavement surface.
- If needed, detackifier or other blotter is applied to reduce tackiness prior to the opening of the pavement to traffic.
- Traffic is not allowed on the pavement until the sealant is tack free.

Formed-in-Place Silicone Sealants

During the installation of silicone sealants, the project inspector should check the following:

- The operator is aware of the joint configuration and material to be installed and has appropriate equipment.
- The joint/crack is filled from the bottom up to the specified level to produce a uniform surface with no voids in the sealant.
- Non-sag sealants are tooled as needed to force the sealant material against the sidewalls and to form a smooth surface at the specified recess from the surface.
- Prior to the opening of the pavement to traffic, the sealant is permitted to cure to a tack-free condition.
- Joints are cleaned and resealed according to contract documents.
- Adequate adhesion and elastic properties are verified by field testing random segments of cured sealant. Examples of tests include the knife test, hand-pull test, and sample-stretch test (ACPA 2018). Sealant manufacturers generally recommend one of these or other similar tests to help ensure good sealant performance.

Cleanup Responsibilities

- Any excess sealant application or spills are removed.
- All loose debris from cleaning is removed from the pavement surface.
- Sealant containers or other miscellaneous debris are removed and disposed of properly.
- Melters and other application equipment are properly cleaned for the next use.

Opening the Pavement to Traffic

The sealed pavement is not opened to traffic until the sealant has adequately cooled or cured so the sealant material is not picked up on vehicle tires.

8. Joint Resealing Troubleshooting

As indicated in the previous section, there are a number of factors to consider to help ensure the proper application of joint or crack sealant. Table 10.3 summarizes some of the more common construction and performance problems associated with joint resealing or crack sealing and provides suggested remedies.

Table 10.3. Potential joint resealing and crack sealing construction problems and associated solutions

Problem	Typical solutions
Punctured or stretched backer rod	A punctured or stretched backer rod can result in an improper shape factor or the adherence of sealant to the bottom of the reservoir. Both of these conditions have detrimental effects on the long-term performance of the sealant. If observed, remove the existing backer rod and install a new backer rod using the recommended procedures.
Burrs along the sawed joints	Burrs along the sawed joint can make it difficult to install the sealant. To remedy, drag a blunt pointed tool along the sawed joint to remove the sharp edges (ACPA 1995). (Note: The joint or crack will afterward have to be recleaned prior to sealing.)
Raveling, spalling, or other irregularities of the joint walls prior to sealant application	This is most likely caused by improper care in the sealant removal or joint cleaning steps. (Note: A V-shaped joint plow blade can spall joint sidewalls. Irregularities on joint walls can reduce a sealant's lateral pressure, thereby allowing the sealant to extrude or pop from the joint (ACPA 1995). Therefore, if irregularities are observed, the agency and contractor should agree on an appropriate method for repairing potential problem areas.)
Sealant not adhering to a joint or crack	• Reclean the joint or crack. • Allow the sidewalls to dry before sealing. • Heat sealant to the correct temperature or verify temperature gauges (for hot-applied sealants). • Wait for a higher ambient temperature before sealing and make sure no condensation is accumulating in the joint. • Use the correct recess for joint/crack width (especially important for cold-applied sealants).
Sealant gelling in the melting chamber (also called the "melter")	• If it is suspected that sealant is overheating, check the melter's temperature gauges. • If it is suspected that sealant has been reheated too many times, use fresh sealant. • Use sealant with a longer pot life or conform to the manufacturer's recommended pot life.
Bumps or irregularities in the surface of the tooled sealant application	• Check the tooling utensil or squeegee and ensure it is leaving the correct finish; repair or replace as necessary. • Ensure that tooling is being conducted within the time after application recommended by the manufacturer. • Decrease the viscosity of the sealant (if applicable).
Cold-applied sealants not setting up	• Use fresh sealant. • Use the correct mix ratios and mixing systems.
Sealant picks up or pulls out when opened to traffic	• Close pavement to traffic and delay opening. • Seal during cooler temperatures. • Apply sealant flush with the surface or with a specified recess. • Use a stiffer sealant if the sealant is too soft for the climate. • Use a detackifier or blotter to reduce any initial tack on hot-applied sealants. • Install at the correct temperature and continuously verify the temperature gauges on the melter. • If sealant has been contaminated with solvent or heat transfer oil from a tank leak, repair or replace leaking tank; do not use contaminated sealant. • If joint faces are contaminated with old sealant or other contaminants, repeat the joint preparation process.
Voids or bubbles in cured sealant	• Seal during cooler periods and then allow concrete to further dry or use non-sag-type sealant able to resist void formation. • If it is suspected that backer rods may be melting under a hot-applied sealant, use heat-resistant backer rod material and check for the proper sealant temperature. • If it is suspected that backer rods may be being punctured during installation, install backer rods carefully to avoid damage. • Apply sealant from the bottom up to avoid trapping air. • Tighten all connections and bleed off entrapped air. • If it is suspected that moisture is building up in joints or on backer rods, ensure joints are dry and replace backer rod material if moisture is present. • Cure primer according to the manufacturer's recommendations.
Sinkholes in the sealant	• If sealant is flowing past gaps in the backer material, use larger-diameter backer rod material, reapply (top off) sealant to the correct level, or—for silicone—use a non-sag sealant. • If the backer rod is melting when using hot-applied sealants, use a heat-resistant backer rod.
Sealant cracking or debonding in winter	• If (hot-pour) sealant is too stiff, use sealant that is more extensible at low temperatures. • If poor cleaning during installation is suspected, improve cleaning methods. • If joint is too narrow for the amount of movement experienced, use wider joints. • If joints have been configured incorrectly, with sealant therefore installed too thickly or too thinly, use the correct depth-to-width ratio.

Sources: Adapted from ACPA 2006, FHWA 2019b

9. Surface Sealers

The use of penetrating sealers has gained recent attention as a means of reducing moisture ingress into concrete (Weiss et al. 2016). These products are targeted at reducing premature joint deterioration by preventing or reducing the ingress of moisture and deicing chemicals into concrete. Surface sealers could potentially be used in conjunction with joint resealing to help reduce the deleterious effects resulting from the infiltration of those substances into concrete pavements located in susceptible environments.

Several different surface sealers have been used for this application, broadly categorized as barrier coatings (e.g., acrylics and epoxies), pore blockers (e.g., linseed oil), and water repellents (e.g., silanes and siloxanes). The silane and siloxane products are commonly used in roadway applications, as these materials are small enough to penetrate into concrete and then form a bond with the hydrated cement paste substrate to produce a hydrophobic barrier within the pores of the concrete (Xiao et al. 2020). The result is that water and deicing fluids are prevented from penetrating, yet these products are also "breathable" in that they allow water vapor to escape from within the concrete (Weiss et al. 2016).

Silanes and siloxanes are formulated as a mixture of silane or siloxane solids and either a water-based or solvent-based (e.g., alcohol, mineral spirits) carrying agent (Xiao et al. 20202). The percent solids, sometimes referred to as concentration level, can range from less than 10% up to 100%. A concentration of 100% indicates a pure silane or siloxane product.

Studies on the effectiveness of surface sealers are ongoing. One recent investigation looked at the field and laboratory performance of various surface sealers, including silane, siloxane, and soy methyl ester-polystyrene (SME-PS) products (Xiao et al. 2020). The field performance aspects of the study found no visible signs of the sealers on in-service pavements that had been previously treated with sealers at 2, 6, and 8 years of service. This was attributed as potentially due to the low permeability of the concrete or to the difficulty in adequately applying the sealers to the vertical surfaces of the joint faces (Xiao et al. 2020).

However, the study's laboratory work found that all sealers applied to concrete samples nevertheless resulted in decreased water and deicer absorption as well as extension of the time before critical saturation. Among the products evaluated in the laboratory study, silane and SME-PS were found to be the most effective.

If a surface sealer is to be used on a concrete pavement, the concrete should be cleaned to remove dirt and sawcut laitance. The pavement should then be allowed to dry sufficiently prior to the application of the sealer. The sealer can be applied using a low-pressure sprayer or roller so as to thoroughly saturate the concrete. The rate of surface sealer application should be in accordance with the manufacturer's recommendations.

10. Summary

This chapter presented information on joint and crack sealing in concrete pavements. The need for sealing operations was discussed, including guidelines for identifying appropriate candidate projects. Various available sealant materials were presented, along with their properties, applicable specifications, and design considerations.

Procedures for the sealing of transverse joints, longitudinal joints, and cracks in concrete pavements were described. In almost every project, a successful sealing operation includes the following steps: removing the old material (joint resealing only), refacing the existing joint or crack reservoir, cleaning the reservoir, installing the backer rod (if specified), and installing the new sealant material in the desired configuration.

Because the quality of construction practices is extremely important to the long-term performance of sealant installations, recommended quality control and troubleshooting procedures were presented. These procedures covered the safety of workers and the traveling public.

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Chapter 11
Concrete Overlays

1. Introduction

Concrete overlays placed on existing concrete and composite (i.e., asphalt on concrete) pavements can be effective in extending pavement life by typically 15 to 30 years and are therefore a valuable option for agencies to have in their pavement preservation toolboxes.

Four types of concrete overlays are described in Figure 11.1, with links to the chapters in the fourth edition of the CP Tech Center's *Guide to Concrete Overlays* (Fick et al. 2021), where further details on the materials, design, and construction of each of these four types of concrete overlays can be found.

As Figure 11.1 shows, concrete overlays can be either bonded or unbonded. Bonded overlays help eliminate surface distresses and add some structural value to the pavement system by forming a monolithic pavement structure with the existing pavement. Unbonded overlays add structural capacity to the pavement system but do not require bonding to the existing pavement.

Historically, highway agencies have used thin bonded and unbonded concrete overlays (≤4 in.) as preservation treatments, while overlays thicker than 4 in. have been considered to add structure and have therefore been classified as rehabilitation strategies. This has relegated pavements that have higher traffic loading and therefore thicker structural requirements (typically unbonded overlays) outside of certain preservation funding mechanisms. This does not necessarily represent a proactive approach, however, as it prevents roads that experience heavier traffic from being restored and extended at the optimum time with the most cost-effective engineering strategy.

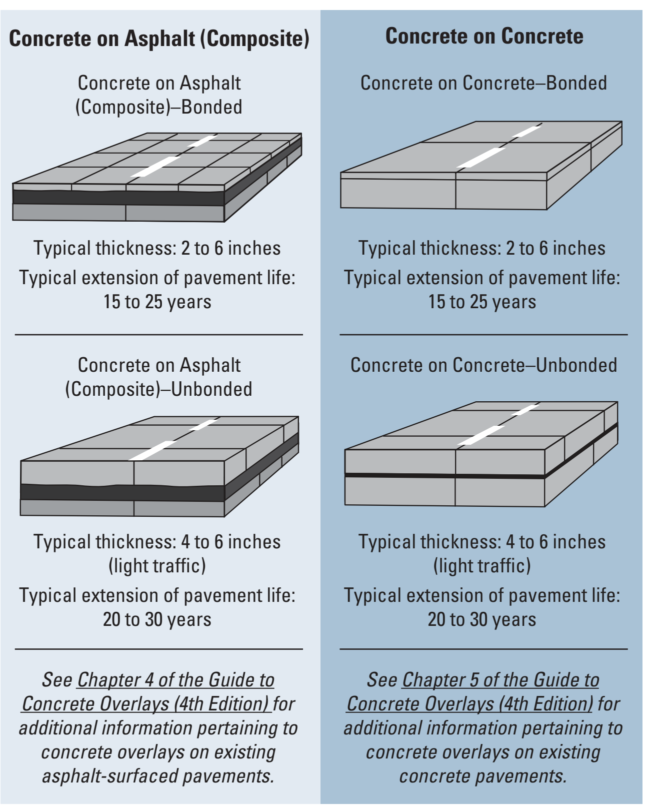

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Figure 11.1. Concrete overlays by type of existing pavement

This chapter provides a brief overview of concrete pavement overlays, focusing on their potential applications. More detailed information is available in the CP Tech Center's *Guide to Concrete Overlays* (Fick et al. 2021).

2. Limitations and Effectiveness

The following considerations must be taken into account when determining whether an existing pavement (either concrete or asphalt/composite) is a viable candidate for a preservation concrete overlay:

- Major structural repairs should not be required.
  - For bonded overlays, the goal is to achieve a monolithic structure by bonding the new overlay to the existing pavement. Bonded concrete overlays are therefore generally not good solutions in any of the following situations:
    - The underlying concrete pavement is not in good structural condition, as evidenced by significant joint deterioration, widespread slab cracking, poor subgrade support, or poor drainage conditions.
    - The underlying asphalt pavement has significant structural deterioration, as evidenced by fatigue cracking and rutting; has known asphalt stripping issues; exhibits inadequate base or subgrade support; or has poor drainage
  - For unbonded overlays, the existing pavement is considered merely as base support for the concrete overlay and therefore does not have to be in good structural condition but must be uniform and stable.
- Potential issues resulting from the construction of the overlay should be identified. For example, overlays introduce additional thickness and can therefore result in clearance issues at structures and narrowing of shoulders or steepening of foreslopes. Similarly, in urban areas, overlays can affect the elevation of curbs and gutters, sidewalks, and driveways, which may drive the need for reconstruction. Utilities and roadway fixtures may also require elevation adjustment.
- The expected service life of the proposed overlay should match or exceed the service life desired by the highway agency.

Table 11.1 provides a summary of considerations for bonded and unbonded overlays on both existing concrete and composite pavements.

Figure 11.2 presents typical overlay use based on the general condition of the existing pavement. This provides quick guidance on the applicability of the different types of overlays to various existing pavement conditions.

3. Project Selection

Concrete overlays require uniform support conditions to deliver satisfactory performance. Nearly all documented cases of premature concrete overlay failure can be traced to some violation of this single requirement. To avoid "picking the wrong project" for a concrete overlay, accurate evaluation of the existing pavement is paramount in determining if uniform support and movement control exist or if they can be cost-effectively achieved. Figure 11.3 is a flow chart of existing pavement conditions, resulting preliminary repairs needed, and type of concrete overlay therefore to consider.

4. Pavement Evaluation

Before an overlay is constructed on either an existing concrete or composite pavement, some spot repairs may be required. As indicated in Figure 11.3, the extent of preliminary repairs needed is an important factor in determining whether a bonded or unbonded overlay will be the most cost-effective solution.

A pavement evaluation summarizes key distresses and performance problems that currently exist and their underlying causes (see Chapter 3). Additional details on conducting a pavement evaluation specifically for concrete overlays is found in the fourth edition of the CP Tech Center's *Guide to Concrete Overlays* (Fick et al. 2021).

Table 11.1. Bonded versus unbonded concrete overlays

Consideration	Bonded concrete overlay	Unbonded concrete overlay
Purpose	A bonded concrete overlay is primarily a preventative maintenance or preservation treatment to improve surface characteristics and/or load-carrying capacity.	An unbonded concrete overlay is primarily a preservation treatment for concrete pavements in fair, poor, or worse condition that adds structural value to the existing pavement.
Condition of existing pavement	The pavement is in good structural condition or can be repaired to achieve that condition (which at times can be difficult to cost-effectively accomplish).	The underlying pavement can be in poor to deteriorated condition but must be uniform and stable, including the existing base and/or subgrade.
Resulting improvements to the pavement	• Load-carrying capacity added • Pavement life extended • Surface defects eliminated • Surface characteristics like smoothness, friction, and/or noise improved • Long-term wearing surface added	• Load-carrying capacity added • Pavement life extended • Surface defects eliminated • Surface characteristics like smoothness, friction, and/or noise improved • Long-term wearing surface added

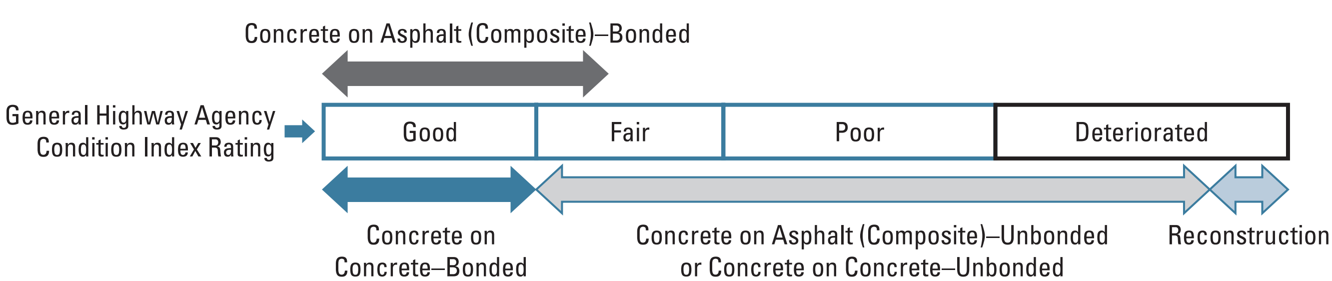

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Figure 11.2. Typical applications of concrete overlays by general condition of existing pavement

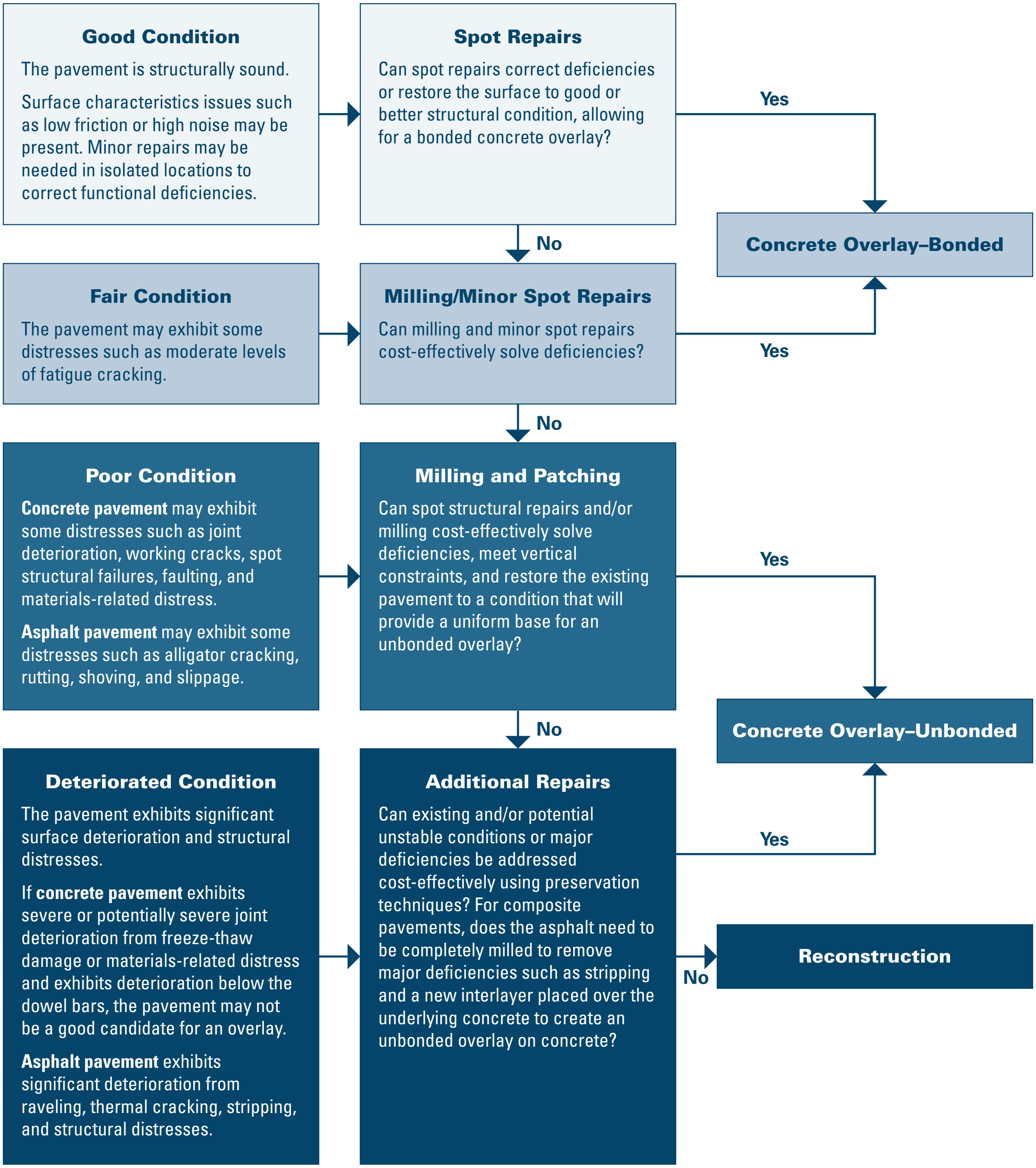

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Figure 11.3. Determining appropriate concrete overlay solution based on existing pavement condition and resulting preliminary repairs needed

5. Brief Technical Considerations

Design

Successful concrete overlay design addresses all overlay system design components (e.g., thickness, panel dimensions, bond condition, joint design, edge support, material properties, pre-overlay repairs) in a manner that achieves the desired performance (i.e., service life, service quality, and load-carrying capacity) as economically as possible.

Concrete overlays are generally designed and constructed using conventional materials, including cements, SCMs, aggregate, water, and chemical admixtures. In some cases, accelerated mixtures may be used to enable a faster rate of strength gain to allow earlier opening of an overlay to construction traffic and/or public traffic. More details are provided in the fourth edition of the *Guide to Concrete Overlays* (Fick et al. 2021).

Construction

Construction steps for concrete overlays include pre-overlay repairs, milling, surface cleaning, concrete placement, curing, and joint sawing.

Normally, the methods and equipment for concrete overlay construction are the same as those used in new concrete pavement construction. There are, however, some special considerations for overlay placement during cooler periods, particularly in cold-weather states. Under such conditions, the existing base and pavement will expand and contract with the daily change in ambient temperature. Cracking may occur if the concrete mixture used for the overlay has not gained enough strength to withstand the stresses caused by differential movement between the underlying pavement structure and the new concrete overlay.

Additional details on quality concrete overlay construction are provided in the fourth edition of the *Guide to Concrete Overlays* (Fick et al. 2021).

6. Summary

Concrete overlays can provide solutions for a range of issues and deficiencies in existing concrete and composite pavements, and they can be designed and constructed as either bonded or unbonded systems.

Over the last decade, changes and improvements in concrete overlay technology have focused on improved design procedures, detailed construction guidelines, and relevant specifications. Over the same time period, concrete overlays have been shown to offer significant sustainability benefits (Ram and Smith 2019):

- Concrete overlays reduce negative environmental impacts by preserving existing pavement structures, thereby minimizing associated waste products that would otherwise go into landfills.
- Concrete overlays reduce user delays during construction when compared to reconstruction activities.
- Concrete overlays provide societal benefits by improving pavement ride quality, noise, albedo, and friction.

7. References

Fick, G., J. Gross, M. B. Snyder, D. Harrington, J. Roesler, and T. Cackler. 2021. *Guide to Concrete Overlays*. 4th Edition. National Concrete Pavement Technology Center, Iowa State University, Ames, IA. https://intrans.iastate.edu/app/uploads/2021/11/guide_to_concrete_overlays_4th_Ed_web.pdf.

Ram, P. and K. Smith. 2019. *Unbonded Concrete Overlays*. FHWA-HIF-17-013. Federal Highway Administration, Washington, DC.

Chapter 12

Treatment Strategy Selection

1. Introduction

As described in Chapter 2, concrete pavement preservation is the strategy of extending concrete pavement service life by arresting, greatly diminishing, or avoiding pavement deterioration processes. This chapter provides information about the types of factors that should be considered in order to select an appropriate preservation treatment strategy for a given pavement. Included among these factors are the existing pavement conditions and the traffic and climatic characteristics of the project, which influence treatment and pavement performance and the projected cost-effectiveness of competing preservation treatment strategies. In addition, some transportation agencies are beginning to include sustainability factors in their agency-level decision-making process by assessing environmental and social impacts along with economic considerations.

2. Treatment Strategy Selection Process

Overview of the Treatment Strategy Selection Process

Chapter 2 discussed the importance of reviewing available pavement management data in determining (1) whether or not a project is a suitable candidate for preservation, (2) which treatments are feasible for a project, and (3) which treatments are most ideal in terms of cost-effectiveness and other considerations. Although such data can be valuable in screening projects for preservation—and even in pointing to possible candidate treatments—more information about a project is usually needed to confirm that preservation is, in fact, appropriate. This is particularly true if the most recent data in the pavement management database are more than 1 or 2 years old.

To adequately capture the current conditions of an existing pavement, an evaluation of the pavement is required, as described in Chapter 3. The primary goal of this activity is to identify the deficiencies in the pavement (i.e., the extent of the needs of the pavement) and then ultimately to determine how to best address these deficiencies. For example, if the existing pavement is exhibiting only functional deficiencies or localized structural problems, the observed deficiencies can most likely be addressed with one or more concrete pavement preservation treatments. However, if more global structural or material problems exist, then the pavement section is more likely suited for a structural overlay treatment or perhaps even complete reconstruction in the most severe case. (Because discussion of reconstruction is outside the scope of this guide, this chapter focuses only on the selection of the most appropriate concrete pavement preservation treatments as well as on concrete overlays.)

At the project level, the process of determining the most appropriate pavement preservation treatment strategy for concrete pavements is a fairly straightforward one. The following step-by-step process can be used to determine the most appropriate treatment (or combination of treatments) for a concrete pavement (Hall et al. 2001, ARA, Inc. 2004, Peshkin et al. 2011):

1. Conduct a thorough pavement evaluation.
2. Determine causes of distresses and deficiencies.
3. Identify treatments that can address deficiencies.
4. Identify constraints and key treatment strategy selection factors.
5. Develop feasible treatment strategies.
6. Assess the cost-effectiveness of alternative treatment strategies.
7. Select the preferred treatment strategy.

Each of these steps is discussed separately below.

Step 1: Conduct a Thorough Pavement Evaluation

As discussed in Chapter 3, conducting a pavement evaluation is the first step in assessing the current deficiencies of a pavement.

This process focuses on determining both the structural and functional adequacy of the existing pavement. The structural condition refers to the ability of the pavement to carry current and future traffic loading and is determined from the results of the condition and drainage surveys, deflection testing, and any materials sampling and testing. The functional condition refers to the ability of the pavement to provide a smooth and safe riding surface for users and is primarily determined by reviewing the results of any roughness and friction testing (and, if appropriate, noise testing).

Table 12.1 presents a summary of the different pavement condition attributes included in a typical pavement evaluation and the methods by which they can be assessed.

Table 12.1. Overall pavement condition attributes included in a typical pavement evaluation and corresponding data sources

Pavement condition attribute	Distress survey	Drainage survey	Deflection testing	Roughness testing	Friction testing	Field sampling and testing
Structural adequacy	✓	✓	✓			✓
Functional adequacy	✓			✓	✓	
Drainage adequacy	✓	✓	✓			✓
Materials durability	✓	✓	✓			✓
Maintenance applications	✓	✓				
Shoulder adequacy	✓		✓			✓
Variability along project	✓	✓	✓	✓		✓

Source: Adapted from ARA, Inc. 2004

Step 2: Determine Causes of Distresses and Deficiencies

One of the most important steps in the treatment selection process is to review at one time all of the data from the pavement evaluation to determine the causes of any observed distresses and identified deficiencies. A summary of typical concrete pavement distresses and their causes is provided in Table 12.2. Through knowing the underlying causes of the distresses that are observed, appropriate preservation treatments can be identified.

Step 3: Identify Treatments That Can Address Deficiencies

The main objective of the third step is to identify the pavement preservation treatments (or series of preservation treatments) that address one or more of the identified pavement deficiencies. Within the scope of this document, this includes the following concrete pavement preservation treatments:

- Slab stabilization and slab jacking (Chapter 4)
- Partial-depth repair (Chapter 5)
- Full-depth repair (Chapter 6)
- Retrofitted edgedrains (Chapter 7)
- Dowel bar retrofit, cross-stitching, and slot-stitching (Chapter 8)
- Diamond grinding and diamond grooving (Chapter 9)
- Joint resealing and crack sealing (Chapter 10)
- Concrete overlays (Chapter 11)

Whereas more specific details on the appropriate uses of each of these treatments are included in Chapters 4 through 11, a summary of the general application of each preservation treatment is presented in Table 12.3.

Table 12.2. Concrete pavement distress types and causes

Distress	Causes	Notes
Linear cracking (transverse, longitudinal, or diagonal)	Traffic loading, often in combination with slab curling and/or warping; drying shrinkage; improper transverse or longitudinal joint design or construction; or foundation settlement and movement	Low-severity transverse cracks in JRCP and CRCP are not considered structural distress; medium- and high-severity deteriorated cracks are. All severities of linear cracking are considered structural distress in JPCP.
Corner breaks	Traffic loading, often in combination with slab curling and/or warping and/or erosion of support at slab corners	The presence of corner breaks suggests structural deterioration. Medium- and high-severity levels can significantly impact ride quality.
D-cracking	Freeze-thaw damage in coarse aggregates	This initiates as hairline cracks in the slab corners and progresses along joints, cracks, or free edges where moisture is present.
Alkali-aggregate distress	Compressive stress building up in slab due to swelling of gel produced from the reaction of certain susceptible aggregates with alkalis in the cement	Alkali-aggregate reaction includes alkali-silica reactivity and alkali-carbonate reactivity.
Map cracking and crazing	Alkali-aggregate reaction, overfinishing, or finishing with bleed water on surface	Hairline cracks in upper surface of slab are cosmetic but can deteriorate into scaling.
Scaling	Overfinishing, finishing with bleed water on the surface, inadequate air entrainment, or reinforcing steel too close to the surface	This is typically limited to the upper few inches of the slab surface.
Joint seal damage	Inappropriate sealant type, improper sealant reservoir dimensions for the sealant type, improper joint sealant installation, and/or aging of the sealant	Joint seal damage includes loss of adhesion to joint walls, extrusion of sealant from joint, infiltration of incompressible materials, oxidation of sealant, and cohesive failure (splitting) of the sealant.
Joint spalling (also called joint deterioration)	Compressive stress buildup in the slab, alkali-aggregate reaction, D-cracking, freeze-thaw damage of hardened paste, misaligned or corroded dowels, poorly consolidated concrete, or damage caused by joint sawing, joint cleaning, cold milling, or grinding	Joint spalling includes cracking, breaking, chipping, or fraying of slab edges within 1 ft of the transverse or longitudinal joint.
Blowups	Compressive stress buildup in the slab due to infiltration of incompressibles, elevated temperatures, moisture profiles in the slab, or alkali-aggregate reaction	A blowup may occur as a shattering of the concrete for several feet on both sides of the joint or as an upward buckling of the slabs.
Pumping	Excess moisture in the pavement structure, erodible base or subgrade materials, and traffic loading	Pumping can lead to loss of support beneath the slab and the development of faulting. Dowel bars and nonerodible bases can help control pumping.
Faulting	Pumping of water and fines from under slab corners, loss of support under the leave corner, and buildup of fines under the approach corner	Faulting becomes a significant factor in ride quality when it is greater than about 0.08–0.12 in.
Roughness caused by curling and/or warping	Moisture gradients through the slab, daily and seasonal cycling of temperature gradients through the slab thickness, and/or permanent deformation caused by a temperature gradient in the slab during initial hardening	Curling and warping are often influential factors affecting the structural (e.g., cracking) and functional (e.g., smoothness) performance of concrete pavements.
Bumps, heaves, and settlements	Foundation movement (frost heave, swelling soil) or localized consolidation, such as may occur at culverts and bridge approaches	Bumps, heaves, and settlements detract from riding comfort and at high severity may pose a safety hazard.
Polishing	Abrasion by tires	Polished wheel paths may pose a wet-weather safety hazard.
Popouts	Freezing in coarse aggregates near the concrete surface	This is a cosmetic problem rarely warranting repair.
Wheel path wear	Abrasion caused by studded tires or tires affixed with chains	Wheel path wear can contribute to wet-weather safety issues such as hydroplaning and increased splash and spray.

Sources: Compiled from Hall et al. 2001, Miller and Bellinger 2014, Harrington et al. 2018

Table 12.3. Applicability of concrete pavement preservation treatments based on distress

Distress	Slab stabilization	Slab jacking	Partial-depth repair	Full-depth repair	Dowel bar retrofit	Cross-stitching/ slot-stitching	Diamond grinding	Diamond grooving	Retrofitted edgedrains	Joint resealing	Crack sealing	Concrete overlay
Corner breaks				✓							✓[a]	✓
Linear cracking				✓		✓[a, b]					✓[a]	✓
Punchouts				✓								✓
D-cracking				✓[c]								
Alkali-aggregate reaction				✓[c]								
Map cracking, crazing, and scaling			✓									
Joint seal damage										✓		
Joint spalling			✓	✓								
Blowup				✓								
Pumping	✓				✓	✓			✓			✓
Faulting		✓			✓		✓		✓			
Bumps, heaves, settlement		✓		✓			✓					
Polishing/ low friction							✓	✓				✓

Note: Many of these treatments are often done in combination to fully address all pavement deficiencies.

[a] *Cracks with limited vertical movements.*

[b] *Longitudinal cracks only.*

[c] *Pavements with slow-acting D-cracking or ASR. In the case of overlays, unbonded concrete overlays are considered viable candidates, but bonded overlays are not. The lower the severity and rate of the MRD as determined through laboratory analysis, the higher the chance of longer service life.*

Source: Adapted from Hall et al. 2001

In general, a step-by-step check of the following can help identify preservation treatments that may be appropriate for a given project:

1. **Assess Slab Support Conditions**—When assessing the support conditions underlying concrete slabs, deflection testing can be performed to identify voids at slab corners and to assess the load transfer capabilities of transverse joints (and cracks). One good indication that there is a slab support problem is the presence of pumping (i.e., fine materials deposited on the pavement or shoulder surface at or near the transverse joints). Concrete slabs that currently do not have structural problems (i.e., corner breaks or linear cracking) but are found to have voids or poor load transfer are good candidates for slab stabilization or DBR.

2. **Correct Localized Distress That Is Limited to the Upper Half of the Slab Thickness**—In concrete pavements, it is not uncommon to have localized areas of distress that are limited to the upper half of the slab thickness. Common distresses in this category include joint spalling, map cracking (i.e., crazing), and scaling. If any of these distresses are present in an amount or severity that requires attention, a PDR is typically the best treatment to correct the distress. A thin concrete overlay, however, may also be a suitable solution for a superficial problem that is widespread over an entire project.

3. **Correct Localized Distress That is Not Limited to the Upper Half of the Slab Thickness**—When a pavement evaluation identifies distress that is not limited to the upper half of the slab thickness (e.g., corner breaks or transverse cracking), an FDR (or DBR for transverse cracking) is typically required to correct the observed distress. If the cracks are not significantly deteriorated and exhibit limited vertical movement under traffic, however, then crack sealing may be a suitable solution.
4. **Correct Functional Distress**—Many otherwise sound concrete pavements may exhibit functional deficiencies, such as poor friction or excessive roughness. Diamond grinding is typically used to correct roughness problems, but it also has a positive impact on a pavement's friction and noise characteristics. If the only functional problem is found to be a localized area of poor friction (such as at curves or intersections), diamond grooving is an effective treatment option.
5. **Assess Joint Sealant Condition**—One final step in the treatment strategy selection process is to assess the condition of the sealant in the joints of the concrete pavement. In general, if the original pavement was sealed at the time of initial construction, then every effort should be made to maintain an effectively sealed joint over the life of the pavement. Therefore, if there are any signs of joint sealant damage, or if any other preservation treatments have caused the effectiveness of the joint sealant to be compromised to a significant extent (e.g., 25% or more of the seal length has adhesion or cohesion failures or contains incompressible material), joint resealing should be considered. When conducted with other treatments, joint resealing should always be the final activity performed on a given pavement preservation project before it is opened to traffic.

Step 4: Identify Constraints and Key Treatment Strategy Selection Factors

After compiling a list of possible effective treatments under Step 3 and before proceeding further in the treatment strategy selection process, it is important to check the possible effective treatments against a list of any project-specific constraints or other key treatment strategy selection factors that may come into play. Some of the potential treatment strategy selection factors that an agency will need to consider when determining whether or not a possible treatment is feasible for a specific project are the following (AASHTO 1993, Hall et al. 2001):

- Traffic level
- Climate
- Available funding
- Future maintenance requirements
- Geometric restrictions
- User impacts during construction (e.g., lane closure time, traffic disruption/congestion, and safety)
- Environmental impact (e.g., energy demands in materials production and greenhouse gas emissions during construction)
- Conservation of natural resources (e.g., recycling and reuse)
- User impacts during service (e.g., smoothness and friction levels and noise emissions)
- Worker safety during construction
- Traffic management (e.g., traffic control) options
- Availability of needed equipment and materials
- Competition among providers of materials
- Agency policies

The role that each of these treatment strategy selection factors plays will depend on the specific characteristics of the particular project, such as its setting (rural versus urban), roadway classification (Interstate, highway, arterial, collector, etc.), and geographic location, among others. What is important, however, is that all outside constraining factors be identified at this point in the treatment strategy selection process to avoid unnecessary work in the upcoming steps.

Step 5: Develop Feasible Treatment Strategies

A treatment strategy is a plan that defines which treatments to apply and when to apply them over a selected time period. For example, a treatment strategy using only one treatment could be to conduct periodic diamond grinding every 8 to 12 years to achieve a 25-year service life. Another treatment strategy might be the one-time application of DBR followed by periodic diamond grinding to accomplish the 25-year performance period. Still another treatment strategy might be the placement of an unbonded concrete overlay with a 25-year service life.

It is important to note that treatment strategies will often integrate more than one concrete pavement preservation treatment in a single project, such as for the case just described that combined DBR with periodic diamond grinding. Similarly, some full-depth patching is often done in conjunction with concrete overlay placement to help restore support conditions. The various preservation treatments all target different deficiencies and conditions, so applying them in conjunction with one another often improves the effectiveness of a given overall treatment strategy. Thus, the purpose of this step is to identify treatment strategies (each generally consisting of multiple preservation treatments) that best address the current needs of a pavement, while also considering the pavement's potential preservation needs at various points in the future.

Step 6: Assess the Cost-Effectiveness of Alternative Treatment Strategies

Because the various individual concrete pavement preservation treatments for the most part address different pavement deficiencies, cost-effectiveness analysis techniques are not typically needed to help select appropriate treatment strategies. Where cost-effectiveness considerations may come into play, however, is when there are competing treatment strategies identified, each of which could potentially address the current pavement conditions; often, this may come down to comparisons between a preservation treatment strategy (e.g., PDR, FDR, diamond grinding, and joint resealing) and an overlay treatment strategy (e.g., a thin concrete overlay), or possibly even reconstruction. Competing treatment strategies can be objectively compared by considering the overall cost-effectiveness of each treatment strategy as one major determinant in the treatment strategy selection process.

A cost-effectiveness analysis provides an objective method of comparing the costs associated with different treatment strategies applied at different times over the life of a pavement. The analysis should include all costs associated with these treatment strategies, including the materials, construction/installation, traffic control, and so on. The results of cost-effectiveness analysis are of particular interest to agencies that are trying to document the benefits of using less expensive preservation treatment strategies that delay more expensive rehabilitation activities.

This section describes and illustrates an approach to analyzing cost-effectiveness that is commonly referred to as the benefit-cost ratio (BCR) analysis method.

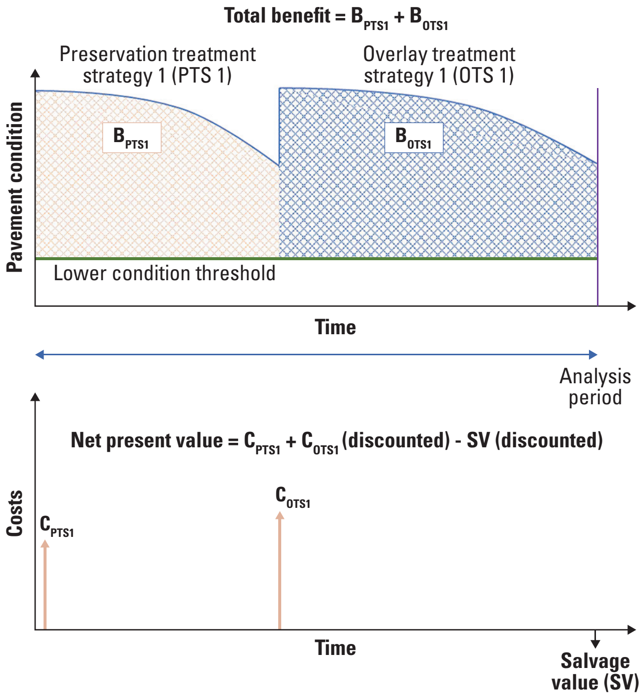
The BCR analysis method considers the performance benefits of one or more treatment strategies and the associated costs of applying these treatment strategies.

The BCR analysis method combines the results of individual evaluations of treatment strategy benefits (B) and treatment strategy costs (C) to generate a BCR (Peshkin et al. 2011). The BCRs of alternative treatment strategies (including a "do nothing" strategy, if desired) are then compared, and the treatment strategy with the highest ratio is deemed the most cost-effective.

As part of BCR analysis, the costs and performance characteristics of the alternative treatment strategies must be estimated. As mentioned in Chapter 2, agencies' pavement management databases provide the best source of data for modeling pavement performance and developing reasonable service life estimates for a given original pavement, preservation-treated pavement, and rehabilitated pavement.

Historical condition data in the form of overall condition indicators and individual distress parameters can be obtained from an agency's pavement management database and used in conjunction with roughness and friction measurements to create performance curves (as a function of time and/or traffic) for unique families of original concrete pavement and for different types of preservation treatments applied to these families of pavements. These curves can then be used as models for projecting pavement performance beyond the range of available time/traffic data to a condition level representative of the needed structural improvement (i.e., major rehabilitation or reconstruction) and/or functional improvement (i.e., friction or smoothness).

In the BCR method, the benefits associated with a treatment strategy are evaluated from the standpoint of benefits accrued to the roadway user over a selected analysis period (Peshkin et al. 2011). They are quantified by computing the area under the pavement performance curve, which is defined by the expected timings of future preservation and rehabilitation treatments and the corresponding jumps and subsequent deterioration in condition or serviceability/smoothness. The expected timings can be obtained from historical condition data, as discussed above, from historical preservation and rehabilitation treatment records, or even from expert opinion. Agencies also select the lower condition threshold at which major rehabilitation or reconstruction is required.



Adapted from Smith et al. 2014, CP Tech Center (after Peshkin et al. 2011, SHRP 2)

Figure 12.1. Benefits and costs associated with a pavement preservation treatment strategy over time

An illustration of the BCR approach is shown in Figure 12.1.

The BCR analysis illustrated in Figure 12.1 is for a single treatment strategy that combines one or more preservation treatments with an overlay treatment. However, BCR analysis can be performed and similar illustrations developed for comparisons across various treatment strategies.

The top portion of Figure 12.1 shows the assessment of benefits using the area-under-the-performance-curve approach. A treatment strategy with a greater area under the curve yields more benefit for the roadway user. The most tangible performance measure for quantifying benefits is pavement roughness (e.g., IRI). However, an overall pavement condition indicator such as the PCI or PSR may also be a suitable measure, as pavements in better overall condition tend to provide a smoother, safer ride.

In BCR analysis, the costs associated with a particular treatment strategy are evaluated using life-cycle cost analysis (LCCA) techniques. The LCCA must use the same analysis period and the same timings for its preservation and rehabilitation treatments as the analysis period and timings used in computing benefits. An appropriate discount rate is then identified and used to convert to present-day costs the costs of the future projected preservation and rehabilitation treatments as well as any salvage value at the end of the analysis period (a negative cost). These costs are then summed together with the cost of the existing pavement (again, either the original structure or the last significant rehabilitation) to generate the total life-cycle cost (expressed as net present value [NPV]) associated with the proposed treatment strategy. The bottom portion of Figure 12.1 illustrates the stream of costs included in the LCCA. These costs represent the costs paid by the agency to construct the existing pavement and apply the subsequent preservation and rehabilitation treatments.

In the final step of the BCR method, the BCR for each treatment strategy is computed by dividing the "benefit" obtained from the area-under-the-performance-curve analysis by the "cost" obtained from the LCCA. (Again, these performance curves can be developed from available historical condition data, historical preservation and rehabilitation treatment records, or expert opinion.) Ultimately, the strategy with the highest BCR is deemed the most cost-effective.

Most state transportation departments have a standardized procedure for conducting LCCA, but if needed, detailed information on all aspects of the LCCA process is available in a number of publications (Walls and Smith 1998, Hall et al. 2001, ACPA 2002, Hallin et al. 2011, Peshkin et al. 2011). In addition, the FHWA offers a spreadsheet program (*RealCost*) that completely automates the LCCA methodology as it applies to pavements; this program is currently being updated, but the latest version can be accessed at https://www.fhwa.dot.gov/infrastructure/asstmgmt/lccasoft.cfm.

There are several other methodologies available that can be used in assessing cost-effectiveness and are briefly described here for informational purposes. These procedures are primarily used in evaluating the cost-effectiveness of pavement preservation strategies at the network level, but they can be adapted for project-level analysis as well:

- **Dollars per lane mile per year (DLMY)** analysis sets up an optimization problem where the objective is to maintain a pavement network in the best possible condition when subjected to funding constraints. The goal is to maximize the lane mile years of acceptable service "purchased" for a given budget level; in other words, DLMY analysis allocates funding to the projects that provide the greatest possible return on investment (Galehouse and Sorenson 2007).

When the DLMY approach is used at the project level, it should only be used to compare treatments or treatment strategies that are expected to provide similar benefits in terms of extension of pavement service life. The measure of cost-effectiveness for this approach is dollars per lane mile for each year of service added by the analyzed treatment. Additional information on using this approach for project- and network-level analyses is available in Van Dam et al. (2019).

- **Remaining service interval (RSI)** is a pavement life-cycle management framework that can be used to develop project- and network-level maintenance and rehabilitation strategies (Rada et al. 2016, Amec Foster Wheeler, Environment & Infrastructure, Inc. 2016, Ram et al. 2020). The RSI framework is focused on identifying a structured sequence of different types of strategically timed preservation and rehabilitation treatments that are needed to provide the desired level of performance to road users over the chosen analysis period at the lowest practical life-cycle cost. The RSI framework allows the application of any feasible preservation or rehabilitation treatment(s) over the chosen analysis period, as long as the established performance constraints and minimum acceptable level-of-service (LOS) criteria are met. Project-level RSI analysis can be used to develop multiple treatment strategy options that satisfy established LOS criteria and other performance constraints. These treatment strategies can then be ranked in increasing order of life-cycle cost such that the treatment strategy that provides the desired performance at the lowest life-cycle cost is chosen as the optimal treatment strategy for the particular pavement segment being analyzed.
- **Life-cycle planning (LCP)** is a network-level analysis that is performed for an agency's entire pavement inventory or any subset of pavement assets within an agency's inventory. LCP refers to the process of developing and comparing strategies "to estimate the cost of managing an asset class or asset subgroup over its whole life, with consideration for minimizing cost while preserving or improving the condition" (23 C.F.R § 515.5 [2016]). LCP assists with the rational evaluation of whether one strategy for maintaining pavement assets is better than another, based on long-term cost and performance considerations. State DOTs are required to use LCP in developing risk-based transportation asset management plans (Zimmerman et al. 2019). At the project level, an LCP analysis is essentially the same as the BCR method discussed earlier.

Step 7: Select the Preferred Treatment Strategy

Decision Factors

A detailed cost-effectiveness analysis can be one part of the treatment strategy decision-making process, but cost-effectiveness analysis by itself does not necessarily identify the optimal alternative. For example, the highest BCR option may not be practical when other considerations, such as available budget, network priorities, environmental factors, and agency and customer preferences, are considered. In some cases, the constraints identified in Step 4 may override the results of the cost-effectiveness analysis. Ultimately, the goal is to select the preferred alternative that best addresses the performance issues of the pavement while meeting all functional and monetary constraints that exist.

A list of some of the critical factors that are appropriate for inclusion in the final treatment strategy selection process is provided below. The factors are grouped according to different attributes. The final determination should properly be one of professional engineering practice and judgment based on the consideration and evaluation of all treatment strategy selection factors applicable to a given roadway section:

- **Economic Attributes**
  - Initial cost
  - Cost-effectiveness (LCCA or BCR)
  - Agency cost
  - User cost
- **Construction/Materials Attributes**
  - Availability of qualified (and properly equipped) contractors
  - Availability of quality materials
  - Conservation of materials/energy
  - Weather limitations
- **Customer Satisfaction Attributes**
  - Traffic disruption
  - Safety issues (friction, splash/spray, reflectivity/visibility)
  - Ride quality and noise issues
- **Agency Policy/Preference Attributes**
  - Continuity of adjacent pavements
  - Continuity of adjacent lanes
  - Local preference

One way of evaluating these different treatment strategy selection factors and identifying the preferred treatment strategy is through a strategy decision matrix (Peshkin et al. 2011). In a strategy decision matrix, various treatment strategy selection factors are identified for consideration and each factor is assigned a weighting that reflects the agency's perception of the importance of that factor. These weightings are then multiplied by rating scores given to each treatment strategy, based on how well the strategy satisfies each of the treatment strategy selection factors. The weighted scores are then summed and compared with the weighted scores of the other treatment strategies. The treatment strategy with the highest score is then recognized as the preferred treatment strategy. Illustrative examples of the strategy decision matrix approach can be found in several references (Hallin et al. 2011, Peshkin et al. 2011).

Sustainability Considerations

As described in Chapter 2, sustainability considerations are being included by a growing number of state and local highway agencies in various aspects of their transportation decision-making processes. Sustainability is made up of three components (economic, environmental, and social factors) whose influence is context sensitive and driven by the characteristics, location, materials, and constraints of a given project, as well as the overarching goals of the agency.

It is important to note that pavement preservation is inherently a sustainable activity, in that it employs low-cost treatments to prolong or extend the life of a pavement. By using relatively low-cost and low-environmental-impact techniques to maintain roads in good condition, pavement preservation helps in delaying major rehabilitation activities and thereby conserves energy and virgin materials while reducing greenhouse gas emissions. Furthermore, well-maintained pavements provide a smoother, safer, and quieter traveling surface over a significant portion of their lives, resulting in higher vehicle fuel efficiencies, reduced crash rates, and lower noise impacts on surrounding communities, which also positively contribute to overall sustainability.

There is currently limited information available regarding the effects of pavement preservation activities on the overall sustainability of pavement systems, but a qualitative evaluation is presented in an FHWA document (Van Dam et al. 2015). In general, longer-lasting treatments (by virtue of their design or good construction quality), thinner treatments, and those that have the greatest impact on preserving ride quality and surface characteristics are noted to have a reduced environmental impact over the pavement life cycle. These relative comparisons, however, are very broad and may vary considerably depending on the prevailing traffic levels, climatic effects, pavement conditions, and material and construction costs associated with each treatment.

Techniques used to assess the environmental and social aspects of pavement sustainability are summarized below:

- **Life-cycle assessment (LCA)** is a technique used to analyze the environmental impacts associated with a product, system, or process. LCA can be used to quantify the energy and material inputs and outputs over the life cycle of a pavement system, from raw material product to end of life. An LCA analysis can help determine and compare the environmental impacts associated with various pavement preservation and rehabilitation strategies, such as energy use, greenhouse gas emissions, ozone depletion, and smog formation, among others (Harvey et al. 2014). The FHWA has published a framework document that provides guidance on conducting LCA studies for pavement systems (Harvey et al. 2016) and is currently developing a pavement LCA tool that can be used to model and evaluate the environmental impacts associated with various pavement preservation and rehabilitation treatments.
- **Sustainability rating systems (SRS)** can be used to assess the social aspects associated with various pavement preservation and rehabilitation strategies. SRS are checklists of various sustainable practices associated with a common metric that can facilitate the communication of sustainability goals, efforts, and outcomes. Although pavements are not the primary focus of SRS, sustainable pavement practices contribute to the overall score determined using SRS. Some examples of social aspects that are assessed using SRS include noise mitigation, light pollution reduction, pedestrian and bicycle access, access to essential services, and water use (Muench 2020).

Construction Sequencing

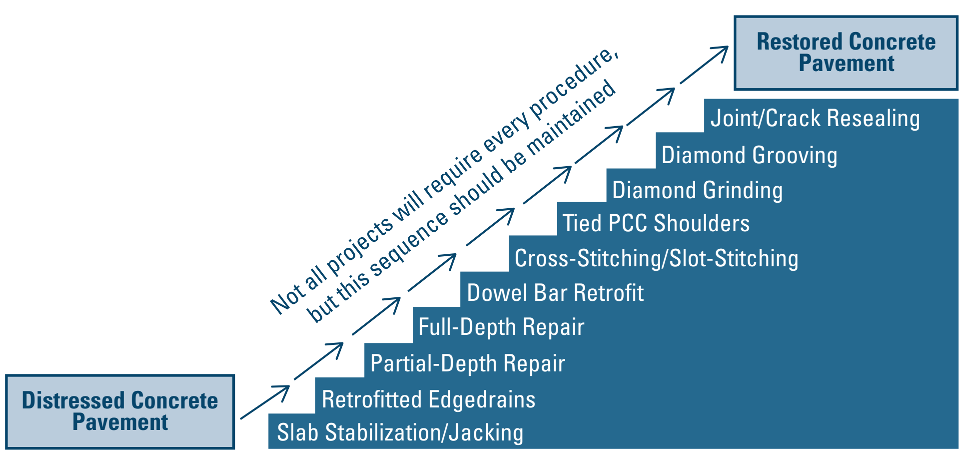
If a treatment strategy encompassing different preservation treatments is selected for use on a project, the treatments it includes are often done concurrently to maximize construction productivity and reduce user delay costs. However, it is important to conduct discrete preservation treatments in a logical construction order that maximizes the effectiveness of each individual treatment while protecting any repairs that were just performed (ACPA 2008). For example, if a treatment strategy for a given project includes FDRs and PDRs, dowel bar retrofitting, diamond grinding, and joint resealing, the FDRs and PDRs and DBR should be performed prior to the diamond grinding to maximize the resulting smoothness, whereas the joint resealing should be done last so that it will not be damaged by the diamond grinding.

Therefore, the preferred order of applying multiple pavement preservation techniques concurrently on a given project is shown in Figure 12.2 (ACPA 2008).

Few projects will require or include all of the listed preservation treatments, but for any given project, the general sequence of treatment applications presented in Figure 12.2 should be maintained. (Although concrete overlay solutions are not shown in Figure 12.2, it is recognized that many of the preservation treatments that are listed—primarily the repair options—can be used to prepare an existing pavement to receive an overlay.)

3. Summary

This chapter described several basic steps that can be used to determine the most appropriate preservation treatment strategy for a given concrete pavement project. The process begins with conducting a pavement evaluation and then determining the causes of any observed distress. Next, treatments that address the identified deficiencies are selected (and ordered in a logical sequence to maximize the effectiveness of all treatments). After filtering against any outside constraints that have been identified, feasible treatment strategies (i.e., combinations of treatments) are determined and a cost-effectiveness analysis is conducted for each, whereby the benefits and costs associated with applying the treatment strategy's selected treatments over a long analysis period are computed. Finally, the most appropriate treatment strategy is selected using a strategy decision matrix that systematically and rationally considers the results of the cost-effectiveness analysis as well as other important economic and noneconomic factors (including environmental and social considerations).



Adapted from ACPA 2018, ©ACPA 2008, used with permission

Figure 12.2. Recommended sequence for performing multiple pavement preservation activities concurrently on a given project

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