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CP Tech Center

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Second Edition

CONCRETE PAVEMENT PRESERVATION GUIDE

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16. Abstract

This document provides valuable guidance and information on the selection, design, and construction of cost-effective concrete pavement preservation treatments. It is based on a document prepared in 2008 but has been revised and expanded to include updated information 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.

In addition to several introductory chapters covering pavement preservation concepts and pavement evaluation, eight chapters on specific concrete pavement preservation treatments are included: slab stabilization, partial-depth repairs, full-depth repairs, retrofitted edge drains, load transfer restoration, diamond grinding, joint resealing, and concrete overlays. Each of these chapters discusses the purpose of each treatment, its limitations and effectiveness, material and design considerations, construction recommendations, and quality assurance/troubleshooting information. In addition, a final chapter is included on strategy selection procedures.

Form DOT F 1700.7 (8-72) **Reproduction of completed page authorized**

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 L shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International Symbol of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

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CONCRETE PAVEMENT PRESERVATION GUIDE

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About This Guide

This 2nd edition of the *Concrete Pavement Preservation Guide* is a product of the National Concrete Pavement Technology Center (CP Tech Center) at Iowa State University, with funding from the FHWA DTFH61-12-H-00010. The entire series of the CP Tech Center's manuals and other concrete-related documents is on the center's website, www.cptechcenter.org/.

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Abbreviations Used in This Guide

AASHTO	American Association of State Highway and Transportation Officials
ACC/MVK	crashes (accidents) per million vehicle kilometers
ACPA	American Concrete Pavement Association
ASR	alkali-silica reactive/reactivity/reaction
BCR	benefit-to-cost ratio
Caltrans	California DOT
CBR	California bearing ratio
CDOT	Colorado DOT
CRCP	continuously reinforced concrete pavement
CTMeter	circular track meter or circular texture meter
dB	decibels
dBA	A-weighting scale
DBR	dowel bar retrofit
DCP	dynamic cone penetrometer
DI	distress index
DOT	department of transportation
DPI	DCP penetration index
ESAL	equivalent single axle load
FDR	full-depth repair
FHWA	Federal Highway Administration
FWD	falling weight deflectometer
GPR	ground penetrating radar
GPS	global position system
HDPE	high-density polyethylene
IGGA	International Grooving and Grinding Association
IRI	international roughness index
IRPS	inertial road profiling system
JPCP	jointed plain concrete pavement
JRCP	jointed reinforced concrete pavement
LCCA	life-cycle cost analysis
LIDAR	light detecting and ranging
LTE	load transfer efficiency
LTPP	long-term pavement performance
MEPDG	Mechanistic-Empirical Pavement Design Guide
MIT	magnetic imaging tomography
MnDOT	Minnesota DOT
MPD	mean profile depth
MRD	materials-related distress
MTD	mean texture depth
MUTCD	Manual on Uniform Traffic Control Devices
NDOR	Nebraska Department of Roads
NDT	nondestructive testing
NGCS	next generation concrete surface
NHI	National Highway Institute
NHS	National Highway System
OBSI	on-board sound intensity
OM	optical microscopy
OTCS	optimized texture for city streets
PCA	Portland Cement Association
PCI	pavement condition index
PDR	partial-depth repair
PennDOT	Pennsylvania DOT
PGED	prefabricated geocomposite edgedrains
PSR	present serviceability rating
QPL	qualified products list
RDD	rolling dynamic deflectometer
RQI	ride quality index
RSL	remaining service life
RWD	rolling wheel deflectometer
SEM	scanning electron microscopy
SHRP 2	Strategic Highway Research Program 2
TSD	traffic speed deflectometer
WSDOT	Washington State DOT
XRD	x-ray diffraction

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Chapter 1
Introduction

1. Overall Learning Outcomes

This *Concrete Pavement Preservation Guide (*the *Preservation Guide)* and the accompanying workshop materials have been prepared to provide guidance on the design, construction, and selection of concrete pavement preservation treatments. The overall learning outcomes of this training course are the following:

- Define pavement preservation.
- Describe the importance of pavement management data in project and treatment selection.
- List the major components of the pavement evaluation process and the types of information gained from each.
- Identify the purpose and suitable application of various concrete pavement preservation treatments.
- Describe recommended materials and construction/installation practices for each preservation treatment.
- List critical factors to consider in the selection of concrete pavement preservation treatments.

2. Introduction

The need for the effective management of transportation assets has never been greater. In an era of an aging infrastructure, ever-increasing traffic demands, and shrinking budgets, transportation agencies are continually being asked to "do more with less" in maintaining the condition of their facilities. Pavements represent a large part of that transportation infrastructure, and the need for their effective management is just as acute. Pavements that are left to deteriorate without timely preservation or maintenance treatments are likely to require major rehabilitation and reconstruction much sooner, and those are costly and disruptive activities. In that vein, pavement preservation activities may be applied for a variety of reasons:

- Reduce water infiltration in the pavement structure.
- Prevent the intrusion of incompressibles into joints or cracks.
- Correct localized distress.
- Improve slab support conditions.
- Improve load transfer capabilities.
- Improve smoothness and rideability.
- Improve friction.
- Reduce noise.
- Improve and manage the overall conditions of a pavement network.

Recent years have also seen a growing number of transportation agencies and organizations embracing principles of sustainability, in which key environmental, social, and economic factors are considered in the decision-making process. The pavement preservation approach fits well into this sustainability framework, as the 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, shorter lane closure durations), and provide cost-effective solutions when applied at the right time and using effective procedures. In other words, 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 as well.

For concrete pavements, there are a variety of preservation treatments available to help agencies effectively manage their pavement network. In order for these treatments to be most effective, however, they must be

- Applied to the right pavement at the right time.
- Effectively designed for the existing design conditions and prevailing design constraints.
- Properly constructed or installed using proven construction practices and procedures.

This *Preservation Guide* provides valuable information and guidance on these and other critical concrete pavement preservation issues. The purpose of the document is to provide the most up-to-date information available on the selection, design, and construction of cost-effective concrete pavement preservation strategies. It concentrates primarily on strategies and methods that are applicable at the project level, and not at the network level, where pavement management activities function and address such issues as prioritizing and budgeting. It is recognized, however, that effective pavement management programs are critical in identifying or forecasting the need for timely pavement preservation treatments, and that important link is highlighted in this document.

In addition to serving as a stand-alone technical document on concrete pavement preservation, this *Preservation Guide* is also the primary reference for a two-day workshop on concrete pavement preservation. The first edition of this book (Smith et al. 2008a) and its accompanying workshop materials (Smith et al. 2008b, Smith et al. 2008c, and a series of PowerPoint® presentations) were released in 2008, and in the ensuing five years the workshop was presented to a number of highway agencies across the country. This second edition, now referred to as the *Preservation Guide*, has been modified and updated to reflect recent advancements and new developments in the concrete pavement preservation arena. For example, among some of the changes/modifications that have been made to the document are the following:

- Discussion of the importance of pavement management systems in identifying candidate projects for pavement preservation.
- Additional information on new pavement evaluation equipment, technologies, and protocols.
- Incorporation of new materials and techniques for partial-depth repairs.
- Addition of precast full-depth repair (FDR) technologies.
- Addition of guidelines for utility cut repairs.
- Addition of a new chapter on concrete overlays.
- Discussion of general sustainability considerations in the selection of pavement preservation treatments.

The accompanying workshop materials were updated to reflect these and all other modifications, and a number of new graphics were incorporated into the workshop presentation slides as well.

The intended audience for the *Preservation Guide* is quite diverse and includes design engineers, quality control personnel, contractors, suppliers, technicians, and trades people. While the course is aimed at those who have some familiarity with concrete pavements and pavement preservation, it should also be of value to those who are new to the pavement field.

3. Document Organization

In addition to this introductory chapter, this *Preservation Guide* contains the following chapters:

- Chapter 2. Preventive Maintenance and Pavement Preservation Concepts
- Chapter 3. Concrete Pavement Evaluation
- Chapter 4. Slab Stabilization and Slab Jacking
- Chapter 5. Partial-Depth Repairs
- Chapter 6. Full-Depth Repairs
- Chapter 7. Retrofitted Edgedrains
- Chapter 8. Dowel Bar Retrofit, Cross Stitching, and Slot Stitching
- Chapter 9. Diamond Grinding and Grooving
- Chapter 10. Joint Resealing and Crack Sealing
- Chapter 11. Concrete Overlays
- Chapter 12. Strategy Selection

Chapter 2 provides general background information on pavement maintenance and pavement preservation, including an overview of anticipated benefits and current initiatives. This is followed by Chapter 3 on pavement evaluation, which includes discussions on condition surveys, nondestructive testing, roughness and friction assessment, and materials and laboratory testing. These two chapters establish a strong foundation for the discussions on concrete pavement preservation treatments, which are covered in Chapters 4–11. Each of these chapters shares the following elements:

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

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

4. Accompanying Workshop

As mentioned previously, this *Preservation Guide* also serves as the technical resource document for a two-day workshop on concrete pavement preservation. That workshop presents the key information contained in the *Preservation Guide* to the pavement practitioner, using visual aids and interactive presentations. The visual aids (typically PowerPoint slides) are used to highlight critical aspects of each chapter and can feature graphics, photographs, and videos. A separate *Participant Workbook* is used to support the presentation and delivery of the workshop materials; it has been developed to assist workshop participants in following the presentation of the technical materials and to facilitate comprehension of the information. Contents of the *Participant Workbook* include the following:

- General course information, including an introduction, learning objectives, and class schedule.
- Introduction to each training module.
- Copies of all visual aids, with space for noting key concepts and discussion topics that are covered in the workshop.

In the delivery of the workshop, the *Preservation Guide* and the *Participant Workbook* are meant to be used together as technical resources, both during the workshop presentation and afterward. The *Preservation Guide* includes detailed technical information on the design and construction of concrete pavement preservation treatments, whereas the *Participant Workbook* has been developed to highlight the key learning points from each chapter.

5. Additional Information

This *Preservation Guide* presents a considerable amount of information on concrete pavement preservation treatments. There are a number of topics, however, that cannot be given a complete treatment because of the scope of the document and overall space limitations. Numerous references are cited throughout the document to provide interested readers with additional (and more detailed) sources of information. Many of these references are available from the organizations listed in Table 1.1.

Table 1.1. Sources of Additional Information

Federal Highway Administration	
Office of Asset Management, Pavement, and Construction 1200 New Jersey Avenue SE Washington, DC 20590 www.fhwa.dot.gov/pavement	Office of Research, Development, and Technology Turner-Fairbank Highway Research Center 6300 Georgetown Pike McLean, VA 22101 www.fhwa.dot.gov/research
National Highway Institute 4600 North Fairfax Drive, Suite 800 Arlington, VA 22203 www.nhi.fhwa.dot.gov/home.aspx	National Center for Pavement Preservation 2857 Jolly Road Okemos, MI 48864 www.pavementpreservation.org
Industry	
American Concrete Pavement Association (ACPA) 5420 Old Orchard Road, Suite A100 Skokie, IL 60077 www.pavement.com	International Grooving & Grinding Association (IGGA) 12573 Route 9W West Coxsackie, NY 12192 www.igga.net
National Concrete Pavement Technology Center 2711 South Loop Drive, Suite 4700 Ames, IA 50010 www.cptechcenter.org	Portland Cement Association (PCA) 5420 Old Orchard Road Skokie, IL 60077 www.cement.org
Other	
American Association of State Highway and Transportation Officials (AASHTO) 444 N. Capitol Street, NW, Suite 249 Washington, DC 20001 www.transportation.org	American Society of Civil Engineers (ASCE) 1801 Alexander Bell Drive Reston, VA 20191 www.asce.org

6. References

Smith, K. D., T. E. Hoerner, and D. G. Peshkin. 2008a. *Concrete Pavement Preservation Workshop: Reference Manual*. Ames, IA: National Center for Concrete Pavement Technology, Iowa State University.

Smith, K. D., T. E. Hoerner, and D. G. Peshkin. 2008b. *Concrete Pavement Preservation Workshop: Instructor Guide.* Ames, IA: National Center for Concrete Pavement Technology, Iowa State University.

Smith, K. D., T. E. Hoerner, and D. G. Peshkin. 2008c. *Concrete Pavement Preservation Workshop: Participant Workbook.* Ames, IA: National Center for Concrete Pavement Technology, Iowa State University.

Chapter 2

Preventive Maintenance and Pavement Preservation Concepts

1. Learning Outcomes

This chapter presents an overview of preventive maintenance and pavement preservation. Upon completion of this chapter, the participants will be able to accomplish the following:

- Define pavement preservation and preventive maintenance.
- Describe the characteristics of suitable pavements for pavement preservation.
- List some of the benefits of pavement preservation.
- List and describe the types of concrete pavement preservation treatments available for use.
- List and describe the key factors affecting project and treatment selection.
- Describe the importance of pavement management data in project and treatment selection.

2. Introduction

The Federal Highway Administration (FHWA) has been a strong proponent and supporter of the concept of cost effectively preserving the country's roadway (pavement) network, which has spurred a nationwide movement of pavement preservation and preventive maintenance programs. This is indeed a radically different approach to managing pavement networks than what has been used in the past. One of the big differences between past approaches and today's emphasis on preservation and preventive maintenance is that preservation focuses on being "proactive" rather than "reactive." The concept of adopting a proactive maintenance approach enables agencies to reduce the probability of costly, time-consuming rehabilitation and reconstruction projects. One result is that the traveling public has benefited from improved safety and mobility, reduced congestion, and smoother, longer-lasting pavements (Geiger 2005). This is the true goal of pavement preservation, a goal that the FHWA—through its partnership with state highway agencies, local agencies, industry organizations, and other interested stakeholders—is committed to achieve (Geiger 2005).

The enactment of the Moving Ahead for Progress in the 21st Century Act (MAP-21) in 2012 reinforces the importance of pavement preservation and recognizes it as a valuable component in the Federal highway program. The two-year transportation reauthorization bill invests in an expanded National Highway System (NHS), with more than half of the funding going to preserving and improving the most important highways (FHWA 2012). The act references asset management as "a structured sequence of maintenance, preservation, repair, rehabilitation, and replacement actions that will achieve and sustain a desired state of good repair over the life cycle of the assets at minimum practicable cost." Moreover, the act lists preservation among the various activities that are eligible for NHS project funding as part of one of the core programs—the National Highway Performance Program (NHPP)—included in the highway legislation. Hence, the opportunities to use preservation techniques now and in the future continue to grow.

This chapter introduces many of the pavement preservation concepts currently being promoted by the FHWA. Specifically, this chapter introduces common pavement preservation-related definitions, discusses the importance and benefits of preservation, presents the different types of concrete pavement preservation treatments that are available for use, and describes the importance of pavement management data in making decisions about pavement preservation.

3. Description of Pavement Preservation

During the evolution of pavement preservation concepts over the past several decades, a number of recurring questions commonly arise:

- What is pavement preservation and how does it differ from rehabilitation?
- What is the difference between "pavement preservation" and "preventive maintenance"?
- How does "preventive maintenance" differ from "corrective maintenance"?
- What characteristics make a treatment fit into the "preventive" category?

These and other questions are perhaps best answered by a graphical illustration and some established terminology. A general schematic indicating the relative timing of pavement preservation, rehabilitation, and other activities is shown in Figure 2.1. As can be seen, the preservation area of the curve is the portion that covers the early years of the constructed pavement. It includes

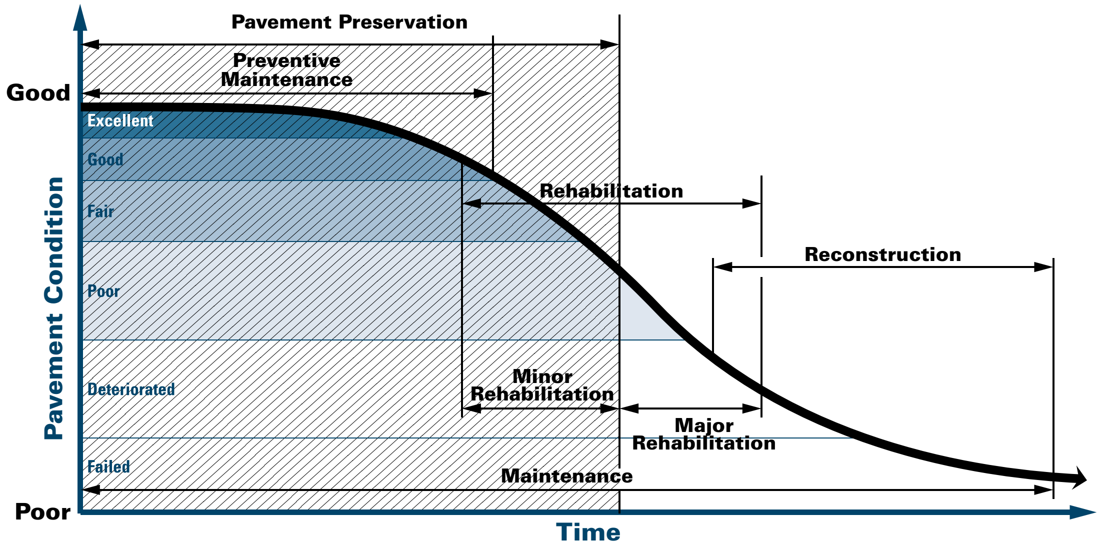

Figure 2.1. Representation of definitions of pavement preservation, rehabilitation, and reconstruction (adapted from Peshkin et al. [2011])

preventive maintenance and some minor or light rehabilitation, which may be applied when the pavement is in relatively good to very good condition.

In order to promote a uniform understanding of pavement preservation among all agencies, the FHWA issued a memorandum in 2005 providing the following terms and definitions for a range of pavement-related activities (Geiger 2005):

- **Preventive Maintenance**—In 1997, the AASHTO Standing Committee on Highways defined preventive maintenance as "a planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system (without significantly increasing the structural capacity)" (Geiger 2005). Preventive maintenance is typically applied to pavements in relatively good condition and that have significant remaining service life.
- **Pavement Preservation**—As defined initially by the FHWA Pavement Preservation Expert Task Group (ETG) and reiterated in MAP-21, pavement 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 extends pavement life, improves safety and meets motorist expectations" (Geiger 2005). This goal is achieved in practice through the application of preventive maintenance, minor rehabilitation (nonstructural), and some routine maintenance activities, as described below. The distinctive characteristics of pavement preservation activities are that they restore the function of the existing system and extend its service life but do not increase its load-carrying capacity or strength.
- **Routine Maintenance**—Routine maintenance consists of day-to-day activities that are scheduled by maintenance personnel to maintain and preserve the condition of the highway system at a satisfactory level of service. Depending on the timing of application, the nature of the distress, and the type of activity, certain routine maintenance activities may be classified as preservation. Routine maintenance activities are often performed using agency forces.
- **Corrective Maintenance**—Corrective maintenance activities are performed in response to the development of deficiencies that negatively impact the safe, efficient operations of the facility and the future integrity of the pavement section. Corrective maintenance activities are generally reactive, not proactive, and performed to restore a pavement to an acceptable level of service.
- **Pavement Rehabilitation**—Pavement rehabilitation activities are those that extend the life of existing pavement structures either by (1) restoring existing structural capacity, or (2) increasing pavement thickness to strengthen it to accommodate existing or projected traffic loadings.
- **Pavement Reconstruction**—Reconstruction is the replacement of the entire existing pavement structure by the placement of the equivalent or increased pavement structure. Reconstruction usually requires

the complete removal and replacement of the existing pavement structure. Reconstruction may utilize either new or recycled materials incorporated into the materials used for the reconstruction of the complete pavement section. Reconstruction is required when a pavement either has failed or has become functionally obsolete.

4. Benefits of Pavement Preservation

Pavement preservation is being embraced by more and more agencies because it is a logical approach to preserving assets that offers measurable benefits to the agency. Some of the benefits that have been cited as being important reasons for implementing or upgrading preservation programs include the following:

- **Higher Customer Satisfaction**—Customer satisfaction is at the heart of successful preservation practices. From project selection to treatment selection to construction, a good preservation program will benefit users by way of safer roads, reduced traffic disruption, and an overall pavement network in a higher level of functional condition in terms of smoothness and noise.
- **Improved Pavement Condition**—Agencies that have implemented pavement preservation programs are not simply looking for a new way of doing the same old thing. The conventional approach most agencies take to manage their pavements consists of a combination of routine and corrective maintenance and rehabilitation. As previously described, routine and corrective maintenance activities are primarily reactive processes in which existing distresses are repaired or treated. Rehabilitation is typically programmed following the "worst first" principle, in which pavements are allowed to deteriorate until the worst one rises to the top of the capital projects list. In contrast, pavement preservation is a proactive approach intended to preserve a pavement and extend its useful performance period or cycle. The difference between these two approaches is substantial and central to the preservation concept. If pavements in good condition are kept in good condition longer, delaying the need for rehabilitation and reconstruction, then an obvious benefit is overall improved conditions.
- **Cost Savings**—From an agency standpoint, probably the most sought-after benefit of pavement preservation is financial. Saving money through a policy of preservation is certainly the primary benefit, but one that has been difficult to substantiate. Nonetheless, a number of agencies have reported or projected cost savings from preservation strategies and have often used the information as a persuasive argument for their adoption. Such savings are in the form of both less expensive treatments and pavements with extended service lives. Additional cost savings may be in the form of decreased user costs that result from reduced time delays (shorter and fewer work zones), lower vehicle operating costs (shorter and fewer work zones, smoother roads), and lower crash-related costs. Preservation treatments are, by their very nature, less expensive than many alternatives. In addition, if these treatments can delay the need for more expensive repairs, agencies are expected to realize cost savings. An example of the savings documented by one agency in the 1990s is shown in Figure 2.2, where the comparative costs of treatments applied at different times in the life of the pavement are presented.

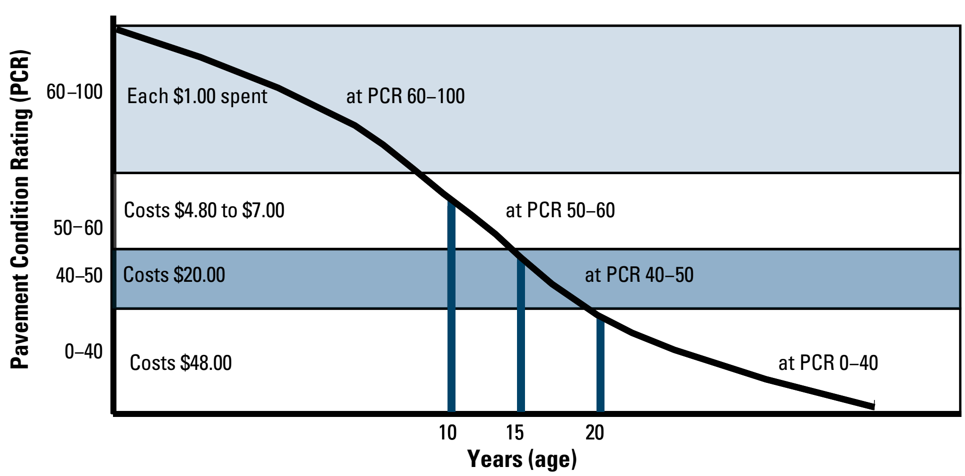

Figure 2.2. Comparison of treatment costs at different conditions/ages (Zimmerman and Wolters 2003)

- **Increased Safety**—Most highway users cite safety as one of their fundamental expectations from the roads on which they travel. As such, safety is an extremely important priority at the national, state, and local levels, and pavement preservation programs provide both implicit and explicit safety benefits that address this priority. Explicitly, today's treatments are specially designed to provide safer surfaces through the use of more polish-resistant aggregates and greater surface macrotexture that, when combined, greatly increase wet-weather surface friction. Implicitly, the safety benefits are obtained by keeping the pavement in better overall condition. Pavements with higher condition ratings are smoother and have fewer defects that contribute to safer operating conditions. Pavements in better overall condition also require fewer and less disruptive repairs, which reduces the chances for work zone-related crashes.

For any of the above benefits to be realized, the preservation treatment must be placed on a pavement that is a good candidate for preservation, and it must be properly designed, properly constructed, and properly maintained throughout its life.

5. Pavement Preservation Treatments

There is a broad range of treatments that can be used in the preservation of concrete pavements; see Table 2.1. These treatments use different materials (or, in some cases, no materials), may be applied either globally across the pavement or locally where specific distresses or other issues exist, and may involve a small amount of removal of the existing pavement and/or the placement of new material. Although each treatment is generally applicable to the various types of concrete pavement (i.e., jointed plain concrete pavement [JPCP], jointed reinforced concrete pavement [JRCP], and continuously reinforced concrete pavement [CRCP]), there are some obvious exceptions. For example, transverse joint resealing is not performed on CRCP because that pavement type contains no regularly spaced transverse joints.

Table 2.1. Concrete Pavement Preservation Treatment Types (adapted from Peshkin, et al. [2011])

Treatment Type	Treatment Description
Slab Stabilization	Filling of voids beneath concrete slabs by injecting cement grout, polyurethane, 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
Partial-Depth Repair	Removal of small, shallow (top one-third of the slab) areas of deteriorated concrete and subsequent replacement with a cementitious or polymeric repair material
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
Retrofitted Edgedrains	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
Dowel Bar Retrofit	Placement of dowel bars across joints or cracks in an existing concrete pavement to restore load transfer
Cross Stitching	Placement of deformed tiebars into holes drilled at an angle through cracks (or, in some cases, joints) in an existing concrete pavement
Diamond Grinding	Removal of a thin layer of concrete (typically 0.12 to 0.25 inches) 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)
Joint Resealing	Removal of existing deteriorated transverse and/or longitudinal joint sealant (if present), refacing and pressure cleaning the joint sidewalls, and installing new material (liquid sealant and backer rod or preformed compression seal)
Crack Sealing	Sawing, power cleaning, and sealing cracks (typically transverse, longitudinal, and corner-break cracks wider than 0.125 inch) in concrete pavement using high-quality sealant materials
Concrete Overlay	Placement of a thin concrete layer (typically 3 to 4 inches thick) to a milled or prepared surface

Table 2.2. Primary Capabilities and Functions of Concrete Pavement Preservation Treatments (adapted from Peshkin et al. [2011])

Treatment	Prevention/Delay				Restoration/Improvement			
	Seal/ Waterproof Pavement/ Minimize Pumping	Fill Voids and Restore Support	Remove Moisture Beneath Structure	Prevent Intrusion of Incompressible Materials	Remove/ Control 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			✓		✓			
Dowel Bar Retrofit					✓		✓	
Cross Stitching					✓		✓	
Diamond Grinding					✓	✓	✓	✓
Diamond Grooving						✓		
Joint Resealing	✓			✓				
Crack Sealing	✓			✓				
Thin Concrete Overlay						✓	✓	✓

Table 2.2, above, indicates the unique capabilities and functions of each treatment in terms of its impact on the structural and/or functional performance of the existing pavement. These impacts 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 the pavement, and improving surface characteristics related to user safety and comfort.

6. Pavement Management Data for Successful Preservation

Pavement preservation programs rely on proper treatment selection and timing of the treatment to be successful. In order to select the right treatment for the right pavement at the right time, the following information must be compiled and analyzed:

- Structure and condition of the existing pavement.
- Current and projected traffic.
- Local climatic conditions.
- Expected performance of the pavement.
- 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 treatments that affect selection.

The expected performance of the treated pavement is perhaps the most challenging information to obtain. The performance is a function of the type of preservation treatment used, the structure and condition of the existing pavement, and the forecasted traffic and climatic conditions. As discussed below, past performance and preservation event data are important in developing estimates of expected future performance.

The availability of the above information is an essential part of the process of managing a successful preservation program. Successful programs exploit the data available from a pavement management system (PMS) to help in the decision-making process. Although highway agencies collect and analyze pavement management data in different ways, the importance of using the following types of data to assess needs and to program and apply a range of treatments is widely recognized:

- Existing pavement structure/history (including past preservation treatment applications).
- Traffic loadings.
- Distress types (e.g., faulting, cracking, spalling), severity levels, and amounts.
- Overall condition indexes/ratings.
- Surface profile/smoothness.
- Surface friction and macrotexture.
- Nondestructive testing (NDT) data (e.g., deflections, load transfer efficiencies, and back-calculated layer moduli).

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. Historical condition and performance data can be used to develop time-series pavement performance models, which can be used to select the preferred project treatment based on expected performance and cost effectiveness.

A survey by the FHWA Pavement Preservation ETG Rigid Subcommittee examined highway agency practices regarding how concrete pavement preservation is integrated into the pavement management systems of state highway agencies (Scofield et al. 2011). Of the 38 responding highway agencies, 23 indicated using trigger values for concrete pavement management/preservation. Among these agencies, smoothness is one of the commonly used indicators for trigger and/or threshold criteria for preservation treatments, although overall condition and key distresses (such as joint faulting and slab cracking) are sometimes used. Nine highway agencies (Kansas, Michigan, Minnesota, New Jersey, North Carolina, Oklahoma, Pennsylvania, South Carolina, and Washington) were identified as having best practices for use of trigger values. As examples, Tables 2.3 and 2.4 highlight the criteria used by Michigan and New Jersey Departments of Transportation (DOTs), respectively.

It is noteworthy that the state practices survey showed very little use of friction and other individual distresses (besides faulting and cracking) for decision criteria for applying preservation treatments. Because preservation objectives and functions are somewhat broad, there is

Table 2.3. Michigan DOT Criteria for Preservation Strategies (Scofield et al. 2011)

Strategy	Minimum RSL	DI	RQI	IRI
FDR	7	< 20	< 54	< 107
Joint Resealing	10	< 15	< 54	< 107
Crack Sealing	10	< 15	< 54	< 107
Diamond Grinding	12	< 10	< 54	< 107
Dowel Bar Retrofit	10	< 15	< 54	< 107
Concrete Pavement Restoration*	3	< 40	< 80	< 212

*Consists of full-depth concrete repairs, diamond grinding, and other.
RSL: Remaining service life
DI: Distress index
RQI: Ride quality index
IRI: International roughness index

Table 2.4. New Jersey DOT Criteria for Preservation Strategies (Scofield et al. 2011)

Condition Status	Condition Index Criteria	Engineering Significance
Poor	IRI > 170 or SDI < 2.4	Roads overdue for treatment
Fair	95 < IRI < 170 and SDI > 2.4 or IRI < 95 and 2.4 < SDI < 3.5	Roads exhibit minimally acceptable ride
Good	IRI < 95 and SDI > 3.5	Roads exhibit good ride quality

IRI: International roughness index
SDI: Surface distress index

a need to consider a multitude of conditions in determining if a treatment is suitable or not. Moreover, there is a need to consider other conditions or indicators—such as NDT data, voids, or delaminations—that would allow for a more proactive approach to applying pavement preservations treatments. It currently is not practical, however, for highway agencies to collect such data at the network level.

Pavement management data are critical to defining the predicted performance of preservation treatments, which helps determine if they are feasible for a project and if they are cost effective. In the past and even to some extent today, treatment performance has been characterized as the life of the preservation treatment (i.e., how long the treatment lasts until another preservation treatment, or even a rehabilitation treatment, is needed). An alternative way of characterizing treatment performance involves quantifying the effectiveness of the 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 the additional amount of 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, spalling) and/or key pavement surface characteristics (e.g., smoothness, friction, texture, pavement-tire noise).

Pavement performance benefits can be quantified in terms of the area under the pavement condition/performance curve; the greater the area, the more benefit provided by the treatment. Like pavement life extension, 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 performance benefit area, but its use in evaluating rigid pavement preservation treatments has been very limited. A recent Strategic Highway Research Program (SHRP) 2 study summarized the performance of several rigid pavement preservation treatments, as reported in various studies (Peshkin et al. 2011). Table 2.5 shows the expected performance ranges for the various treatments in terms of treatment life. Although the study attempted to also provide estimates of pavement life extension, insufficient data were available for most of the treatments.

Table 2.5. General Expected Performance of Preservation Treatments Applied to Concrete-Surfaced Pavements (adapted from Peshkin et al. [2011])

Treatment	Expected Performance (Treatment Life), Years
Concrete joint resealing	2 to 8
Concrete crack sealing	4 to 7
Diamond grinding	8 to 15
Diamond grooving	10 to 15
Partial-depth concrete patching	5 to 15
Full-depth concrete patching	5 to 15
Dowel bar retrofit	10 to 15

A valuable resource in the evaluation of the need for and the timing of preservation treatments is the *Mechanistic-Empirical Pavement Design Guide* (MEPDG), available from AASHTO and now referred to as Pavement ME Design (AASHTO 2008). This mechanistic-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 procedure can be used to forecast rehabilitation and preservation needs using established trigger values for distress and/or smoothness, and it can also be used to predict the performance of preservation treatments. For example, the procedure can be used to define the pavement preservation window in terms of smoothness and distresses, which can aid an agency's pavement planning efforts. It can also be used to determine when a diamond-ground pavement will reach a certain smoothness and/or faulting threshold, which in turn can provide an estimate of both treatment life and pavement life extension.

7. Enhancing and Sustaining Pavement Preservation Programs

Many state highway and local agencies continue to enhance their preservation programs and practices. Key factors in developing and sustaining successful preservation programs include the following:

- **Preservation Engineer**—Some state agencies, such as California, Louisiana, North Carolina, Minnesota, and New York, have either a person whose specific title is Preservation Engineer or a person who is solely responsible for the preservation program. This designation provides several benefits. In addition to having an individual who can help improve preservation practices throughout the agency, it also helps to substantially raise the profile of preservation and preventive maintenance and thereby ensure that the programs are sustainable beyond the short term.

- **Guide Documents**—Best practices manuals and guideline documents that describe how to go about performing effective preservation can be a tremendous boon to an agency. These materials typically include information about the various treatments in use locally, what they do, when they should be used, where they should be used, how they should be constructed, what benefits result from the proper use of the treatments, and so on. Examples include Caltrans's *Maintenance Technical Advisory Guide* (Caltrans 2007), Nebraska's *Pavement Maintenance Manual* (NDOR 2002), Pennsylvania's Pavement Policy Manual (PennDOT 2010), and Colorado's *Preventive Maintenance Program Guidelines* (CDOT 2004). By having a manual or guide, agencies can communicate what constitutes accepted practice, what works well locally, and what resources are available for additional information.

- **Test Sections**—By constructing local test sections (using locally available or appropriate treatments perhaps applied to pavements of varying ages and stages of deterioration), an agency can develop a better understanding of what works well for its specific materials and conditions. Test sections can also supplement pavement performance information in a pavement management database to help improve treatment timing and to establish performance trends/models that can help determine which treatments are feasible and cost effective for a particular type of project. Information from the test sections could provide greater clarity to the inconclusive results of previous nationwide studies, such as the SHRP SPS-4 (*Preventive Maintenance of Jointed Concrete Pavement*) and SPS-6 (*Rehabilitation of Jointed Concrete Pavement*) studies (both in progress).

- **Research and Training**—In addition to the research associated with test sections (described above), research into the use of locally available materials, construction methods, and programming issues can only help to improve practice through an expanded knowledge base. And because quite often preservation is so different from previous practice, training targeted at specific audiences will provide a better understanding of the importance of preservation and help to improve implementation efforts. Training is available from a number of industry sources, as well as the National Highway Institute (NHI) (www.nhi.fhwa.dot.gov), which has been offering several courses on pavement preservation and preventive maintenance since 1999. Some state highway administrations (SHAs), such as California, Texas, and Ohio, have developed their own training programs, while others (Pennsylvania, North Carolina) have adapted NHI courses for local conditions.

8. Summary

Pavement preservation is by no means a new concept, but as its use grows, more and more agencies are getting a better idea of what it means. Although there are several refined definitions of what preservation means, the definition "keeping good roads in good condition" has emerged as a popular mantra for many highway agencies.

There are many good reasons to implement a pavement preservation program, and the forces that are at play in today's public agencies–tightened budgets, staff reductions, and greater public scrutiny of their decision making–almost require a preservation approach. The benefits of preservation, however, will not be realized if sound practice in project evaluation and selection are not employed. The role of the timing of the treatment application, as well as the types of data collection that are required to help in the decision-making process, are briefly introduced. While these topics are covered in detail elsewhere (for example, Peshkin et al. 1999; Peshkin, Hoerner, and Zimmerman 2001; Peshkin et al. 2011), they are briefly introduced here.

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Chapter 3

Concrete Pavement Evaluation

1. Learning Outcomes

This chapter presents a summary of the overall pavement evaluation process, and it includes a description of the various pavement evaluation activities that are commonly conducted. Furthermore, this chapter describes how the results from the different pavement evaluation activities are brought together in an overall project evaluation. The results of the overall evaluation are used to assist in selecting cost-effective pavement preservation treatments. Upon completion of this chapter, the participants will be able to accomplish the following:

- Describe the need for a pavement evaluation and the uses of pavement evaluation data.
- List the common pavement evaluation components and what information is obtained from each.
- Describe how pavement evaluation data are used and interpreted.

2. Introduction

Prior to selecting any preservation or rehabilitation treatment for a given pavement, it is important to conduct a pavement evaluation to determine the causes and extent of pavement deterioration. This requires a systematic data collection effort and an analysis of the structural and functional condition of the pavement, as well as several other factors. 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—Manual of Practice* (AASHTO 2008a).

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 shortened to meet available funding levels). Additionally, critical projects on major highways and projects subjected to high traffic volumes require more comprehensive pavement evaluations than those on low-volume highways. This is not because data collection is less important on lower-volume highways, but because the effects of premature failures on the higher-volume highways are much more serious.

This chapter describes pavement evaluation procedures and processes that can be used to assist in the selection of appropriate preservation treatments for concrete pavement projects. The 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 evaluation process. Components of the pavement evaluation are then presented:

- Distress and drainage surveys.
- NDT.
- Evaluating pavement surface characteristics (including noise, roughness, friction, and texture).
- Field sampling and testing.

It should be noted that not all of these evaluation components are needed for a given pavement evaluation 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 conditions and characteristics exhibited by the existing pavement.

3. Data for Pavement Evaluation

Pavement evaluation can require the collection of a substantial amount of data about and from the existing pavement, depending on the condition of the pavement, the location, the type of facility, and so on. These data can be divided into the following major categories:

- 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).
- 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 to be collected under each of these general categories also depends upon the treatment alternatives being considered. For example, if 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 (AASHTO 1993). The data are classified as those that are “Definitely Needed,” “Desirable,” or “Not Normally Needed” in the pavement evaluation process.

The data collection effort serves the following important purposes in the overall pavement evaluation process:

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

In pavement evaluation, the design engineer’s objective 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.

4. Pavement Evaluation Overview

The activities included in a pavement evaluation will vary from project to project, depending on the type of project and its relative significance. Generally speaking, the overall pavement evaluation process can be broadly divided into the following general steps (Hoerner et al. 2001; AASHTO 2008a):

- 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. This process involves the collection of data from office files, from the agency’s pavement management system, and from any other project records that provide basic information needed for conducting the pavement evaluation. The goal is to collect as much information on the existing pavement as possible, such as original design data, construction information, subgrade data, materials testing data, traffic data, performance data, and so on. Possible data sources for this data collection effort are the following:

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

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

In the early stages of the pavement evaluation process, it may also be useful to perform an assessment of remaining structural capacity of the pavement by comparing the original design traffic loadings to those that have occurred to date. This can provide the first indication as to whether or not the pavement is performing as intended, and it may suggest whether or not preservation is an appropriate solution. To aid in the assessment of the current condition, a re-evaluation of the existing pavement design can be conducted using the AASHTO *Mechanistic-Empirical Pavement Design Guide—Manual of Practice* (AASHTO 2008a), the results of which can be compared to the actual performance to see how much life has been consumed.

Table 3.1. Suggested Data Collection Needs for Concrete Pavement Treatment Alternatives (adapted from AASHTO [1993])

Data Item	Full-Depth Repair	Partial-Depth Repair	Thin Concrete Overlay	Diamond Grinding	Diamond Grooving	Slab Stabilization	Slab Jacking	Retrofitted Edgedrains	Joint Resealing	Crack Sealing	Dowel Bar Retrofit	Cross Stitching	Slot Stitching
Pavement Design	X	X	X	X		X	X	X	X	X	X	X	X
Original Construction Data			*	*	*			*	*	*	*	*	*
Age	*	*	*	*	*			*					
Materials Properties	*	*	X	X	X	*	*	X					
Subgrade			X			*	*	X	X	X			
Climate			X			X	X	X	X	X			
Traffic Loading and Volumes	X	X	X	X	X	*	*	X	*	*	X	X	X
Distress	X	X	X	X	X	X	X	X	X	X	X	X	X
Friction			*	*	*								
Crashes			*	*	*								
Potential NDT[1]	* (a, b, c, d)		X (a, b, c, d)			X (a, b, d)	X (a, b, d)				X (a, b, d)	* (a, b, c, d)	* (a, b, c, d)
Potential Destructive Testing/ Sampling[2]	X (e, h, i)	X (e, h, i)	X (e, f, g, h, i)	* (e, h, i)	* (h, i)	* (e, f, g)	* (e, f, g)	X (e, f, g)			* (e, f, g, h, i)	* (e, h, i)	* (e, h, i)
Roughness			*	*	*	*	*						
Surface Profile			*	X	X	*	*						
Drainage	X		X	X	X	X		X	X				
Previous Mainte-nance	*	*	*	*	*	*		*	*	*			
Bridge Transitions	X	X	*						X				
Utilities	X		X			*	*	*					
Traffic Control Options	X	X	X	X	X	X	X	X	X	X	X	X	X
Vertical Clearances			X										
Geometrics			X										

[1] See Section 6 of this Chapter
[2] See Section 8 of this Chapter

a = FWD (falling weight deflectometer); b = GPR (ground-penetrating radar);
c = MIT (magnetic imaging tomography) Scan 2; d = MIRA device
e = coring; f = DCP (dynamic cone penetrometer); g = subsurface material testing/characterization;
h = strength testing; i = MRD (materials-related distress) evaluation

KEY: X Definitely Needed * Desirable [blank] Normally Not Needed

Step 2: Initial Site Visit and Assessment

An initial site inspection is conducted to initially gain a general knowledge of the performance of the pavement, and then to help determine the scope of the field testing activities to be conducted in Step 3. As part of this activity, information on distress, surface roughness, surface friction, shoulder conditions, and moisture/drainage problems should be gathered. Condition data can be collected through "windshield" surveys, shoulder surveys, or using automated means (i.e., collection of longitudinal and transverse profile for determining roughness and faulting, as well as video images of surface distress at posted speeds). In addition, an initial assessment of traffic control constraints, obstructions, right-of-way zones, presence of bridges and other structures, and general safety aspects should be made during this visit.

Information obtained from this initial site visit and assessment will be used to formulate the type and extent of 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.

Discussions with local design and maintenance engineers may also be beneficial to help gain a better understanding of the overall pavement performance and whether or not 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 performance. The specific field testing activities are guided by the information obtained from the initial site visit and assessment, and they may include the following:

- **Distress and Drainage Surveys**—These surveys provide a visual indication of the structural condition of the existing pavement. The information gained from these surveys will have the greatest impact on the selection of the appropriate preservation or rehabilitation treatment, and consequently they must be carefully performed. Most highway agencies have developed their own manuals for quantifying pavement distress. The FHWA's *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2003) serves as a good source of information on pavement distresses and distress identification.
- **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, and ground-penetrating radar (GPR). These technologies may be conducted to evaluate the overall structural condition of the pavement, to assess the joint load transfer capabilities, to determine the depth of steel reinforcement, and to determine layer thicknesses. The scope of the NDT program 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 the noise, roughness, and safety (friction) demands of the traveling public.
- **Field Sampling and Testing**—Field sampling and testing activities serve several purposes, such as the confirmation of layer materials and thicknesses and the retrieval of cores and subsurface samples for later 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 a more limited component of a project evaluation and is not required on every project. When included as part of the pavement evaluation process, 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. 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 layers.
- Presence of MRD, such as alkali-silica reactivity (ASR) or D-cracking.
- Petrographic testing and analysis of the concrete layer.

- Resilient modulus of the unbound layers and of the subgrade.
- Density and gradation of underlying granular layers.

Again, however, it should be noted that the above types of information are not needed on 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 along the project to illustrate varying conditions. 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; continuous plots of load transfer, noise, roughness, and friction can also be displayed. 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 charts to help 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 both general trigger and limit values for different pave-

Table 3.2. Example of Critical Trigger and Limit Values (adapted from ACPA [1997])

Pavement Type and Performance Measure	Trigger Value/Limit Value[a]		
	High (ADT>10,000)	Medium (ADT 3,000 to 10,000)	Low (ADT<3,000)
JPCP (Joint Space <20 ft)[b]			
Low-High Severity Fatigue Cracking (% of slabs)	1.5/5.0	2.0/10.0	2.5/15.0
Deteriorated Joints (% of joints)	1.5/15.0	2.0/17.5	2.5/20.0
Corner Breaks (% of joints)	1.0/8.0	1.5/10.0	2.0/12.0
Average Transverse Joint Faulting (in.)	0.08/0.50	0.08/0.6	0.08/0.7
Durability Distress (severity)	Medium-High		
Joint Seal Damage (% of joints)	>25/—		
Load Transfer (%)	<50/—		
Surface Friction	Minimum Local Acceptable Level/—		
IRI (in./mi)	65–80/160–180	75–90/180–200	90–110/200–220
JRCP (Joint Space >20 ft)[c]			
Med-High Severity Trans. Cracking (% of slabs)	2.0/30.0	3.0/40.0	4.0/50.0
Deteriorated Joints (% of joints)	2.0/10.0	3.0/20.0	4.0/30.0
Corner Breaks (% of joints)	1.0/10.0	2.0/20.0	3.0/30.0
Average Transverse Joint Faulting (in.)	0.16/0.50	0.16/0.60	0.16/0.70
Durability Distress (severity)	Medium-High		
Joint Seal Damage (% of joints)	>25/—		
Load Transfer (%)	<50/—		
Surface Friction	Minimum Local Acceptable Level/—		
IRI (in./mi)	65–80/160–180	75–90/180–200	90–110/200–220
CRCP			
Failures (Punchouts, FDRs) (no./mi)	3/10	5/24	6/39
Durability Distress (severity)	Medium-High		
Surface Friction	Minimum Local Acceptable Level/—		
IRI (in./mi)	65–80/160–180	75–90/180–200	90–110/200–220

[a] Trigger values indicate when pavement preservation may be appropriate. Limit values indicate the need for major structural improvements. Values should be adjusted for local conditions. Actual percentage repaired may be much higher if the pavement is restored several times.

[b] Assumed slab length = 15 ft   1 mi = 1.609 km; 1 m = 3.281 ft; 1 in. = 25.4 mm

[c] Assumed slab length = 33 ft

ment performance indicators. Trigger values define the point when pavement preservation may be appropriate, whereas limit values define the point at which the pavement is in need of major structural improvements and essentially beyond the level of pavement preservation effectiveness. The values shown in Table 3.2 are examples only, and many agencies have developed their own trigger and limit values for their pavement structures.

The interpretation of NDT of concrete pavements can be used in a number of ways, including 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 evaluation process is the preparation of the field evaluation report, which summarizes the results of the data collection and analyses. In addition, critical nonpavement factors that could impact the selection of treatment alternatives should be identified as part of the report; this could include such items as shoulder condition, ditches, right of way, geometrics, curves, bridges, ramps, and traffic patterns.

5. Pavement Distress and Drainage Surveys

Section 4 provided an overview of the pavement evaluation process, and the remaining sections of this chapter describe the specific field testing components of that evaluation process, namely pavement distress and drainage surveys, NDT, surface characteristics (noise, roughness, surface friction, texture), and material 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 the pavement to receive preservation treatments. These 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 exhibiting similar performance.
- Gain insight into causes of deterioration (e.g., structural vs. functional).
- Identify additional testing needs.
- Identify potential treatment alternatives.
- Identify repair areas and repair quantities.

Pavement distress is any visible defect or form of deterioration on the surface of a pavement, and it is the most basic measure of the performance of an existing pavement. In order 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 will require more extreme rehabilitation measures.
- **Extent**—The quantity and severity level of each type must be measured and recorded.

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

Because excess moisture in the pavement structure contributes to the development of many pavement distresses, it is recommended that a drainage survey be conducted if moisture is suspected to be the cause of distress. In a drainage survey, visible signs of poor drainage are noted and can be coupled with information from materials sampling testing and NDT to provide some insight into the role that moisture is playing in the performance of the pavement.

The remainder of this section presents many of the important details associated with conducting distress and drainage surveys. The first section discusses the importance of using a distress identification manual to standardize the way distresses are interpreted by the pavement condition raters. Next, separate sections are used to present the guidelines associated with conducting distress and drainage surveys, respectively. Finally, guidance is provided on how to interpret the results of the collected distress and drainage data.

a. Joint faulting

b. Transverse cracking

c. Joint spalling

d. Corner break

e. Longitudinal cracking

f. Joint seal damage

g. ASR distress

h. D-cracking

Figure 3.1. Common concrete pavement distress types

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 surveys. In recent years, significant progress has been made in the standardization of distress survey procedures. Most state highway agencies have developed their own protocols or adopted various AASHTO standards for assessing the condition of their pavement structures.

In the FHWA's long-term pavement performance (LTPP) program, a detailed distress survey procedure and standardized distress definitions are available (Miller and Bellinger 2003). That document 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 that manual and also notes whether the distresses are primarily traffic related or climate/materials related. Because this manual was developed for the LTPP program, the manual is more research oriented and consequently requires that the pavement distress data be collected in considerable detail and at relatively high levels of precision.

Another common pavement distress survey procedure is the pavement condition index (PCI) procedure as defined in ASTM D6433-11, *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 the pavement's structural integrity and its surface operational condition based on the observed distress. The resulting index, the PCI, ranges from 0 (failed pavement) to 100 (perfect pavement) and accounts for the

Table 3.3. Concrete Distress Types Defined in LTPP Distress Identification Manual (Miller and Bellinger 2003)

Distress Type	Unit of Measure	Severity Levels?	Primarily Traffic/Load	Primarily Climate/Materials
Cracking				
Corner Breaks	Number	Yes	X	
Durability Cracking	Number of Slabs, Sq. Meters	Yes		X
Longitudinal Cracking	Meters	Yes	X	X
Transverse Cracking	Number, Meters	Yes	X	X
Joint Deficiencies				
Transverse Joint Seal Damage	Number	Yes		X
Longitudinal Joint Seal Damage	Number	No		X
Spalling of Longitudinal Joints	Meters	Yes		X
Spalling of Transverse Joints	Number, Meters	Yes		X
Surface Defects				
Map Cracking	Number, Sq. Meters	No		X
Scaling	Number, Sq. Meters	No		X
Polished Aggregate	Square Meters	No	X	
Popouts	Number/Sq. Meter	No		X
Miscellaneous Distresses				
Blowups	Number	No		X
Transverse Const. Joint Deterioration	Number	Yes		X
Faulting of Transverse Joints/Cracks	Millimeters	No	X	
Lane-to-Shoulder Dropoff	Millimeters	No		X
Lane-to-Shoulder Separation	Millimeters	No		X
Patch/Patch Deterioration	Number, Sq. Meters	Yes	X	
Punchouts	Number	Yes	X	
Water Bleeding and Pumping	Number, Meters	No	X	

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 it 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 the pavement 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 R36, *Standard Practice for Evaluating Faulting of Concrete Pavements*, which provides a method for fault measurements at highway speeds.
- AASHTO PP68, *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.

Guidelines for Conducting Condition Distress Surveys

Although modern technology has made automated distress data collection a more feasible alternative at the network level, manual distress surveys are often preferred 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 profiles (for determining roughness and faulting) at posted speed limits. If an automated survey is conducted, the level of detail should be sufficient to quantify pavement condition necessary for preservation treatment type selection. 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:

- Hand odometer (measuring wheel) for measuring distances.
- Stringline or straightedge for measuring rut depth and/or dropoff.
- Small scale or ruler for fine measurements.
- Marking paint or lumber crayon to mark distresses or record stationing along project.
- Mid- to full-sized vehicle.
- Faultmeter or other means for measuring joint/crack faulting and lane-shoulder dropoff.
- Notebook computer or tablet (or data collection forms or sheets) for recording distress.
- Agency-approved distress-identification manual.
- Camera or videotape for capturing representative distresses and conditions.
- Hard hats and safety vests.
- Traffic control provisions.

Elements of distress surveys are described in the following sections.

Presurvey Activities

Prior to any fieldwork, a preliminary records review should be conducted on the project. This should include information needed to assist in the conduct of the field surveys, such as general location information, general structural design information (pavement type, layer thicknesses, subgrade type, and so on), traffic information, data from any previous distress surveys, and location of utilities (especially if destructive testing will be conducted). Ideally, 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 of the work can be performed from the shoulder, the pavement surveyor 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 complete access to the pavement. In the case of higher-volume roadways, road closures are generally limited to nighttime closures or not at all, depending on traffic patterns. In these instances, assessment of pavement condition may be limited to surveys conducted using high-speed vehicles or 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 in both directions at posted speed limits to get 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 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.

The manual survey data collection forms that are used to record the distresses can be easily developed to fit an agency's objectives for distress surveys. These should be developed with the intended use of the data in mind in order to minimize future work. In addition, it is generally recommended that mapping of the project be conducted in order to help identify critical repair areas. An example field survey form is provided in Figure 3.2, and it shows how distresses are mapped and identified using predefined codes (Miller and Bellinger 2003).

More and more agencies are using portable, handheld computers or tablets to aid in the collection of distress data. Users can input distress quantities and amounts directly into the computer, which can then be downloaded for further evaluation. These 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 with data collection forms, but the time is made up during data processing.

At the conclusion of the manual distress survey, it is recommended that a complete photo or video summary of the project be performed (Van Dam et al. 2002a). 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.

Automated Distress Survey

Although developed primarily for network-level condition surveys, distress surveys employing automated methods can be used for evaluating project-level distress and condition. As described previously, automated surveys are typically conducted using vans equipped with specialized data and video collection equipment. Pavement condition related to ride and faulting are typically collected using noncontact sensor equipment, whereas surface distress is typically collected using high-speed cameras. 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 quantify the presence of 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.

In addition to obtaining surface condition information, automated survey vehicles can be outfitted to obtain right-of-way images, grade and cross-slope information, global position system (GPS) coordinates, and 3-D images using Light Detecting and Ranging (LIDAR) technology.

The majority of automated survey vehicle manufacturers provide manuals and guidelines for conducting pavement condition data collection. In addition, AASHTO PP68-10, *Standard Practice for Collecting Images of Pavement Surfaces for Distress Detection*, provides guidance for collecting images associated with automated methods.

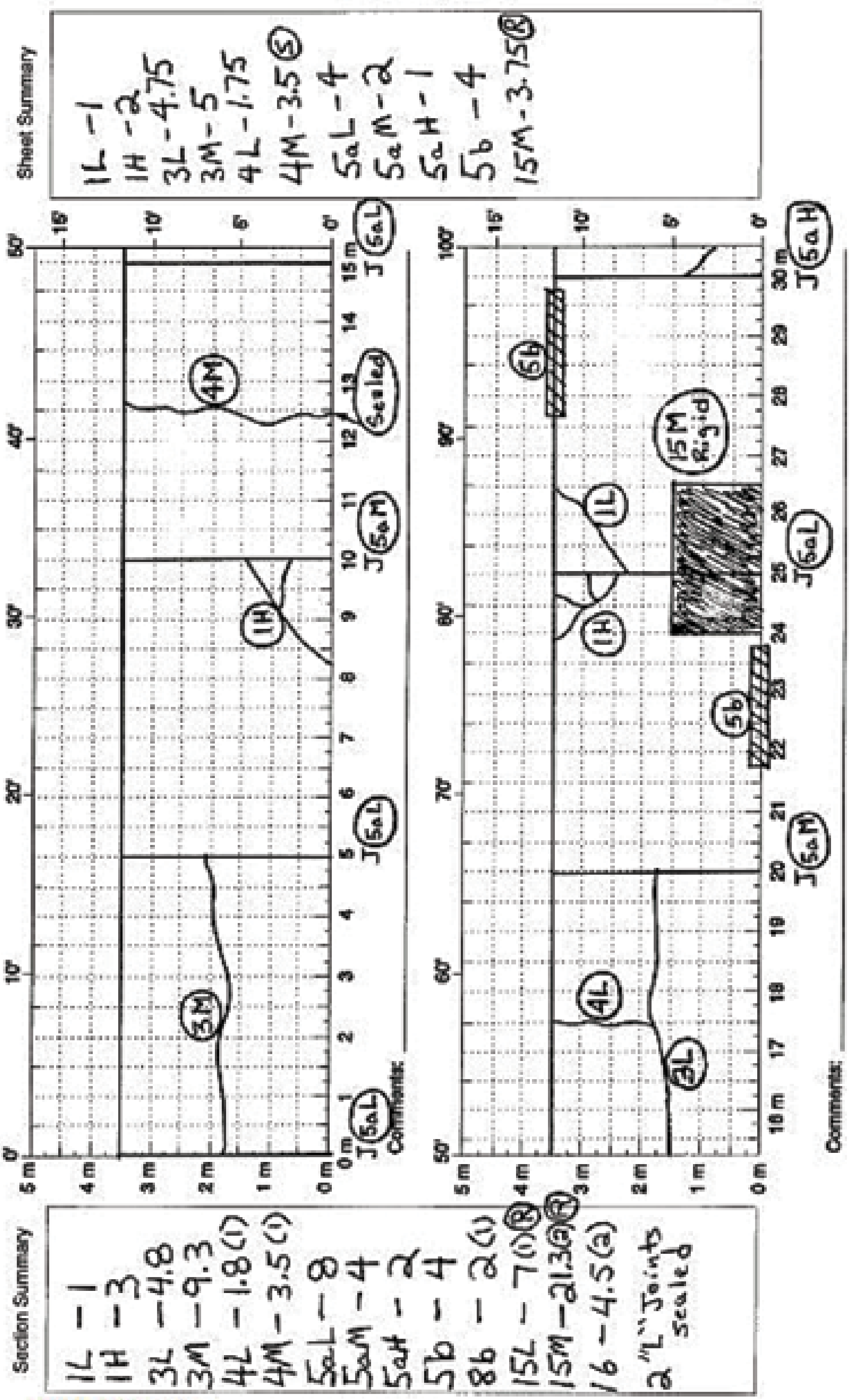


Figure 3.2. Example of LTPP field data collection form (Miller and Bellinger 2003)

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 to accomplish the following:

- Identify the presence of moisture-related distresses.
- Document the prevailing drainage conditions of the pavement (e.g., cross slopes, cut/fill areas, depth and condition of ditches).
- 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. A computer program called DRIP (Drainage Requirements In Pavements) is available from FHWA that can assist in such an analysis (Mallela et al. 2002).

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 noted 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 2 percent cross slope for mainline pavements and a 3 percent cross slope for shoulders (Anderson et al. 1998).
- **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, 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 1.2 m (4 ft) below the surface of the pavement, be at least 1 m (3 ft) wide, and have an absolute minimum longitudinal slope of 1.5 to 2.0 percent in urban areas and 1.0 percent in rural areas.
- **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 ditchline. 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 highway agencies have adopted video edgedrain inspections as part of new pavement construction.

All of the information collected from the drainage surveys should be marked and noted on strip maps 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.

While it is beyond the scope of this course, there are established procedures that can be used to analyze a pavement system and estimate the time it takes to drain water from the pavement to a prescribed level of saturation. The DRIP computer program, mentioned previously, can be used to conduct a detailed drainage analysis of a given project (Mallela et al. 2002).

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, noise, and surface friction could also be included. And 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 strip chart is shown in Figure 3.3. This figure plots the severity of concrete slab cracking along the length of the project. Three different slab cracking "conditions" are noted 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.

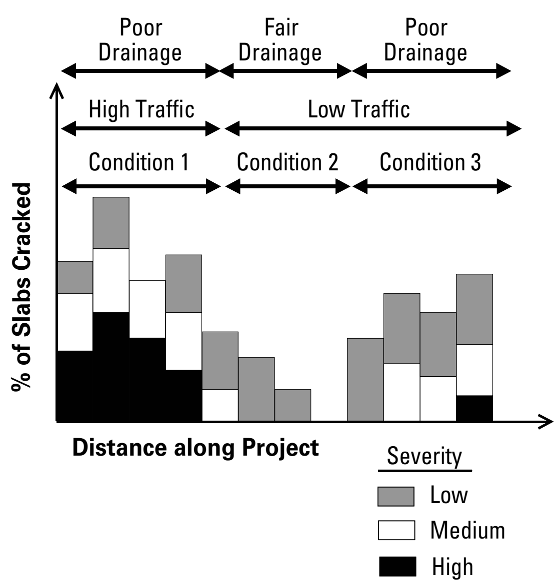


Figure 3.3. Example of project strip chart

A complete summary of the uses of 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 conduct of the field survey and the construction, a follow-up survey may be needed to ensure that contract quantities are still valid.
- An overall examination of the data along the project will reveal if 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.
- 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 data provide valuable insight into the mechanisms of pavement deterioration and, consequently, the type of treatment alternative 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.

6. Nondestructive Testing

A number of 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, to determine layer thicknesses, to establish the location of reinforcing steel, and to 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.

Table 3.4. Overview of Selected NDT Technologies

NDT Device	Image	Measurement Capabilities				
		Load Transfer Efficiency (LTE)	Depth to Steel	Layer Thickness	Void Detection	Structural Assessment
FWD		Yes	No	No	Yes	Yes
GPR		No	Yes	Yes	Yes	No
MIRA		No	Yes	Yes	No	Yes
MIT Scan 2		No	Yes	No	No	No
MIT Scan T2		No	No	Yes	No	No

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.

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 a project and for assessing the structural condition of the pavement.
- Load transfer efficiency 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, there are a number of different commercially available deflection testing devices. The most common deflection-measuring device, however, is the FWD. As shown schematically in Figure 3.4, 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. A series of sensors is located at fixed distances from the load plate, so that a 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 13 to more than 222 kN (3,000 to more than 50,000 lbf) can be applied, depending on the equipment. Figure 3.5 shows a photo of FWD testing on a concrete pavement with a view of the sensor bar.

Figure 3.5. Example of FWD showing sensor bar

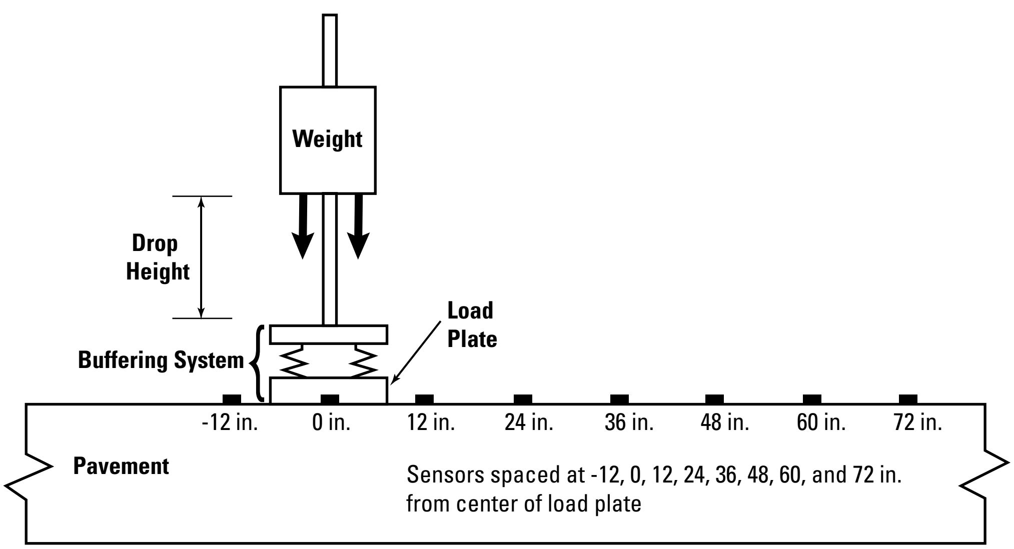


Figure 3.4. Schematic of FWD device

In the last decade, 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.
- 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.

Currently, three devices are available for the continuous collection of deflection data: the rolling wheel deflectometer (RWD), the traffic speed deflectometer (TSD), and the rolling dynamic deflectometer (RDD); see Figure 3.6. To date, these have seen greater applicability on asphalt-surfaced pavements than on concrete pavements. More detailed information on these devices is available elsewhere (Bay and Stokoe 1998; Grogg and Hall 2004; Flintsch et al. 2013).

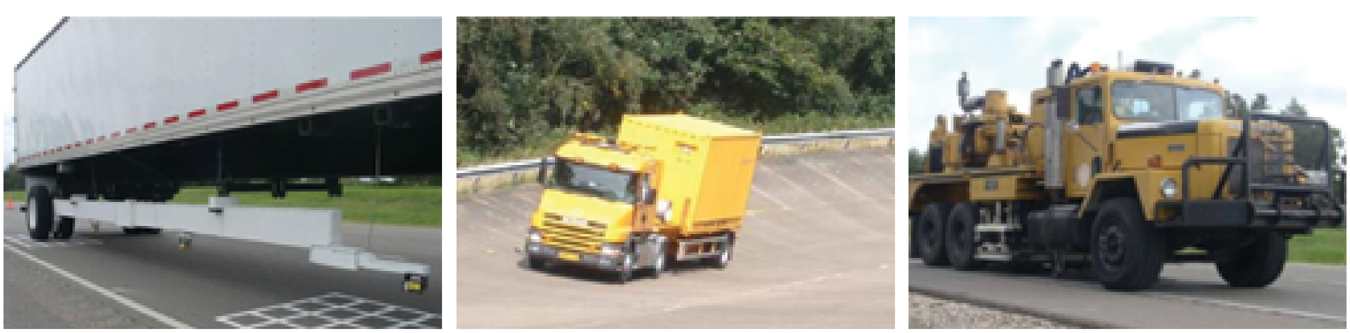
Factors That Influence Measured Deflections

There are a number of factors that 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:

- **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**—Pavement temperatures (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 21°C (70°F) and, ideally, when the pavement is not curled due to temperature 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.

These factors should be considered when developing an appropriate deflection testing program for an existing pavement structure.

Details of the aspects and features of a concrete pavement deflection testing program are beyond the scope



a. Rolling wheel deflectometer b. Traffic speed deflectometer c. Rolling dynamic deflectometer

Figure 3.6. Continuous deflection measurement equipment (Flintsch et al. 2013; ERES 2004)

of this document but are available elsewhere (FHWA 2006a; Pierce et al. 2010). In addition, the following standards and guides are available from AASHTO and ASTM:

- AASHTO T256, *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 3.7 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. This information is very helpful in identifying subsections within the project and also for indicating locations where distress, poor moisture conditions, cut/fill, and other conditions may be adversely affecting the pavement.

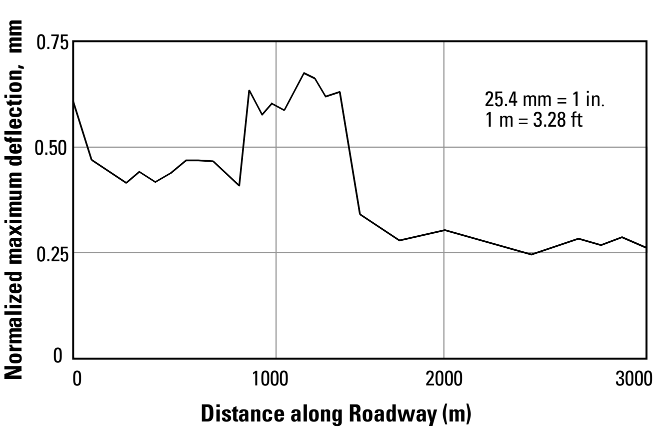


Figure 3.7. Illustration of deflection variation along a project

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 estimate remaining life. While the details of the procedures used to compute these parameters are outside the scope of this course, more detailed information on the backcalculation methods for concrete pavements are contained in published reports by AASHTO (1993); Khazanovich, Tayabji, and Darter (2001); and Pierce et al. (2010).

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 a number of ways, it is commonly expressed in terms of the deflections measured at the joint or crack:

$$LTE = \frac{\delta_U}{\delta_L} \cdot 100\% \tag{3.1}$$

where:

LTE	=	Load transfer efficiency, percent
δ_U	=	Deflection on unloaded side of joint or crack, mm (mils)
δ_L	=	Deflection on loaded side of joint or crack, mm (mils)

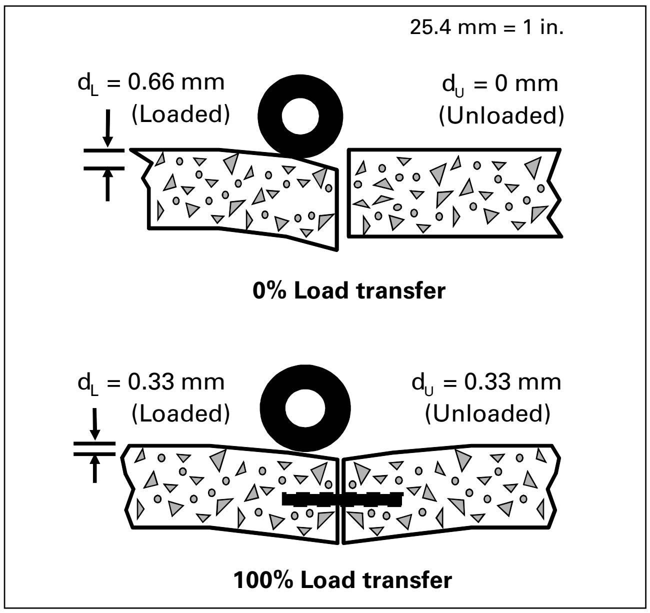
Figure 3.8a provides a schematic illustration of the concept of deflection load transfer, whereas Figure 3.8b shows the FWD device measuring LTE in the field. It should be noted that different LTE values may be obtained depending on which side of the joint is loaded, so it is generally recommended that both sides of the joint be load tested and the lowest value used. Furthermore, temperatures will significantly affect the LTE results, and it is generally recommended that load transfer testing be conducted at temperatures below 21°C (70°F).

The following general guidelines may be used to interpret LTE results (NCHRP 2004):

- Excellent: 90 to 100 percent.
- Good: 75 to 89 percent.

- Fair: 50 percent to 74 percent.
- Poor: 25 to 49 percent.
- Very Poor: 0 to 24 percent.

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, which can still cause pumping, faulting, and corner breaks. Generally speaking, it is desirable that peak corner deflections be less than 0.63 mm (25 mils) and that the difference in deflections across a load or crack be limited to 0.13 mm (5 mils) or less (Odden, Snyder, and Schultz 2003; Snyder 2011).



a. Deflection load transfer concept

b. Measuring deflection load transfer

Figure 3.8. Deflection load transfer

Identification of Locations of 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 26, 40, and 53 kN [6, 9, and 12 kips]) is dropped on both the approach and leave sides of a transverse joint, and a load-versus-deflection plot is generated, as shown in Figure 3.9. 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 0.05 mm (2 mils), whereas lines projecting more than that distance from the origin suggest the presence of a void. In Figure 3.9, the results suggest that there is a void beneath the leave side of the joint. The 1993 AASHTO Guide (AASHTO 1993) provides a summary of the available procedures for using the FWD to determine loss of support beneath concrete pavements.

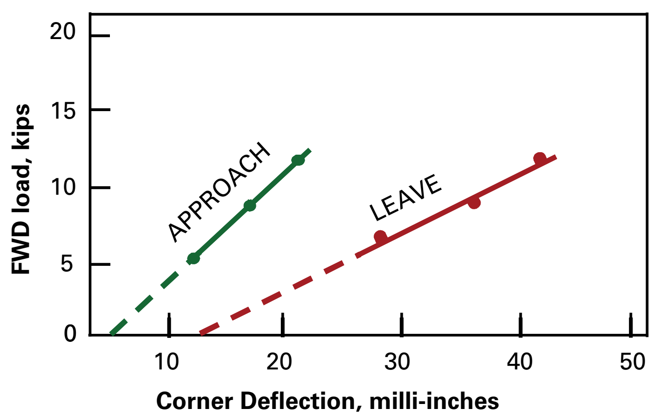


Figure 3.9. Example of void detection plot using FWD data

Ground-Penetrating Radar

Ground-penetrating radar utilizes radar pulses to locate pavement layers, embedded steel, and the presence of underlying voids. Ground-penetrating radar 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 dialectic constants of the pavement layers are not significantly different, cores may be required to aid with interpretation of the data. Ground-penetrating radar estimated layer thicknesses generally are within 3 to 15 percent of core measured thicknesses; see Table 3.5.

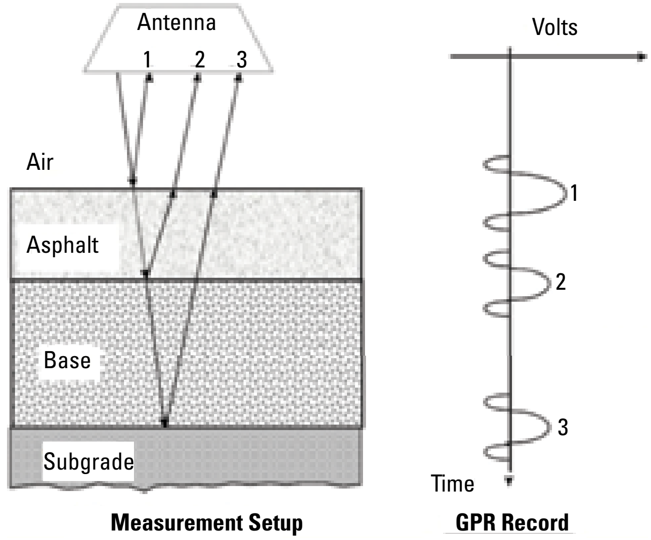


Figure 3.10. GPR principles (Maser 2010)

Table 3.5. GPR Accuracy by Pavement Type (Maser 1996, 2000)

Layer Type	Accuracy (vs. Cores)
New Asphalt	3–5 percent
Existing Asphalt	5–10 percent
Concrete	5–10 percent[a]
Granular Base	8–15 percent[a]

[a]Requires adequate contrast between layer materials

GPR Principles

Ground-penetrating radar technology uses radio wave pulses that are emitted, reflected, and recorded at each testing location; see Figure 3.10. 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 increases; and as the amplitude of the reflected wave pulse increases, the layer moisture content increases (Scullion, Chen, and Lau 1995). For thickness determination, as long as the dialectic constants of the paving layers are different, layer thickness can be readily determined using GPR testing. Since the dialectic constants of concrete and granular base materials may not be significantly different, however, the interpretation of layer thicknesses for this type of pavement structure may be difficult (Maser 1996). Still, because the wave pulses completely reflect metal, the location of embedded steel is easy to detect through GPR testing.

GPR 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. Figure 3.11 depicts different types of GPR equipment.

Applicable AASHTO and ASTM procedures for GPR testing are provided below:

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

a. Rear air-coupled unit

b. Front air-coupled unit

c. Ground-coupled unit

Figure 3.11. Examples of GPR equipment (Maser 2010)

GPR Interpretation

Analysis and review of GPR results requires 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). Figure 3.12 illustrates an example output from a GPR scan of a CRCP. The GPR scan shows the location of the reinforcing steel, the bottom of the CRCP, as well as the location and extent of an underlying void.

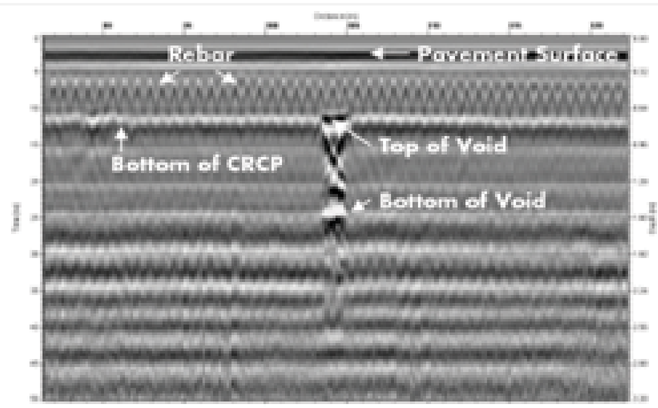


Figure 3.12. Example GPR scan of CRCP (Chen 2010)

Magnetic Imaging Tomography

Magnetic imaging tomography technology can be used to determine concrete pavement layer thicknesses and to evaluate 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 Scan-2 is used for determining dowel bar location, while the MIT Scan T2 is used for determining the thickness of freshly placed concrete. Although the MIT technology is beneficial for new concrete pavement construction, this technology does have a few preservation treatment applications. For example, the MIT Scan-2 device can be used for locating existing dowel bars and tiebars prior to conducting full- and partial-depth spall repairs, and it can also be used for determining steel location prior to dowel bar retrofit, cross stitching, and slot stitching.

Each of these devices is further described in the following sections.

MIT Scan 2

The MIT Scan 2 device, shown in Figure 3.13, is used in the evaluation of dowel bar placement. It is able to measure vertical and horizontal misalignments within the following limits (Yu and Khazanovich 2005):

- Dowel bar depth—100 to 190 mm (3.9 to 7.5 in.).
- Dowel bar side shift—+100 mm (+4 in.).
- Dowel bar horizontal misalignment—+40 mm (+1.6 in.) plus a uniform rotation of +80 mm (+3.1 in.).
- Dowel bar vertical misalignment—+40 mm (+1.6 in.).

Figure 3.13. MIT Scan 2 device (Yu and Khazanovich 2005)

A two-person crew can measure dowel bar placement on 200 or more joints in an 8-hour shift using the MIT Scan 2 device (Yu and Khazanovich 2005). Although the results are not influenced by weather conditions, the operating temperature of the MIT Scan 2 device is -5°C–50°C (23°F–122°F). Data analysis can be conducted in real time or stored on a memory card for more detailed analysis using sophisticated software.

Figure 3.14 illustrates an example output of the field report from the MIT Scan 2 device. 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.), the dowel bar distance from the beginning of the test (Bar Loc.), the dowel bar spacing (Bar Spc.), the dowel bar depth (Depth), side shift, and horizontal (Hor.) and vertical (Vert.) alignment.

The results of the MIT Scan 2 can be used to evaluate the dowel alignment in relation to contract specifications or as part of a forensic investigation. Additional information on guidelines for specifying dowel bar alignment can be found in Khazanovich, Hoegh, and Snyder (2009).

Highway : I20
Station No.: 0+31
Bar Spacing : 300 mm
Concrete Thickness : 300 mm
Bar type : 456 x 32.4 mm

Bar No.	Bar Loc. mm	Bar Spc. mm	Depth mm	Side Shift mm	Alignment Hor. mm	Alignment Vert. 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	---	143	-7	3	1

Figure 3.14. MIT Scan 2 field report (Yu and Khazanovich 2005)

MIT Scan T2

The MIT Scan T2 device, shown in Figure 3.15, is used to determine the thickness of a freshly placed concrete pavement. It uses technology similar to that used for the MIT Scan-2 device, but it requires the placement of a metal reflector prior to paving as part of the testing process; see Figure 3.16. The MIT Scan T2 device is used to test at the location of the metal reflector in order to estimate the slab thickness. The MIT Scan T2 device is able to measure concrete thickness up to 510 mm (20 in.) and is reported to be accurate within a 0.5-percent tolerance when compared to core measurements (Ye and Tayabji 2009). The results of the MIT Scan T2 are immediately displayed on the device readout screen or can be downloaded to the device software for analysis.

Figure 3.15. MIT Scan T2 device (Ye and Tayabji 2009)

Figure 3.16. Placement of metal reflector prior to paving (Ye and Tayabji 2009)

Ultrasonic Tomography

The MIRA device, shown in Figure 3.17, is based on impact-echo technology and features a linear array of transducers that allows for 45 transmitting and receiving pair measurements in each scan. The results can be used for determining concrete layer thicknesses and relative concrete strengths, identifying cracking in the concrete layer, identifying areas of debonding between concrete layers, locating embedded steel, and identifying areas of joint deterioration and poor consolidation (Hoegh, Khazanovich, and Yu 2011; NCPTC 2013).

Figure 3.17. MIRA device (NCPTC 2013)

7. 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. Three easily measurable characteristics that give an indication of a pavement's functional condition are roughness, noise, and surface friction. Excessive roughness can create user discomfort and irritation and can lead to increased vehicle operating costs, user delay, and crashes. Excessive noise can be disruptive to the traveling public and adjacent property owners. Inadequate surface friction can also contribute to crashes, especially under wet weather conditions.

Definitions

This section defines a number of important roughness-, noise-, and friction-related terms. For convenience, these definitions are presented in alphabetical order.

- **Noise Levels**—This sound level based on a logarithmic scale is expressed in terms of decibels (dB). For traffic noise measurements, the sound level is adjusted to the human ear and is referred to as the A-weighting scale (dBA). The A-weighting scale ranges from a low of 0 dBA (inaudible) to a high of 140 dBA (threshold of pain). The human ear, in general, is only able to distinguish a 3-dBA change of a similar sound (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 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 (AASHTO 2008b). 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, condensation], and snow and ice) (AASHTO 2008b).
- **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 1 µm–0.5 mm (0.00004–0.02 in.) with a vertical amplitude range of 1 µm–0.2 mm (0.00004–0.008 in.). Microtexture is the surface "roughness" of the individual coarse aggregate particles and of the binder, and it contributes to friction through adhesion with vehicle tires. For concrete surfaces constructed for speeds under 80 km/h (50 mi/h), microtexture is usually all that is needed to provide adequate stopping in wet weather conditions (Hibbs and Larson 1996).
    - Macrotexture—wavelengths of 0.5 mm–50 mm (0.02–2 in.) with a vertical amplitude range of 0.1 mm–20 mm (0.004–0.8 in.). Macrotexture refers to the overall texture of the pavement, which in concrete pavements is controlled by the surface finish (tining). For concrete pavements constructed for speeds greater than or equal to 80 km/h (50 mi/hr), good macrotexture is needed to reduce the water film thickness and prevent hydroplaning (Hibbs and Larson 1996). The difference between microtexture and macrotexture (and the relative different degrees of each) is illustrated in Figure 3.18.
    - Megatexture—wavelengths of 50 mm–500 mm (2–20 in.), with a vertical amplitude range of 0.1 mm–50 mm (0.004–2 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, Harrington, and Ferragut 2006). At slower speeds, the powertrain noise is the predominant source of highway 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 Figure 3.19.
- **Tire-Pavement Noise**—This is noise attributed to the interaction of the tire-pavement interface as well as vehicle vibration and aerodynamic noise (Cackler, Harrington, and Ferragut 2006). At higher speeds, the tire-pavement noise is the primary source of roadside noise.

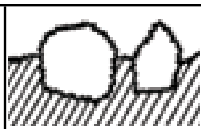
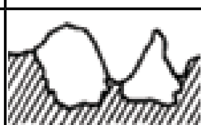
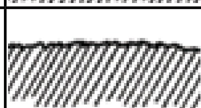
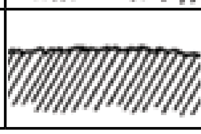
Surface		Scale of Texture: Macro (Large)	Scale of Texture: Micro (Fine)
A		Rough	Harsh
B		Rough	Polished
C		Smooth	Harsh
D		Smooth	Polished

Figure 3.18. Differences between microtexture and macrotexture (Shahin 1994)

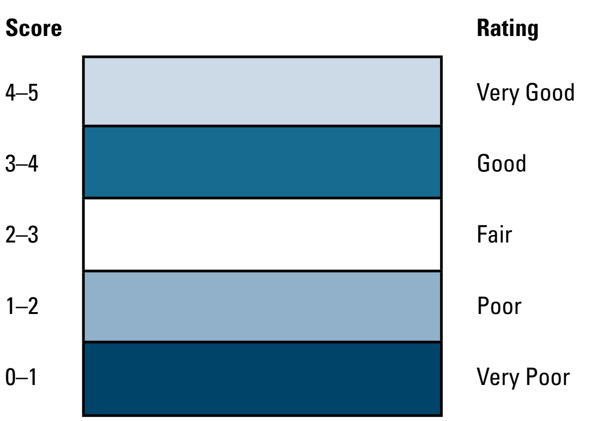


Figure 3.19. PSR rating scale

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 problematic to property and business owners adjacent to roadway facilities, as well as to the traveling public. Methods of measuring tire-pavement noise and suggested remedial measures are provided in this section, with more detailed discussions provided elsewhere (Rasmussen et al. 2010).

Measuring Pavement Noise

Although a number of different methods have been used for measuring pavement noise, the primary method used today is AASHTO TP76, *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 3.20. Additional details on measuring and reporting tire-pavement noise using OBSI are provided by Rasmussen, Sohaney, and Wiegand (2011).

Figure 3.20. OBSI testing configuration (Rasmussen, Sohaney, and Wiegand 2011)

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 measured at 96 km/h (60 mi/hr), is an A-weighted overall sound intensity level between 101 and 102 dB (Rasmussen et al. 2012). Figure 3.21 illustrates the OBSI measurements from a variety of concrete textures and indicates that diamond grinding and longitudinal tining produce some of the quieter concrete textures.

There are a number of 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 25 mm (1 in.) or greater.
  - Avoid extremely smooth surfaces.
  - Texture should point down (e.g., grooves) rather than up (e.g., fins).
  - When possible, grooves should be oriented in the longitudinal direction.
  - Transverse grooves should be closely spaced and randomized whenever possible.
- Concrete properties.
  - Surface mortar should be consistently strong, durable, and wear resistant.
  - A consistent, dense mixture should be used.
  - Siliceous sands should be used 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, there should be an adequate and consistent depth of mortar near the surface to hold the intended geometry.
- Transverse joints.
  - Narrow, single-cut joints are preferred over widened (reservoir) cuts.
  - Avoid excess joint sealant and joint sealant that protrudes above the pavement surface.
- Paving equipment.
  - Minimize vibrations.
  - There should be smooth and consist paver operation.
  - Maintain a constant head of uniform concrete at the proper level.
- Texture/cure equipment.
  - Minimize vibrations.
  - Minimize the buildup of latency on the tining equipment.

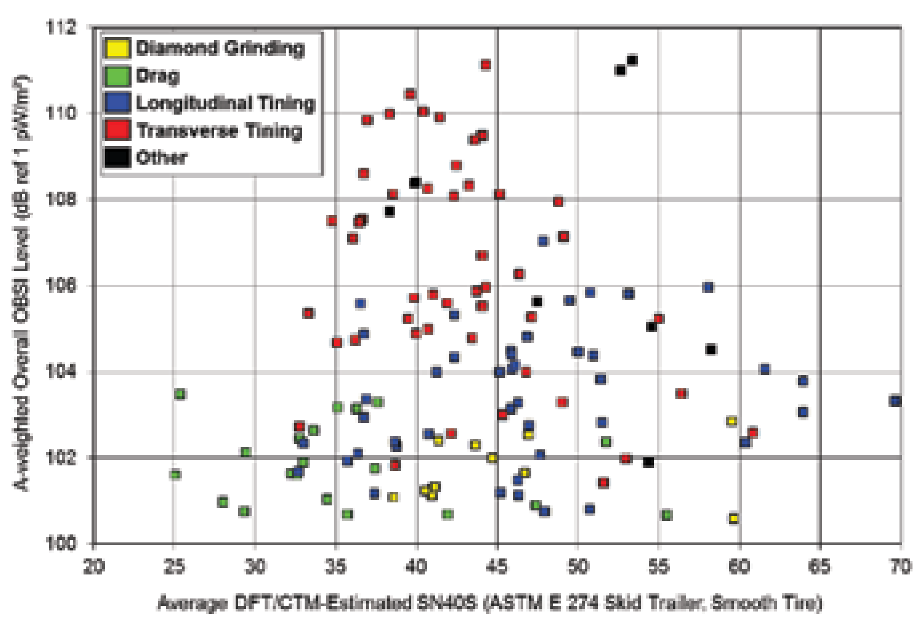


Figure 3.21. Noise and friction for various concrete pavement textures (Rasmussen et al. 2012)

- ◦ Ensure consistent tracking of texture equipment.
  - ◦ Provide a multiple pass (or higher concentration) of the curing application.
- Grinding equipment.
  - ◦ There does not appear to be an optimum size and 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 wheelpath.
  - ◦ 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) or objectively (with roughness-measuring equipment). The primary purpose of the survey is to identify areas of severe roughness on a given project, as well as to provide some insight into its cause. 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. 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, 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, spalling] versus roughness due to differential elevations [e.g., swells and depressions]).

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. A range of IRPS measurement equipment is available, such as the noncontact lightweight profilers seen in Figure 3.22 that are commonly used on new pavement and overlay projects, the portable laser profilers seen in Figure 3.23, and the

Figure 3.22. Noncontact lightweight profiler

Figure 3.23. Examples of portable laser profiler (SSI 2013, ICC 2013, Ames 2013)

high-speed profilers seen in Figure 3.24 that are commonly used to monitor the roughness of a pavement network.

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. 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 strongly recommended for use when measuring roughness on textured concrete pavement surfaces.

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 0.16-km (0.1-mi) increment. Roughness equipment that only measures one wheelpath should measure the right wheelpath in the direction of traffic for the outer and inner lanes. Special efforts 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 provides applicable AASHTO standards for quantifying pavement roughness and profile measurements:

Figure 3.24. Example of high-speed profiler (photo courtesy of WSDOT)

- AASHTO M328, *Standard Specification for Inertial Profiler.*
- AASHTO R43, *Standard Practice for Quantifying Roughness of Pavements.*
- AASHTO R57, *Standard Practice for Operating Inertial Profiling System and Evaluating Pavement Profiles.*

In addition, the FHWA has developed a *Manual for Profile Measurements and Processing* (Perera, Kohn, and Rada 2008) that provides guidance on the calibration of laser profilers for use in its LTPP monitoring program.

A particular 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 cause significant variations in the measured pavement profile over the course of the day. These effects are more noticeable on short-jointed concrete pavements, and they will result in the highest level of roughness occurring in the early morning hours when the slabs are more likely to be curled up. Thus, for project-level profiling, 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.

PSR

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 at the 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. 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).

IRI

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 measured with any valid profiler (Sayers and Karamihas 1998). Furthermore, the IRI provides a common numeric scale of measuring roughness that can be correlated to roughness measurements obtained from both response-type and inertial-based profiler systems (Sayers 1990).

The IRI scale ranges from 0 m/km to 20 m/km (0 in. to 1267 in./mi), with larger values indicating greater roughness. The approximate break point between "rough" and "smooth" concrete pavements is often considered to be 2 m/km (125 in./mi). The FHWA has presented guidelines in which an "acceptable" ride quality for highway pavements is defined by an IRI range of 0–2.7 m/km (0–170 in./mi) (FHWA 2006b). The specific FHWA guidelines that relate IRI levels to condition and PSR are presented in Table 3.6. The IRI can be computed in accordance with AASHTO R43, *Standard Practice for Quantifying Roughness of Pavements*, or ASTM Standard E1296, *Standard Practice for Computing International Roughness Index of Roads from Longitudinal Profile Measurements*.

Table 3.6. Relationship between IRI and Condition (FHWA 2006b)

Ride Quality Terms*	All Functional Classifications	
	IRI Rating, in./mi (m/km)	PSR Rating
Good	<95 (1.5)	>3.5
Acceptable	<170 (2.7)	>2.5
Not Acceptable	>170 (2.7)	<2.5

*The threshold for "Acceptable" ride quality used in this report is the 170 in./mi (2.7 m/km) IRI value as set by the FHWA Performance Plan for the NHS. Some transportation agencies may use less stringent standards for lower functional classification highways to be classified as acceptable.

Surface Friction Testing

The importance of maintaining adequate pavement surface friction is evident as pavement safety continues to be a major concern of most highway agencies around the world. Based on 2011 data, there were more than 32,000 deaths and 2 million injuries in the United States, and another 3.7 million crashes that resulted in property damage only (NHTSA 2013). Previous research suggests that about 14 percent of all crashes occur in wet weather and that 70 percent of those crashes are preventable with improved pavement texture/friction (Larson, Scofield, and Sorenson 2005).

Two primary causes of wet weather crashes are (1) uncontrolled skidding due to inadequate surface friction in the presence of water (hydroplaning), and (2) poor visibility due to splash and spray (Snyder 2006). 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 expressed as a single number index (e.g., "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 are also important.

Types of Friction-Measuring Equipment

There are four basic types of 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 are described in more detail below.

Locked-Wheel Testers

Locked-wheel testing devices simulate emergency braking conditions for vehicles without antilock brakes (i.e., a 100-percent slip condition). Today, the majority of agencies in the United States measure pavement friction with an ASTM locked-wheel trailer in accordance with ASTM E274, *Standard Test Method for Skid Resistance of Paved Surfaces Using a Full Scale Tire* (Henry 2000). In this procedure, the locked-wheel trailer is towed on a pavement that has been wetted with a specified amount of water, and then a braking force is applied. Testing can be done with either a ribbed (treaded) or blank (smooth) tire, but measurements using 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 friction times 100. Skid numbers are reported in the form of SN(test speed [in mi/hr]) 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 km/h, 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 km/h (50 mi/hr), the skid number would be reported as SN(80)R or SN50R (metric and English units, respectively).

Side-Force Testers

Side-force testers are designed to simulate a vehicle's ability to maintain control in 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-percent 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 distance while the wheel is locked (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.

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 percent 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

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).

Friction Testing Procedures

The pavement friction should be measured at uniform increments along the project in each traffic lane. As a minimum, most state highway agencies test in the left wheelpath of the driving lane (under normal conditions, this is the location where the surface friction is minimum). The increments should be tied into the milepost markers so that intersections, interchanges, curves, and hills can be identified. Sharp curves are particularly important to consider.

Pavement Surface Texture

In recent years, it has been recognized that measuring pavement surface texture is necessary to accurately represent a pavement's true functional characteristics. As described previously, pavement texture is primarily divided into three categories: microtexture, macrotexture, and megatexture. While all three are known to influence the 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 circular track meter or circular texture meter (CTMeter). To provide adequate surface friction, the average mean texture depth (MTD) should be 0.8 mm (0.03 in.) with a minimum of 0.5 mm (0.02 in.) for any individual test (Hibbs and Larson 1996).

In the past decade, advances in laser technology and computational power have led to the development of systems that measure pavement longitudinal profile at traffic speeds (Henry 2000). Analysis of these data can be used to compute a mean profile depth (MPD), which can be used to estimate the more traditional

MTD measurement. The MPD is measured using modern high-speed vehicle-mounted laser-based measuring devices or with portable devices such as the CTMeter.

Evaluation of Roughness, Friction, and Texture Survey Results

Any collected 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 the 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 (corner breaks, durability, longitudinal, and transverse) 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 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.

8. 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 indications of MRD in the concrete, 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 material sampling and testing are primarily dependent upon the following factors:

- **Observed Pavement Distress**—The type, severity, extent, and variation of visible distress on 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 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 trafficked roadways because of worker and driver safety concerns. 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.

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

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 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) for strength testing and possible petrographic examination. A visual inspection of retrieved cores can also provide valuable information when trying to assess the causes of visual 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 50, 100, or 150 mm (2, 4, or 6 in.), the selection of which depends on the purpose. If thickness verification is all that is needed, 50-mm (2-in.) diameter cores are sufficient. Strength testing is most commonly conducted on 100-mm (4-in.) diameter cores; however, a 150-mm (6-in.) diameter core is recommended when the maximum aggregate size is greater than 38 mm (1.5 in.). Although 100-mm (4-in.) diameter cores can be used for petrographic testing, 150-mm (6-in.) diameter cores are often preferred.

If desired, material samples of subsurface layers (i.e., subgrade soil, subbase, and base) can be obtained from the core holes. Other specialized testing may also be conducted at these locations, such as split-spoon (split-barrel) sampling and Shelby (push) tubes. More details on all of these material-sampling methods are available elsewhere (Hoerner et al. 2001).

DCP

The 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). In the last decade, the DCP has gained widespread popularity, largely because it is fast, is easy to use, and 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). Figure 3.25 shows a schematic of the DCP apparatus (U.S. Army 1989). The test is performed by driving the cone into the pavement/subgrade by raising and dropping the 8-kg (16.7-lb) hammer from a fixed height of 57.5 cm (22.6 in.). Earlier versions of the DCP used a 30° cone angle with a diameter of 20 mm (0.8 in.) (Newcomb and Birgisson 1999). More recent versions of the DCP use a 60° cone angle and also have the option of using a 4.6-kg (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 device.

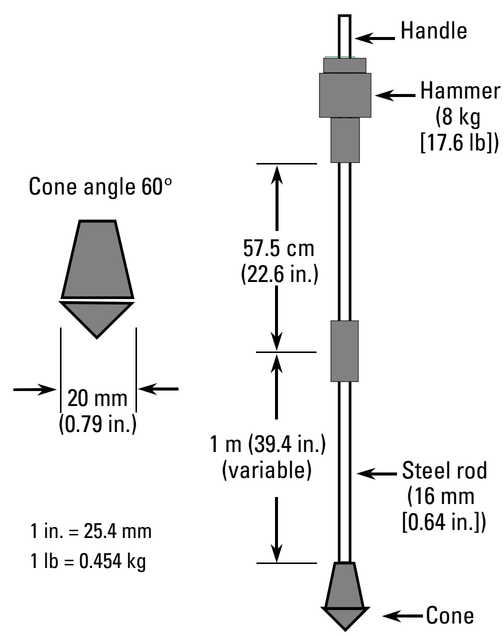


Figure 3.25. Dynamic cone penetrometer (U.S. Army 1989)

During a DCP test, the cone penetration (typically measured in millimeters or inches) associated with each drop is recorded. This procedure is completed 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 50 mm (2 in.), and midpoint plus 50 mm (2 in.). The DCP PRs can be used to identify pavement layer boundaries and subgrade strata, as well as to estimate the CBR values of those individual layers.

Results of the DCP 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 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 DPI and CBR is the following developed by Webster, Grau, and Williams (1992) for the manual DCP:

$$CBR = \frac{292}{DPI^{1.12}} \tag{3.2}$$

where:

CBR = California bearing ratio

DPI = DCP penetration index (measured in mm per blow)

Other research has provided variations to this equation that are applicable for heavy and lean clays (Webster, Brown, and Porter 1994). These correlations are illustrated in Figure 3.26.

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

$$CBR = 2.57 - 1.25 \times \log DPI \tag{3.3}$$

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 percent greater than CBR values computed using DPI from the manual DCP (Newcomb and Birgisson 1999).

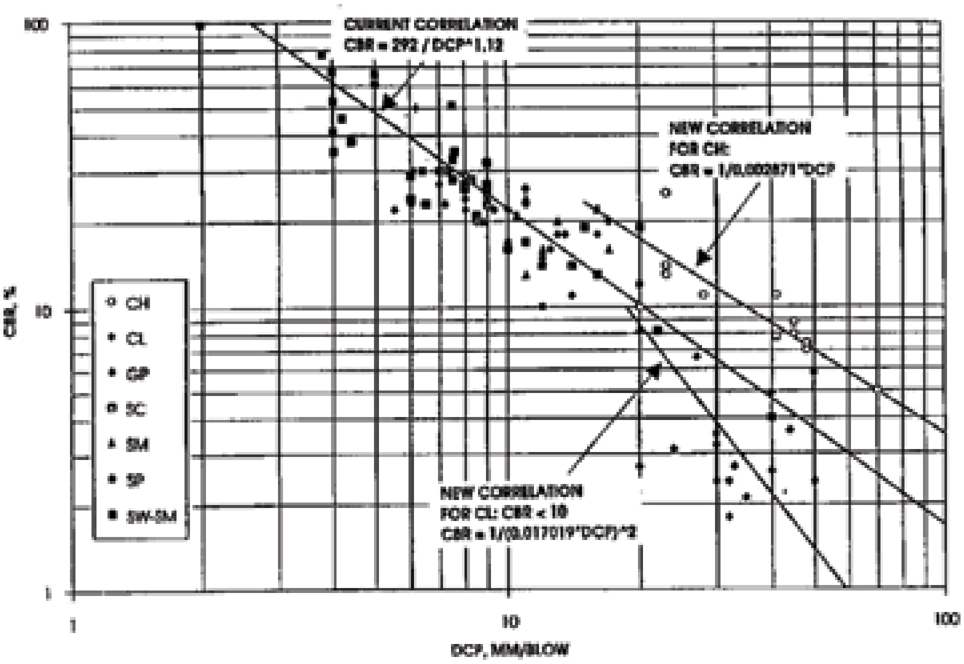


Figure 3.26. Correlations between DPI and CBR (Webster, Brown, and Porter 1994)

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 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. These tests are primarily run to show whether or not 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) in order 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 Figure 3.27 (PCA 1992).

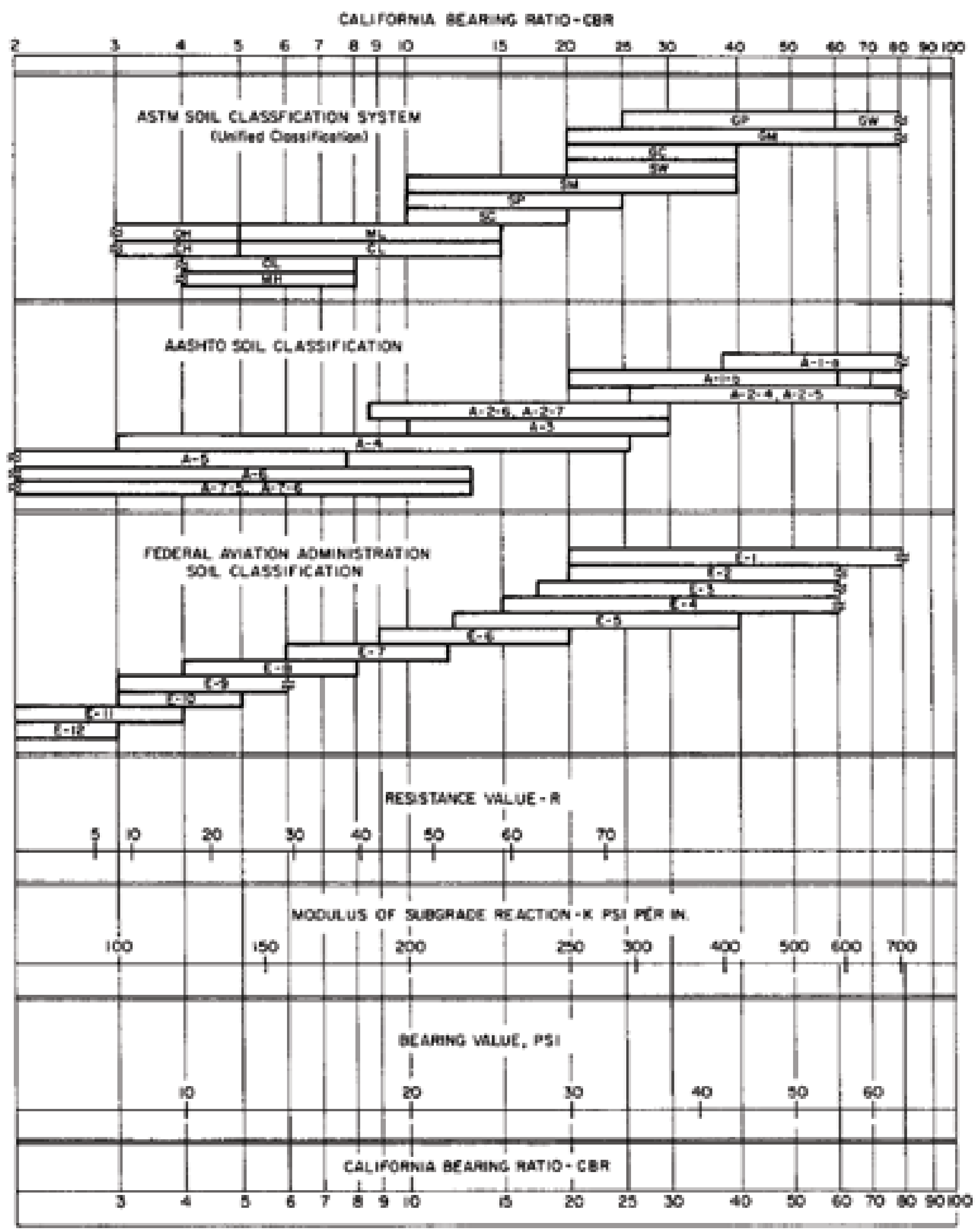


Figure 3.27. Approximate correlations between soil classification and subgrade soil parameters (PCA 1992)

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.

CBR

The CBR test measures the resistance of an unbound soil, base, or subbase sample to penetration by a piston with an end area of 1,935 mm^2 (3 in.^{2}) being pressed into the soil at a standard rate of 1.3 mm (0.05 in.) per minute. A schematic of the test and typical data are shown in Figure 3.28. 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 2.5 mm [0.1 in.] penetration), multiplied by 100, is the CBR of the soil. California bearing ratio values will typically range from 2 to 8 for silts and clays up to 50 to 70 (or more) for granular bases and high-quality crushed stones (PCA 1992).

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

Hveem Resistance Value

An Hveem Stabilometer measures the transmitted horizontal pressure associated with the application of a vertical load (PCA 1992). In accordance with AASHTO T246, *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 (100 mm [4 in.] in diameter and 6 mm [0.25 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. The R-value method has been used by several western state highway agencies, but it is an empirical test method and does not represent a fundamental soil property.

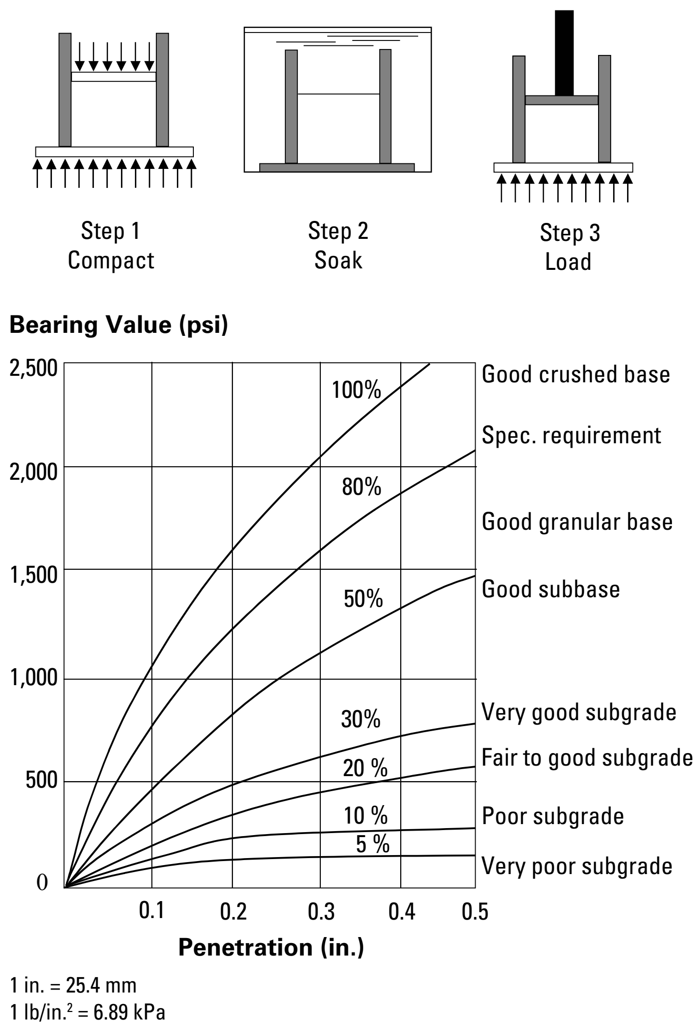


Figure 3.28. CBR testing procedures and load penetration curves for typical soils (Oglesby and Hicks 1982)

Triaxial Strength Testing

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. The test procedure is described in AASHTO T296, *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

The resilient modulus test provides a material parameter that more closely simulates the behavior of the material under a moving wheel. In the laboratory, the resilient modulus test is conducted by placing a compacted material specimen (ideally an undisturbed in situ sample; however, it may be necessary to recompact the sample) in the triaxial cell, as shown in Figure 3.29. The specimen is 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 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).

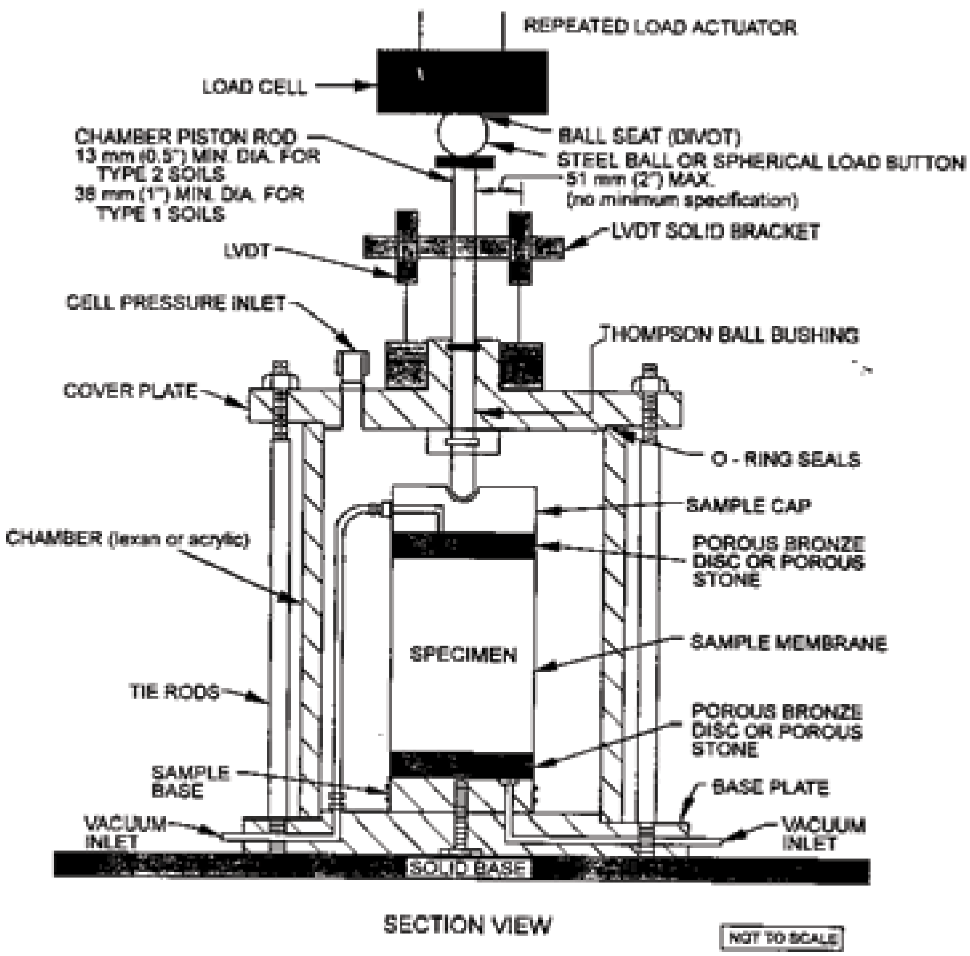


Figure 3.29. Subgrade resilient modulus test apparatus

Resilient modulus testing is performed on subgrade soils and on unbound base/subbase materials in accordance with AASHTO T307-99, *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 resilient modulus, CBR, and R-value are given below. These correlations, however, should be taken only as general indicators and therefore should be applied with extreme caution.

- Resilient Modulus vs. CBR:

$$M_R = B^*CBR \tag{3.4}$$

where:

M_R	=	Resilient modulus, lbf/in.^{2}
CBR	=	California bearing ratio
B	=	Coefficient = 750 – 3000 (1500 for CBR < 10)

- Resilient modulus vs. R-value:

$$M_R = A + B(R) \tag{3.5}$$

where:

M_R	=	Resilient modulus, lbf/in.^{2}
R	=	Resistance value obtained using the Hveem Stabilometer
A	=	Constant = 772 – 1155 (1000 for R < 20)
B	=	Constant = 369 – 555 (555 for R < 20)

Unconfined Compressive Strength

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 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, S*tandard 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.

Elastic Modulus Testing

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

The indirect tension test, also called the splitting tensile 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 (1.3 mm [0.05 in.] per minute) on the diameter of a cylindrical sample, as shown in Figure 3.30. The sample will fail in tension along the vertical diameter of the sample and the indirect tensile strength is calculated from the following equation:

$$\sigma_t = \frac{2P_{ult}}{\pi LD} \tag{3.6}$$

where:

σ_t	=	Indirect tensile strength, Pa (lbf/in^2)
P_{ult}	=	Vertical compressive force at failure, N (lbf)
L	=	Length of sample, m (in.)
D	=	Diameter of sample, m (in.)

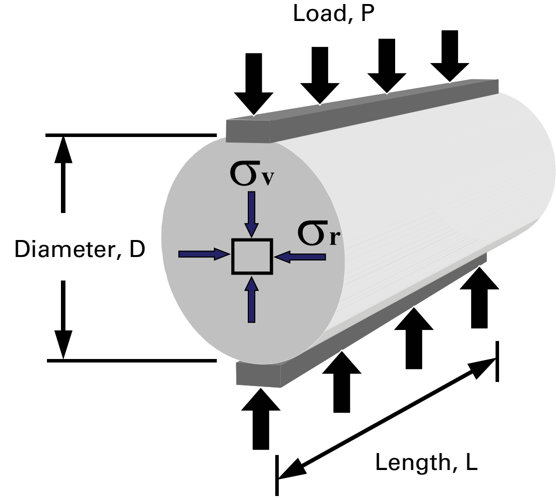


Figure 3.30. Indirect tension test (Mindess and Young 1981)

Ch 3. Pavement Evaluation

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. Materials-related distresses are those distresses that develop due to the concrete's inability to maintain its integrity (changes in concrete microtexture) when subjected to changes in physical (environmental) and chemical mechanisms. Materials-related distress 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.

The occurrence of MRD is a function of many factors, including the constituent materials (aggregate, cement, admixtures) and their proportions, the pavement's location (maritime or inland), the climatic conditions (temperature, 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. Table 3.7 summarizes details of the most common MRD types, including information regarding their causes, typical time of appearance, and prevention (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 concrete 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 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., water-to-cement ratio [w/c], chloride content).

Recent field studies have identified joint deterioration on some concrete pavements, generally located in the Midwest. This deterioration is thought to be the result of water and freeze-thaw action, but it can be exacerbated by improper joint detailing, poor construction practices, and marginal or poor-quality aggregates (Taylor 2009). A guide document for the identification, mitigation, and prevention of this distress is available (Taylor et al. 2011).

Alkali-silica reactivity is one particularly troublesome MRD that can produce severe performance problems in concrete pavements. As described in Table 3.7, ASR can lead to slab cracking, pressure-related distresses such as spalling and blowups, and damage to adjacent structures (bridges, abutments, utilities). In recognition of its potentially significant effects on pavement performance, the FHWA established a program in 2007 to further the development and deployment of techniques to prevent and mitigate ASR. Considerable research focused on the development of a performance-based prescriptive approach for designing concrete mixtures that will be resistant to ASR (Thomas, Fournier, and Folliard 2008); that work served as the basis for the AASHTO PP-65-11 protocol (*Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction*) that was developed in 2011. Efforts also looked at ways, however, 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), and a procedure for evaluating and managing ASR in existing pavements was released in 2010 (Fournier et al. 2010). The procedure involves three general steps: (1) condition survey; (2) preliminary investigation for diagnosis of ASR; and (3) detailed investigations for diagnosis/prognosis of ASR (Fournier et al. 2010). A flowchart for this process is depicted in Figure 3.31.

Table 3.7. Summary of Key MRDs (Van Dam et al. 2002a)

Type of MRD	Surface Distress Manifestations and Locations	Causes/ Mechanisms	Time of Appearance	Prevention or Reduction
MRD due to Physical Mechanisms				
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	Addition of 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	Provide minimum cement content of 335 kg/m^3, limit water-cement ratio to no more than 0.45, and provide a minimum 30-day “drying” period after curing before allowing the use of 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 of nonsusceptible aggregates or reduction in maximum coarse aggregate size
MRD due to Chemical Mechanisms				
Alkali–Silica Reactivity	Map cracking over entire slab area and accompanying expansion-related distresses (joint closure, spalling, blowups)	Reaction between alkalis in the pore solution and reactive silica in aggregate resulting in the formation of an expansive gel and the degradation of the aggregate particle	5–15 years	Use of nonsusceptible aggregates, addition of pozzolans to mix, limiting total alkalis in concrete, minimizing exposure to moisture, addition of lithium compounds
Alkali–Carbonate Reactivity (ACR)	Map cracking over entire slab area and accompanying pressure-related distresses (spalling, blowups)	Expansive reaction between alkalis in 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, 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, deicing chemicals) react with the calcium sulfoaluminates	1–5 years	Use w/c below 0.45, minimize tricalcium aluminate content in cement, use blended cements, 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 either 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, avoid high curing temperatures
Corrosion of Embedded Steel	Spalling, cracking, and deterioration at areas above or surrounding embedded steel	Chloride ions penetrate concrete, resulting in corrosion of embedded steel, which in turn results in expansion	3–10 years	Reduce the permeability of the concrete, provide adequate concrete cover, protect steel, or use corrosion inhibitor

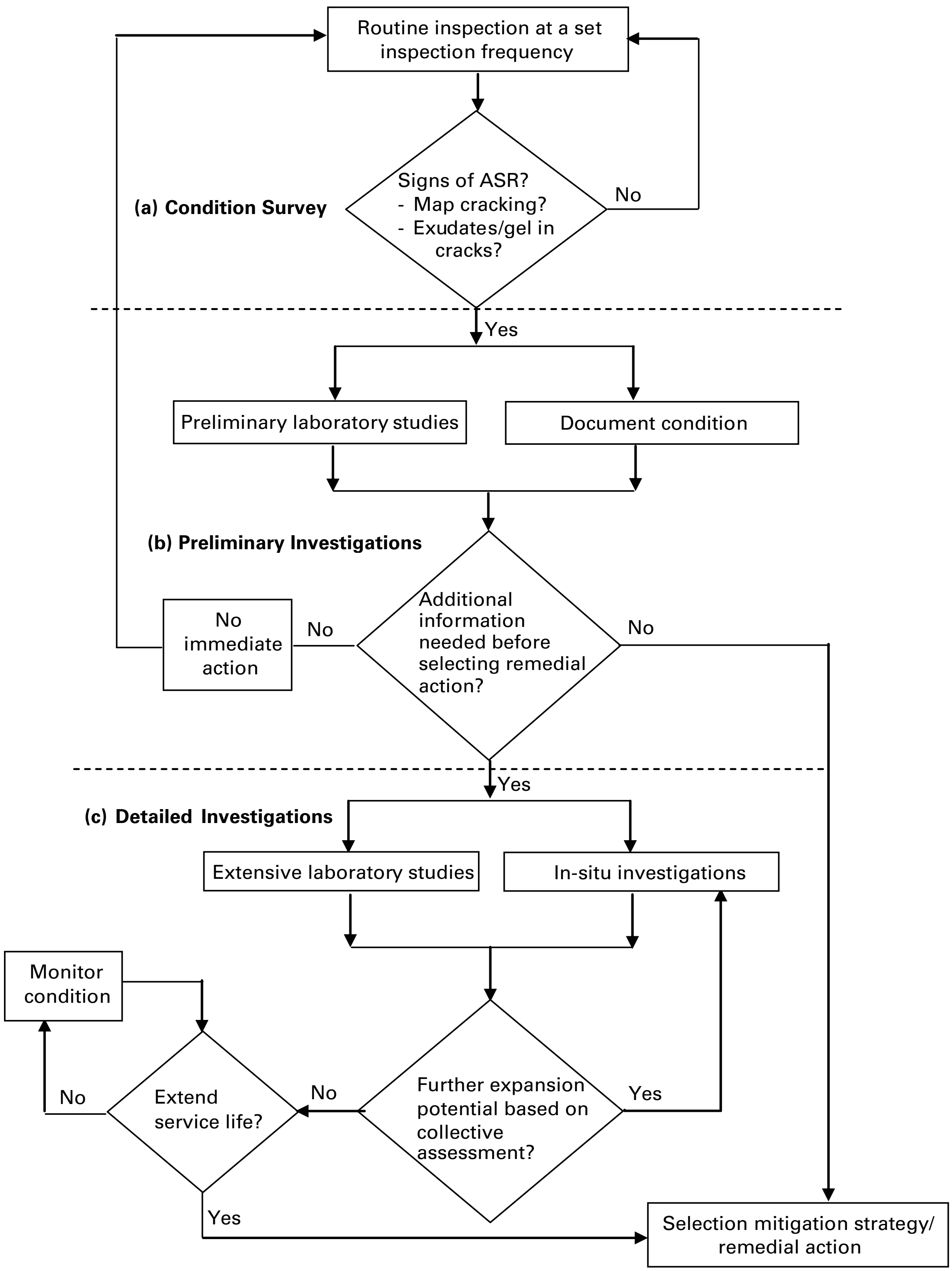


Figure 3.31. Procedure for evaluation and management of ASR in concrete (adapted from Fournier, et al. [2010])

9. Summary

This chapter presents guidelines and procedures on conducting an overall pavement project evaluation. A pavement evaluation is essential to the identification of appropriate and cost-effective solutions to the observed problems. Many premature failures can be attributed to a lack of understanding about the cause or extent of pavement deterioration.

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, performance data, and so on. A collective review of this 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 in order 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 the appropriate treatment alternative. Drainage surveys are performed as part of a pavement distress survey in order 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.

A number of other field testing procedures are available for evaluating an existing pavement, although they may not commonly be needed for candidate pavement preservation projects. These procedures include deflection testing, noise, roughness and friction testing, and field sampling and testing.

Nondestructive testing procedures, such as can be provided by deflection devices, GPR equipment, 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 quieter, smoother, safer 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 are 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. This additional material property data is 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 are described in this chapter.

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Chapter 4

Slab Stabilization and Slab Jacking

1. Learning Outcomes

This chapter covers the use of two different pavement preservation treatments: (1) slab stabilization (also known as undersealing), which is performed to restore support beneath concrete slabs; and (2) slab jacking, which is conducted to physically lift a depressed slab back to the elevation of the adjacent slabs. The participants will be able to accomplish the following upon successful completion of this chapter:

- List benefits of slab stabilization and slab jacking.
- Describe recommended materials and mixtures.
- Identify recommended construction activities.
- Identify typical construction problems and remedies.

2. 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 pavement. Slab stabilization has been 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 provide riding discomfort, but 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 pressure inserting 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.

3. Purpose and Project Selection

Slab stabilization consists of pressure insertion of a flowable material (commonly a cement grout or polyurethane, 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; Wu 1991). To be most effective, it is important that slab stabilization be performed prior to the onset of pavement damage due to loss of support (Wu 1991; ACPA 1994).

Slab jacking consists of the pressure insertion of a cement-grout mixture or polyurethane material beneath the slab to slowly raise it until it reaches a smooth profile. 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 that is more effectively addressed through dowel bar retrofit and/or diamond grinding.

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 and slab jacking performed by themselves 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). Thus, candidate pavements should be evaluated and the need for additional preservation treatments (e.g., dowel bar retrofit, diamond grinding, joint sealing) carefully considered.

4. Limitations and Effectiveness

Slab Stabilization

Over the years, 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 actually 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 DOT concluded the following (Donahue, Johnson, and Burks 2000):

- Slab stabilization and diamond grinding can be an effective concrete pavement rehabilitation CPR 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 undersealing/diamond grinding. Slab stabilization in isolated areas may still be effective.
- Retrofitting edgedrains provide little, if any, additional performance benefit to the combination of undersealing/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.

Slab Jacking

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 6 mm (0.25 in.) at a time 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.

5. Materials and Design Considerations

Determining the Repair Area

Slab Stabilization

For slab stabilization, the first step in the process is locating the areas of voids beneath the slab caused by the base material deterioration. The following techniques have been used to determine whether or not loss of support has occurred beneath a concrete pavement slab:

- **Deflection Data**—This is the most commonly used 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 21°C (70°F) in order to minimize the impact of slab curling (which could erroneously indicate a void) and joint lock-up. Several deflection-based void detection methods are available and include the following:
  - Measure and plot the profile of both the approach and leave corner deflections. An example of this procedure is shown in Figure 4.1, in which deflection measurements are recorded at a constant load at both the approach slab corner and the leave slab corner (Darter, Barenberg, and Yrjanson 1985). 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, Barenberg, and Yrjanson 1985). The 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 0.5 mm (0.02 in.).
  - Measure the magnitude of the corner deflection at three different load levels (Crovetti and Darter 1985). Typically, load levels of 27, 40, and 63 kN (6, 9, and 14 kips) have been used to develop load versus deflection plots for each test location (Crovetti and Darter 1985). The three load levels are required so that a load versus deflection plot can be

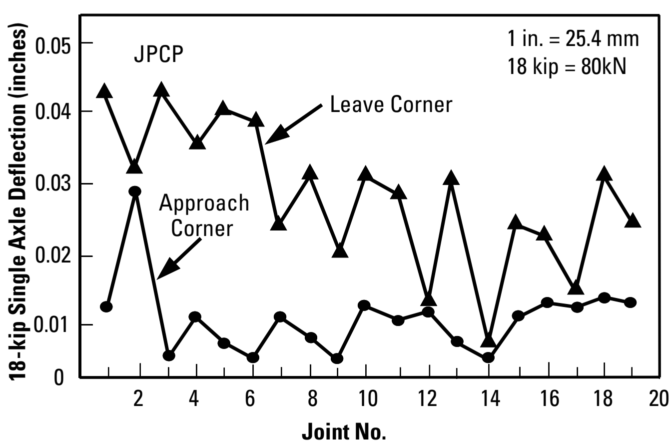

Figure 4.1. Corner deflection profile (adapted from Darter, Barenberg, and Yrjanson [1985])

Ch 4. Slab Stabilization and Jacking

generated. Using these 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; see also Chapter 3, Figure 3.9.

- Measure maximum corner deflection. Some agencies use a maximum corner deflection criterion to determine if a void is present beneath the slab; the deflections can be measured using an FWD or by using a loaded truck with dial gauges placed on the slab corners. Table 4.1 summarizes some available agency-defined maximum corner deflection values that are used to trigger the need for slab stabilization. Specifications based on a single corner deflection, however, may not always provide reasonable estimates of the presence of a void. This is because the variation in load transfer from joint to joint can cause considerable variation in corner deflections. And, as noted previously, curling of the slab may erroneously indicate the presence of a void, so load testing should be conducted when ambient temperatures are below 21°C (70°F).

Table 4.1. Maximum Corner Deflection Criteria Used by Selected States for Assessing the Presence of Voids (Taha et al. 1994)

State	Test Load, kN (lb)[1]	Maximum Corner Deflection, mm (in.)
Florida	[2]	0.38 (0.015)
Georgia	40 (9,000)	0.76 (0.030)
Oregon	[2]	0.64 (0.025)
Pennsylvania	40 (9,000)	0.50 (0.020)
South Dakota	>35.6 (8,000)	0.25 (0.010)
Texas	40 (9,000)	0.50 (0.020)
Washington	40 (9,000)[2]	0.89 (0.035)

[1] Based on current agency Standard Specifications
[2] Not included in current Standard Specifications

- **Ground Penetrating Radar**—Ground-penetration radar equipment and data interpretation techniques have enabled the detection of air-filled voids as small as 6 mm (0.25 in.) thick (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 4.2. Chapter 3 provides additional information on GPR testing.

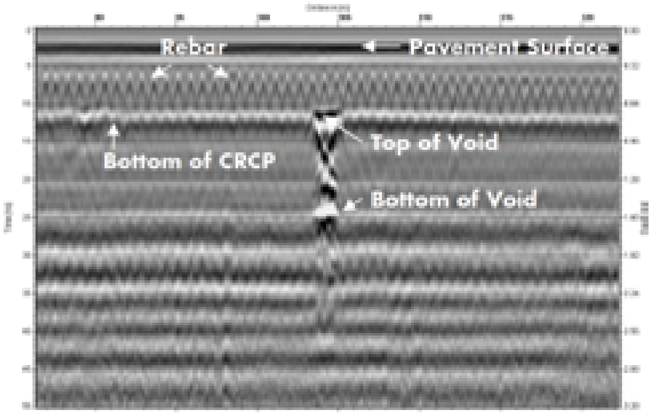

Figure 4.2. Example of GPR image of underlying void (TxDOT 2010)

- **Epoxy/Core Method**—In this procedure, a hole is drilled in the slab at a suspected void location and then a low viscosity, two-part epoxy is poured into the hole, which fills any void that might be present. After the epoxy hardens, a core is taken over the injection hole and examined to note the existence of a void (Chapin and White 1993).
- **Visual Observations**—Faulting of transverse joints and cracks, pumping, corner breaks, and shoulder dropoff all indicate that loss of support has occurred (ACPA 1994). Figure 4.3 shows the progression of deterioration in nondoweled concrete pavements as it occurs in four stages (Darter, Barenberg, and Yrjanson 1985). Ideally, slab stabilization should be conducted in the second stage before the onset of significant void development; at later stages, more substantial preservation treatments (e.g., FDRs) are required.

Slab Jacking

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 or inadequate compaction of the underlying fill. Localized settlements may also occur over embankment areas. Subsurface testing (such as through the use of a dynamic cone penetrometer) may be performed to identify soil and base properties and the potential extent of the settlement area. Figure 4.4 shows before and after photos of the raising of a settled slab.

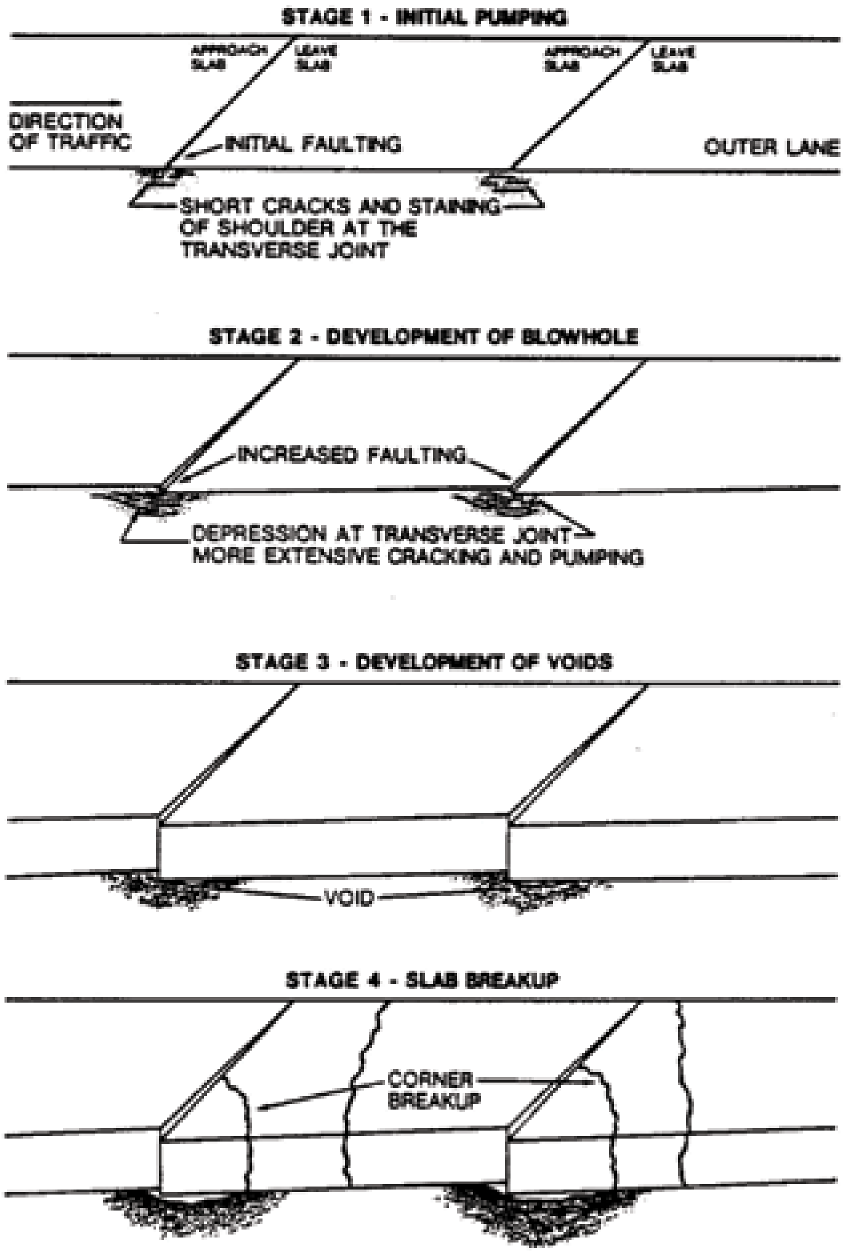

Figure 4.3. Typical stages in the deterioration of a concrete pavement (Darter, Barenberg, and Yrjanson 1985)

Figure 4.4. Settled slab before (left) and after (right) slab jacking (courtesy of John Roberts, International Grooving and Grinding Association [IGGA])

Selecting an Appropriate Injection Hole Pattern

Slab Stabilization 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 pattern is dependent on a number of factors, including the following (ACPA 2003):

- Pavement type (i.e., JPCP, JRCP, CRCP).
- Transverse joint spacing (jointed pavements).
- Estimated size and shape of the detected void.
- The flowability of the material being used.
- Location of cracks and joints near 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 close enough to achieve a flow of grout from one insertion hole to another when a multiple hole pattern is used. Figure 4.5 illustrates recommended initial trial hole patterns for different void locations on jointed concrete pavements. 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. A typical hole spacing for CRCP is shown in Figure 4.6.

Slab Jacking Hole Pattern

The best 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 305 mm (12 in.) nor more than 457 mm (18 in.) from a transverse joint or slab edge (MnDOT 2006). In addition, holes should be spaced 1.8 m (6 ft) or less center to center, so that less than 2.32–2.78 m^2 (25–30 ft^2) of the slab is raised by grouting a single hole (MnDOT 2006). Figure 4.7 illustrates an example pattern in which the holes are placed in a triangular fashion to cor-

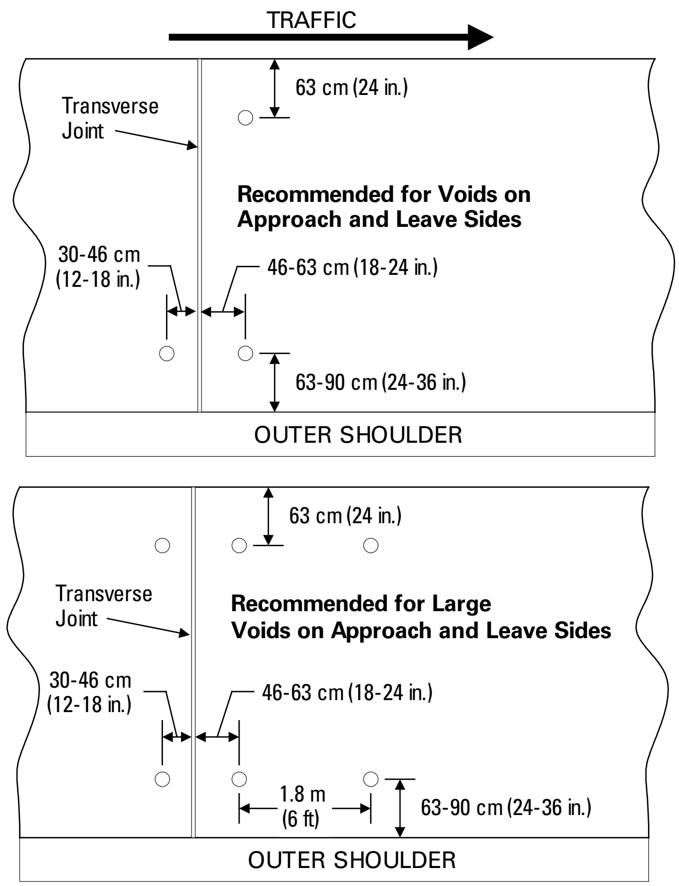

Figure 4.5. Typical hole patterns for jointed concrete pavements (Darter, Barenberg, and Yrjanson 1985)

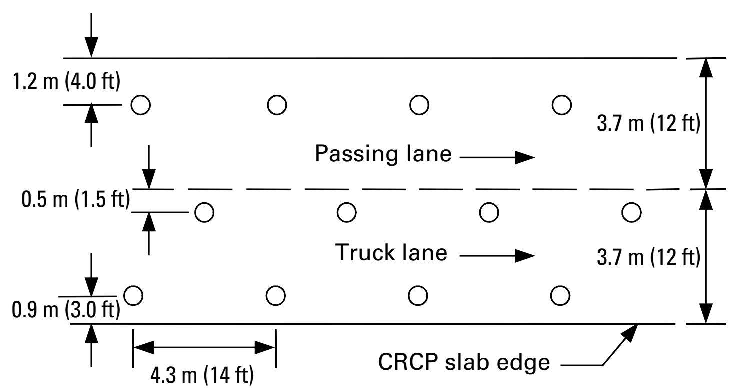

Figure 4.6. Typical hole pattern used on CRCP (Barnett, Darter, and Laybourne 1980)

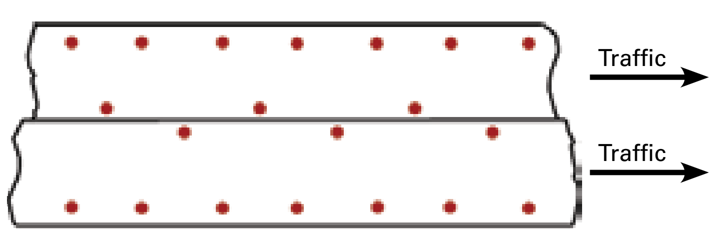

Figure 4.7. Example pattern of grout pumping holes used to correct a settlement

rect 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.

Selecting an Appropriate Material

The material chosen for slab stabilization must be able to penetrate into very thin voids while having the strength and durability to withstand pressures caused by traffic, moisture, and temperatures. Many different slab stabilization materials have been used, with cement grout and polyurethane being the most common. Other materials used less frequently include asphalt cement, limestone dust-cement grouts, and silicone rubber foam (Taha et al. 1994), with asphalt seeing a slight resurgence in recent years.

Materials used for slab jacking are typically slightly stiffer than those used for slab stabilization. Cement grout and polyurethanes are commonly used for slab jacking.

Cement Grout Mixtures

Historically, the more common cement-based grout mixtures included pozzolanic-cement and limestone-cement grout (Taha et al. 1994). The typical flow cone time for limestone grouts is 16 to 22 seconds, whereas the typical time for flyash 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 concrete 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 flyash; it may be possible to reduce the cement component if Class C flyash is used); pozzolans shall conform to the requirements of ASTM C 618, if used, and limestone dust shall comply with AASHTO M 17 for mineral fillers.
- Water (usually about 1.5 to 3.0 parts) to achieve required fluidity.
- If ambient temperatures are below 10°C (50°F), an accelerator may be used (if approved).
- A minimum compressive strength (typically 4.1 MPa [600 lbf/in^2] 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 10–28 MPa [1,500–4,000 lbf/in.^{2}]).
- 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 chemical and physical properties of the pozzolan or limestone; 1-, 3-, and 7-day compressive strength tests; flow cone results; time of initial set; and shrinkage/expansion results.

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 flyash) to one part concrete, with enough water to produce the desired consistency (MnDOT 2006).

Polyurethane

Polyurethane materials have become 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 48–64 kg/m^3 (3–4 lb/ft^3) and the compressive strength ranges from about 0.4 to 1.0 MPa (60 to 145 lbf/in.^{2}) (ACPA 1994). One laboratory study indicated that the injected polyurethane will consistently penetrate openings as small as 6.4 mm (0.25 in.) and will penetrate some openings as small as 3.2 mm (0.125 in.) (Soltesz 2002).

Polyurethane materials offer a number of advantages for use in slab stabilization and slab jacking, including the following (ACPA 1994; Soltesz 2002; Gaspard and Morvant 2004):

- Lightweight (so does not contribute to additional settlement).
- High compressive and tensile strengths.
- Expansive (ability to fill surrounding voids).

- Insensitive to moisture.
- Rapid curing (opening times of 15 to 30 minutes).

A number of highway agencies, including Oregon (Soltesz 2002), Missouri (Donahue, Johnson, and Burks 2000), and Kansas (Barron 2004), have had good success in using polyurethane foam for slab stabilization. Moreover, for slab jacking, the Wisconsin DOT (Abu al-Eis and LaBarca 2007) reported that the lifting process was successful and that trial projects are performing well after 1 year of service, but it also indicated shortcomings in the ability to estimate material quantities. On the other hand, a few agencies report 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 be deflecting under traffic loadings; however, it was also reported that the load transfer devices in this pavement were not functioning on the project in question (Gaspard and Morvant 2004).

6. Construction Considerations

Slab Stabilization

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). Specifically, for concrete-based grout projects, any pneumatic or hydraulic rotary percussion drill that is capable of cutting 38- to 51-mm (1.25- to 2.0-in.) diameter holes through the slab is suitable. A general specification recommends limiting the downward pressure on any drill to 90 kg (200 lb) to avoid conical spalling at the bottom of the slab (ACPA 1994). When large pieces of the underside of the slab spall, these pieces can potentially block the void and make it impossible to fill.

For polyurethane slab stabilization, handheld electric-pneumatic rock drills are typically used to drill the injection holes (ACPA 1994). For these procedures, the maximum hole diameter should not exceed 15 mm (0.625 in.) (ACPA 1994). Figure 4.8 provides an illustration of the hole-drilling process.

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 grout is pumped into the hole). If the hole does not take water, there is no void and therefore no need to grout. 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 in order to effectively fill each void. If the 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.

Figure 4.8. Drilling injection holes (photo courtesy of WSDOT)

Step 2: Material Preparation

Most slab stabilization contractors use mobile, self-contained equipment that carries all of the tools and materials needed for slab stabilization (ACPA 1994). As past procedures typically utilized labor-intensive, small batch mixers with bagged materials, these modern systems have been found to reduce both labor and materials costs by as much as 30 to 50 percent (ACPA 1994). The differences in preparing cement-based and polyurethane materials are discussed in this section.

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. Colloidal mixers provide the most thorough mixing for pozzolanic-cement grouts, because the material stays in suspension and resists dilution by free water (ACPA 1994). Two of the more common types of colloidal mixers include the following (ACPA 1994):

- **Centrifugal Pump Mixer**—This mixer pulls grout through a mixing chamber at high pressure and velocity.
- **Shear Blade Mixer—**For this mixer type, blades rotate at 800 to 2,000 revolutions per minute.

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 tend to agglomerate and clump in the mix (ACPA 1994).

Polyurethane

When using polyurethane, all material is stored, proportioned, and blended within a self-contained pumping unit. The handling and usage of these materials should be in accordance with the material manufacturer's instructions and specifications.

Step 3: Material Injection

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

Injection of Cement-Grout Mixtures

It is recommended that positive-displacement injection pumps, or nonpulsing progressive-cavity pumps, be used during the slab stabilization process. 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 0.15 and 1.4 MPa (25 and 200 lbf/in.^{2}) during grout injection (ACPA 2003). Maintaining a lower pumping rate (ideally about 5.5 liters [1.5 gallons] per minute) and lower pumping pressure ensure better placement control and lateral coverage, and it usually keeps the slab from rising (AASHTO 1993). Typical pumping pressures are in the 275- to 413-kPa (40- to 60-lbf/in.^{2}) range (ACPA 2003).

Concrete-based grouts are typically injected using a grout packer in order 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 25-mm (1.0-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-inch 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, Barenberg, and Yrjanson 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 4.9. 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.

Slab stabilization should not be performed when the ambient temperature is below 4°C (40°F). Unless a fast-setting material is used, traffic should be kept off of a stabilized slab for at least 3 hours after grouting to allow adequate curing of the grout (Darter, Barenberg, and Yrjanson 1985).

Figure 4.9. Patching drill holes (photo courtesy of WSDOT)

Injection of Polyurethane

The injection process for polyurethane materials is similar to that used for cement-grout stabilization, but it does employ pumping equipment specific to the use of polyurethane materials. Pressure and temperature control devices are found on the equipment that is capable of maintaining proper temperature and proportionate mixing of the polyurethane component materials. In addition, the polyurethane grouting operations use slightly different injection equipment consisting of plastic nozzles that screw onto the hoses and deliver the material into the holes (ACPA 1994). And, as previously described, the injection of polyurethane materials uses a smaller injection hole, typically 15 mm (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 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 4.10 shows a photo of the polyurethane injection process.

Figure 4.10. Injecting polyurethane (photo courtesy of WSDOT)

Slab Jacking

The slab jacking process for grout or polyurethane injection is similar to that of slab stabilization; however, procedures are required for monitoring the raising of the slab and ensuring that the profile meets the desired grade. The taut stringline method (illustrated in Figure 4.11) is the traditional way not only to control the pumping sequence, but also to achieve the proper grade. In the stringline method, small wooden blocks, 19 mm (0.75 in.) high, are

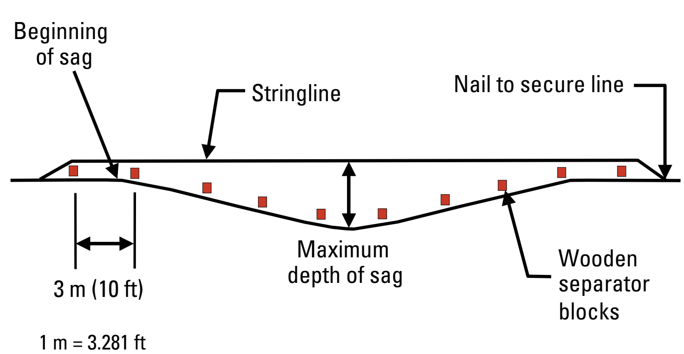

Figure 4.11. Stringline method of slab jacking

set on the pavement surface along the outer and inner edges and a stringline is secured at least 3 m (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 6–9 mm (0.25–0.38 in.). Although the stringline method has worked well, laser technology is being used by many contractors for monitoring pavement elevations because of its increased speed and accuracy.

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

- After all preliminary work has been completed (holes drilled, relief opening 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 6 mm (0.25 in.) while pumping in any one hole. No portion of the slab should be more than 6 mm (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 6 mm (0.25 in.), throughout the entire operation to avoid cracking.
- 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 it 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.
- Care must be taken not to 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.
    - Figure 4.12 shows a plan view of a dip. Pumping should begin in the middle of the dip, shown as point A. 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 holes, followed by the inside row of holes.
    - Pumping at point B relieves the strain that may have resulted from lifting the slab at point A. The third hole to be grouted will be at point A again, and then material is pumped following steps 4 to 8 as shown in Figure 4.12. This results in material being pumped four times into the middle hole at point A and twice at the hole on either side at points B. If the same amount of material was pumped at each time and traveled the same distance away from the hole, the slab would be raised twice as much at the middle hole as at the other two. Pumping should never be performed along a series of holes back and forth across the slab; instead, work always proceeds along the length of the slab to avoid cracking. A concrete slab can withstand more twisting than transverse bending.
    - The line of holes in the middle of the pavement is pumped after the outer row, using the same sequence. If both sides of the slab are at about the same elevation, the next pumping is at the outer side of the adjoining slab at point C, following the same sequence, with additional pumping conducted further from the center of the dip (i.e., grout applications 9, 10, 11, 12, and 13). Pumping is continued in this order until the slab has been brought to the desired elevation.

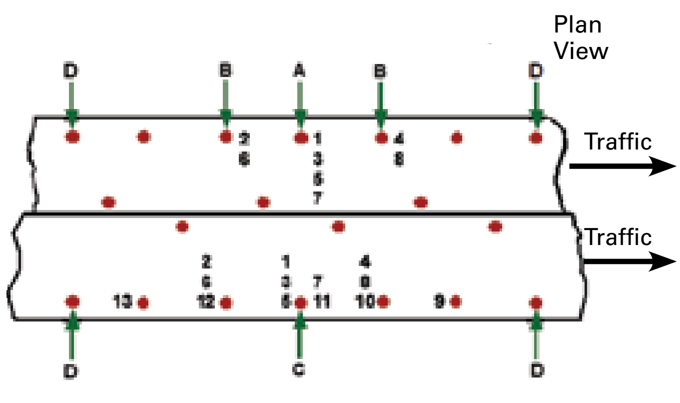

Figure 4.12. Order of grout pumping used to correct a settlement

- ◦ The last hole at each end of the dip, shown as point D in Figure 4.12, 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.

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).

7. Quality Assurance

Slab Stabilization

The purpose of slab stabilization is to fill existing voids and 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 and may lead to slab cracking. The success of the slab stabilization operations is highly dependent upon the skill of the contractor.

The injection process should start with a low pumping rate and pressure and should be pumped until one of the following conditions occurs (Darter, Barenberg, and Yrjanson 1985):

- A maximum allowable pressure of 0.69 MPa (100 lbf/in.^{2}) is obtained (for cement grouts). Note that a short surge of up to 1.38 MPa (200 lbf/in.^{2}) can be allowed when starting to pump in order for the grout to penetrate the void structure, if necessary.
- The slab lift exceeds 3 mm (0.125 in.).
- 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 0.025 mm (0.001 in.) of uplift movement. Several methods of monitoring slab uplift are shown in Figure 4.13. As indicated in these photos, 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. Table 4.1, presented earlier, listed critical deflection values used by agencies in determining the potential for voids and ranged from 0.25 to 0.89 mm (0.010 to 0.035 in.). Thus, reduc-

Figure 4.13. Methods of monitoring slab uplift (photos courtesy of Wouter Gulden, retired Georgia DOT)

tions below those threshold levels are desirable, along with concomitant increases in load transfer efficiency at the transverse joints (values greater than 70 percent are desirable).

As an example of assessing the effectiveness of slab stabilization, Figure 4.14 presents the deflection at the slab corners before and after slab stabilization for a Missouri DOT project. In general, Figure 4.14 indicates that the slab corner deflection decreased by more than 30 percent for 16 of the 22 testing locations, with 9 of those locations showing more than a 50 percent reduction in corner deflection.

If the follow-up deflection testing still indicates a loss of support, the slabs should be regrouted using new drilled holes. Guidelines from the 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).

Slab Jacking

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 6 mm (0.25 in.) at a time when pumping at each hole. In addition, no portion of the slab should be more than 6 mm (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 stringline 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 4.15 shows the profile of a bridge approach slab, both before and after the slab jacking operation.

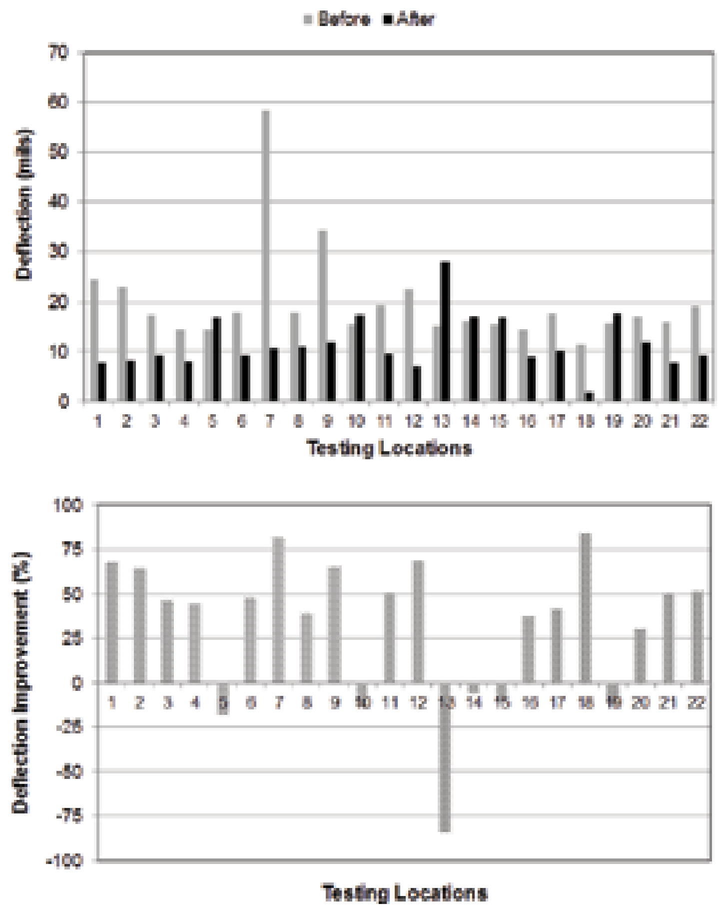

Figure 4.14. Corner slab deflection before and after slab stabilization (after Donahue 2004)

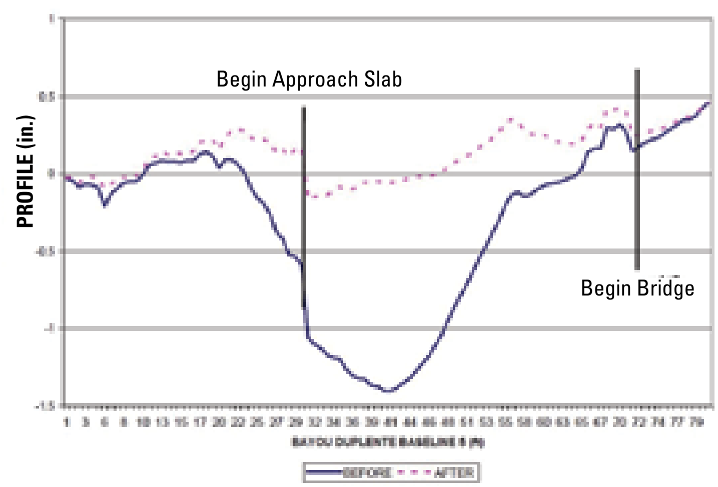

Figure 4.15. Bridge approach slab profile before and after slab jacking (Gaspard and Morvant 2004)

8. 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 4.2. Typical causes and recommended solutions associated with known problems are also provided.

Table 4.2. 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: 1. 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. 2. 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 0.125 in.).	Overgrouting occurred.	Overgrouting a void can cause immediate cracking or, as 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 close 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.

9. 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 is caused by several factors, however, slab stabilization is often done in conjunction with other rehabilitation activities (e.g., patching, diamond grinding, dowel bar retrofit) in order to address the causes of the voids (ACPA 1994). Commonly used slab stabilization materials include cement-based mixtures (limestone-cement dust slurry and pozzolanic-cement slurry) 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 in order to avoid overgrouting the slab and associated slab damage. An experienced contractor and proper inspection are essential to a successful 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 and careful monitoring of the lift at different insertion holes until the desired profile is obtained. Slightly stiffer cement grouts than those used for slab stabilization are required for slab jacking. During slab jacking, the stringline 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. Learning Outcomes

This chapter describes recommended procedures for the partial-depth repair (PDR) of concrete pavements. Upon completion of this chapter, the participants should be able to accomplish the following:

- List benefits and appropriateness of using PDRs.
- List the advantages and disadvantages of different repair materials.
- Describe recommended construction procedures.
- Identify typical construction problems and appropriate remedies.

2. Introduction

Partial-depth repairs are defined as the removal of small, shallow 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 the service life of pavements that have spalled or distressed joints. 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 FDRs in areas where slab deterioration is located primarily in the upper one-third to upper one-half of the slab and the existing load transfer devices (if any) are still functional. When applied at appropriate locations and using effective materials and procedures, PDRs can be more cost effective than FDRs. The costs of a PDR are largely dependent upon the size, number, and location of the repair areas, as well as the materials used. Lane closure times and traffic volumes also affect production rates and costs.

There are a number of key reference documents available on PDRs, including a manual of practice from the FHWA (Wilson, Smith, and Romine 1999), a training course reference document from the National Highway Institute (Hoerner et al. 2001), a field guide from the American Concrete Pavement Association (ACPA 2006), and, more recently, a detailed guide document from the National Concrete Pavement Technology Center (Frentress and Harrington 2012).

3. Purpose and Project Selection

Partial-depth repairs replace deteriorated concrete only, and most repair materials cannot accommodate the movements across working joints and cracks without experiencing high stresses and material damage. As a result, they are appropriate only for certain types of concrete pavement distresses that are confined to the top one-third to one-half of the slab thickness. Distresses that have been successfully corrected with PDRs include the following:

- Spalling caused by the intrusion of incompressible materials into the joints.
- Spalling caused by poor consolidation, inadequate curing, or improper finishing practices.
- Spalling 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 or scaling that are limited to the upper one-third to one-half of the slab thickness and are of sufficient size and depth to warrant repair.

Concrete pavement distresses that are not candidates for PDRs include the following:

- Spalling caused by dowel bar misalignment or lock-up.
- Spalling of transverse or longitudinal cracks caused by shrinkage, fatigue, or foundation movement.
- Spalling caused by MRD, such as D-cracking or reactive aggregate.

4. Types of Partial-Depth Repairs

Frentress and Harrington (2012) define three general types of PDRs for cracks, joints, and spalls, as illustrated in Figure 5.1. More details on these repair types are provided in the following sections.

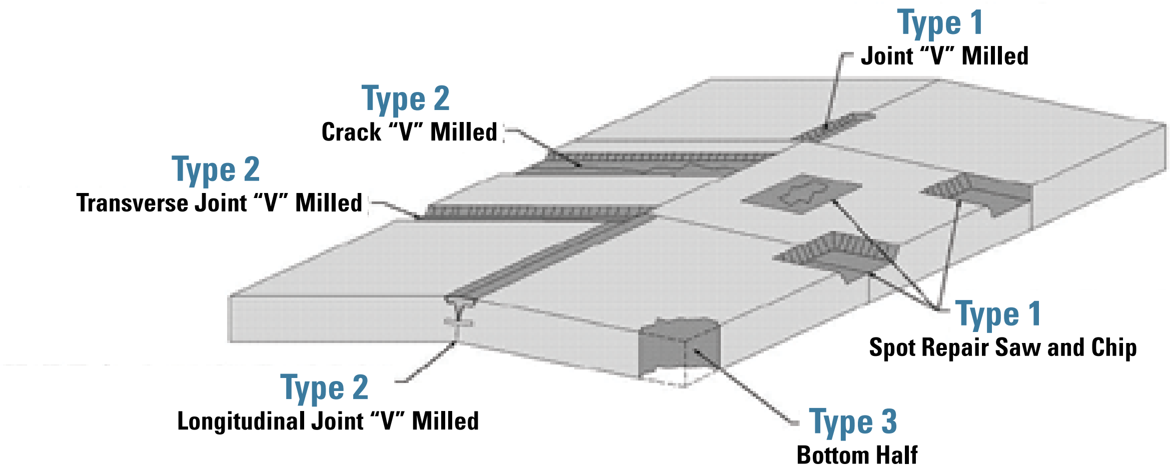


Figure 5.1. Types of PDRs (Frentress and Harrington 2012)

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

These types of repairs are generally performed to address localized areas of deterioration and are not recommended for long, continuous repairs. These repairs are typically less than 1.8 m (6 ft) long and usually extend to a depth of around 50 mm (2 in.) with a tapered edge from 30 to 60 degrees to the bottom of the crack/joint. Type 1 repairs can be used to address the following distresses (Frentress and Harrington 2012):

- Joint spalling.
- Mid-slab surface spalling or cracking.
- Severe surface scaling.
- Joint reservoir issues.

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). The repair area should be angled out slightly (approximately 30 to 60 degrees) at the edges to help facilitate bonding. Typical details for Type 1 repairs using these removal methods are shown in Figures 5.2.

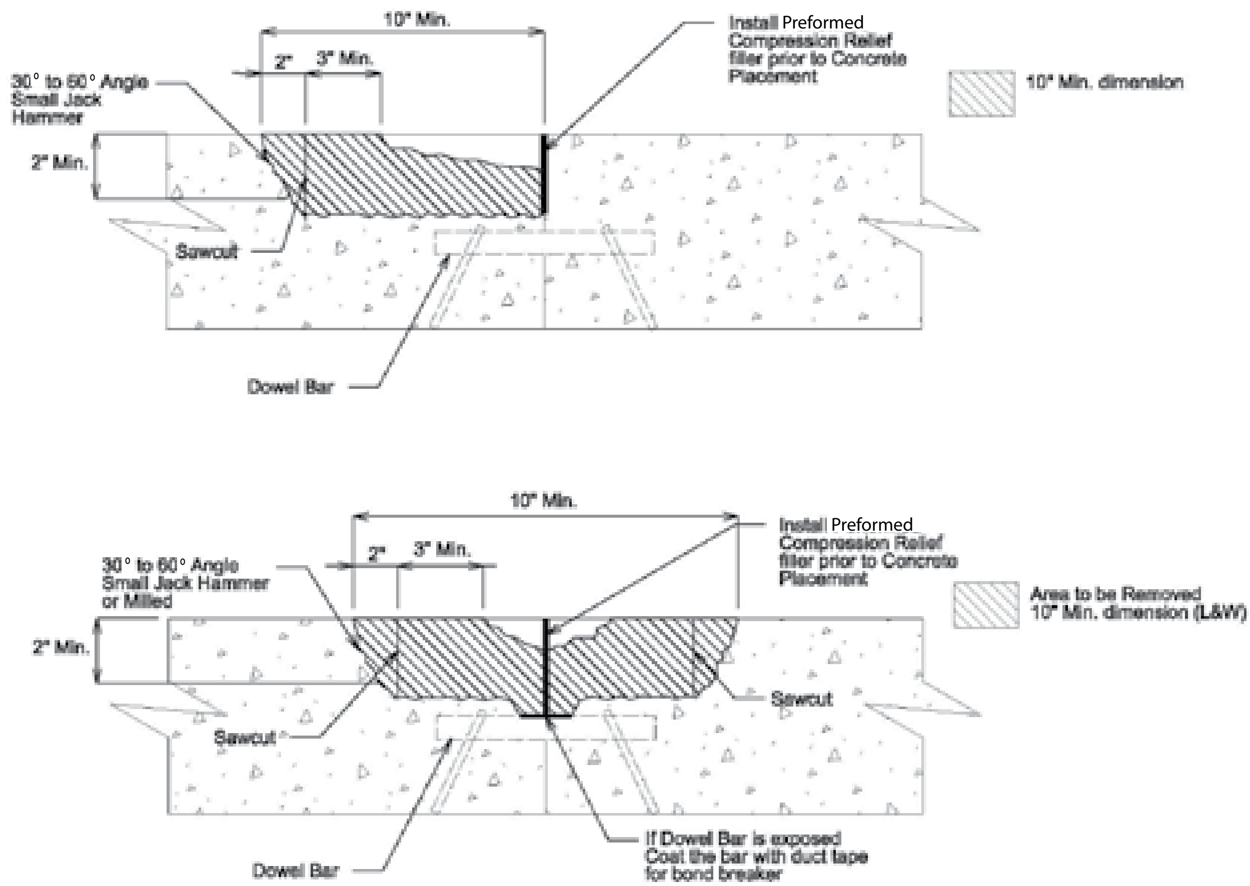


Figure 5.2. Typical details for Type 1 repairs: saw and chip (top) and milled (bottom) (Frentress and Harrington 2012)

Type 2: Joint and Crack Repairs

These types of repairs are performed on longitudinal or transverse joints (Type 2A) or cracks (Type 2B) longer than 1.8 m (6 ft) and can go to one-half of the depth of the slab (Frentress and Harrington 2012). Figure 5.3 shows candidate distresses for Type 2 repairs.

Compression relief is constructed differently for Types 2A and 2B repairs. For Type 2A repairs (which are performed on joints), the joint is reestablished, typically by sawing; for Type 2B repairs (which are performed on cracks), a preformed joint compression material is installed in the crack. Typical details for Type 2 repair are shown in Figure 5.4.

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 6–25 mm (0.25–1 in.). The general procedure for constructing joint and crack repairs is the same as for spot repairs, with the exception of the provision of compression pressure relief (Frentress and Harrington 2012).

Figure 5.3. Candidates for Type 2 repairs (Frentress and Harrington 2012)

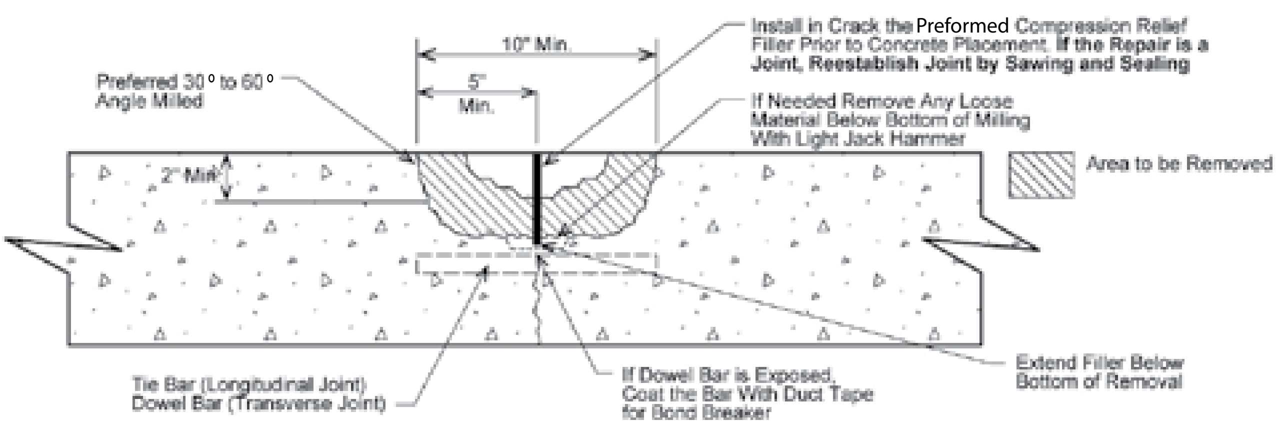


Figure 5.4. Typical details for Type 2 joint repair (Frentress and Harrington 2012)

Type 3: Bottom-Half Repairs

Bottom-half repairs are essentially full-depth corner repairs and are used to repair slab edges and corners that deteriorate to a depth extending beyond one-half of the thickness of the slab for a short distance (around 460 mm [18 in.]) (Frentress and Harrington 2012). Typical candidate distresses for bottom-half repairs are shown in Figure 5.5.

Bottom-half repairs performed at the outer edges of a slab should not protrude more than 460 mm (18 in.) in the transverse direction at the bottom of the repair. Type 3 repairs performed at longitudinal joints can extend beyond 460 mm (18 in.) along the longitudinal joint but not transversely into the lanes on either side beyond 460 mm (18 in.). Typical details for Type 3 repair (without deterioration beneath the dowel bars) are shown in Figure 5.6. It is noted that FDRs are recommended when the transverse length of the repair extends beyond 460 mm (18 in.) into lanes on either side of the longitudinal joint.

Figure 5.5. Candidate distresses for bottom-half repairs (Frentress and Harrington 2012)

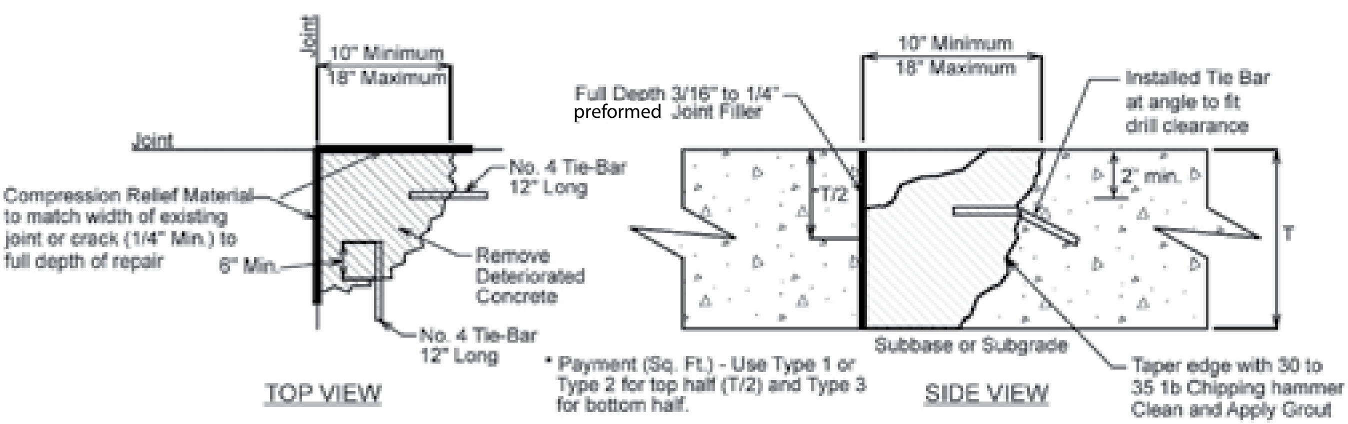


Figure 5.6. Typical details for Type 3 bottom-half repair (Frentress and Harrington 2012)

5. Installation Costs and Payment Methods

The cost of PDRs typically depends on the type of materials used, the location of the repair areas, and the size and number of repairs. Lane-closure restrictions and traffic volumes are also expected to impact the production rates and costs. Typical costs (2013) for PDRs range from \$269 to \$377 per m^2 (\$25 to \$30 per ft^2) or \$49 to \$82 per lineal m (\$15 to \$25 per lineal ft), although, again, this will vary depending on the size of the project, the materials used, and the project conditions and constraints (e.g., night work, early opening requirements). These cost values are for informational purposes only, however, and local agency and industry representatives should be consulted for more accurate numbers.

The payment method for PDRs is typically determined in one of two ways: by length of repair area or by surface area of repair area. Payment by length is typically done for Type 2A repairs of longitudinal and transverse joints longer than 1.8 m (6 ft), whereas payment by repair surface area is generally used for all other repair situations (Type 1, Type 2B, and Type 3). Warranty requirements and allowance for design changes based on site-specific requirements are also typically included on repair contracts (Frentress and Harrington 2012).

6. Limitations and Effectiveness

As described earlier, PDRs are an effective treatment for joint or crack spalling that is isolated in the upper portion of the slab, or for surface scaling or spalling. It is not recommended for use in addressing MRD, dowel bar lock-up, or spalling of working cracks. These recommendations are based on minimizing the potential risk that can be associated with using PDR in inappropriate locations. Care should also be exercised in using PDRs where the depth of the deterioration exceeds one-third to one-half of the slab thickness. In particular, when deterioration reaches the depth of the dowel bar, an FDR is often required.

The performance of PDRs depends on the general condition of the existing pavement, the type of materials used, construction, and placement techniques. In general, when sound construction practices and a durable material are used, PDRs can last up to 15 years or longer, but when poor materials or workmanship are encountered, PDRs may fail in as little as 2 to 3 years (ACPA 2006).

7. 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, what to consider 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 of 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 percent or more) to estimate quantities for bidding purposes. Actual quantities will emerge during the construction process when each individual repair area is examined and sounded. It is important that all weak and deteriorated concrete must be located and removed if the repair operation is to be effective.

It is generally recommended that the repair boundaries extend 75 mm (3 in.) beyond the detected delaminated or spalled area to ensure removal of all unsound concrete; see Figure 5.7, but some judgment is still required based on the severity of the deteriorated conditions. A minimum repair length of 250 mm (10 in.) and a minimum repair width of 100 mm (4 in.) are recommended for cementitious materials (Wilson, Smith, and Romine 1999), but proprietary materials should follow the manufacturer's recommendations for repair dimensions. The repair area should also be kept 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 600 mm (24 in.) apart, they should be combined to help reduce costs and eliminate numerous small repairs (ACPA 2006).

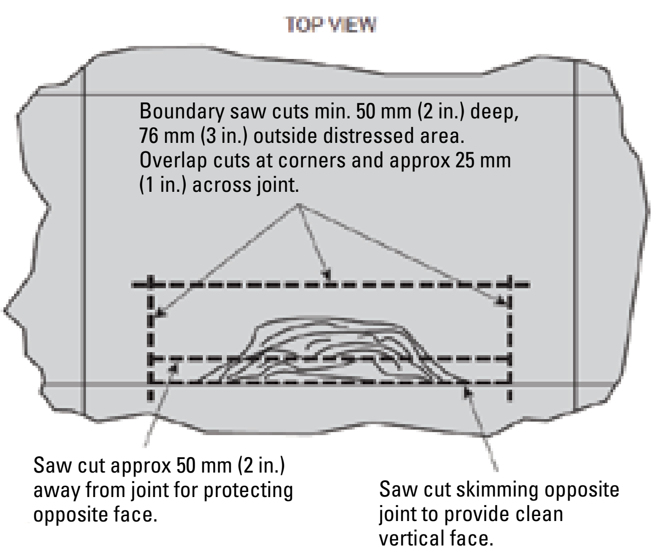


Figure 5.7. Recommended repair boundaries for PDRs (ACPA 2006)

Repair Material Types

A variety of materials is available for use in PDRs, from conventional cementitious materials to proprietary, early-strength cementitious and polymeric products; in addition, various bituminous-based mixtures (both conventional and proprietary) may also be used. 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. This section describes some of the characteristics and properties of common materials used for PDRs.

Concrete

High-quality concrete is generally accepted as the most appropriate material for PDRs. Typical mixes combine concrete with coarse aggregate not larger than one-half the minimum repair thickness (a 9.5 mm [0.375 in.] maximum size is often used). The material should be a low-slump mixture of air-entrained concrete having a water-cement ratio not exceeding 0.44. For repairs that must be opened to traffic quickly, a mix 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 (up to eight bags of cement, or 446 kg/m^3 [752 lb/yd^3]) gain strength rapidly in warm weather, although the rate of strength gain may be too slow to permit quick opening to traffic in cool weather. In those 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 can be more difficult to work and place; hence, additional caution and care should be exercised with their use and placement.

Since PDRs are confined and supported by the existing concrete, the minimum compressive strength requirements to support traffic loading without experiencing any deterioration are lower (typically between 11 and 12.5 MPa [1,600 and 1,800 lb/in.^{2}]) than those required for conventional FDRs (typically 13.8 to 20.7 MPa [2,000 to 3,000 lb/in.^{2}] or higher). In addition to strength, other material properties that affect the short- and long-term performance of PDRs include the coefficient of thermal expansion (CTE), the elastic modulus, shrinkage, and bond strength.

Most highway agencies have developed standard mixtures for use in PDRs. As an example, the Minnesota DOT has had good success with a cementitious mixture that provides an 18-hour opening strength of 20.7 MPa (3,000 lb/in.^{2}); the composition 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 w/c ratio 0.35.
- Type E water reducing and accelerator.
- 6.5 percent air.

As dictated by opening requirements, there are also several proprietary concrete-based repair materials available to achieve high-early strength for critical PDRs.

Modified Hydraulic Cements

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

- Gypsum-based cements contain calcium sulfates that provide setting times of 20 to 40 minutes and can be opened to traffic in as little as 1 hour (depending on conditions). They are recommended for use in temperatures above freezing and are not affected by deicing chemicals, but they generally require dry conditions during placement. Furthermore, they 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, Patel, and Romine 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. They are particularly good in low-temperature applications or those needing flexibility of placement time while still supplying good early strength. It is noted that, either during the initial curing or at some point later, 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 mix 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, Gosselin, and Barborak (2013).

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 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 quicker 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.

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.

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 mix (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 concrete may lead to failures of the repair, but a more critical concern is avoiding point-to-point contact between the existing concrete and the new repair material. In addition, deep repairs must frequently be placed in multiple lifts to control heat development.

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. The manufacturer's recommendations should be followed for the depth of placement and repair configuration for these repair materials.

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 water, either on the substrate or in the mix (even very small amounts of excess water cause severe strength reduction). Furthermore, magnesium phosphate concretes are very sensitive to aggregate type (limestones are not acceptable). In hot weather (greater than 32°C [90°F]), many mixes experience short setting times (less than 15 minutes) and can be difficult to work with; as a result, some manufacturers produce different 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 D-cracked or otherwise deteriorated to permit FDRs. It is again emphasized, however, that the use of conventional bituminous materials should be limited as a stop-gap, temporary repair measure.

Proprietary and Modified Bituminous Materials

As previously described, several proprietary modified bituminous materials are available for use in PDR applications on highway pavements. Although the 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.

Repair Material Selection Considerations

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 they are but one aspect of the PDR "system." Thus, the performance of the PDR not only depends on the repair material itself, but also on the proper application of the repair on a suitable project and on the repair being properly constructed and installed. 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 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 opening requirements used by several highway agencies (Frentress and Harrington 2012).

In addition to strength, other factors also play a role in the short- and long-term performance of the repair, as was described previously. For example, shrinkage characteristics and the CTE of the material should be considered. Drying shrinkage of most repair materials is greater than normal concrete, and when the material is restrained it can induce a tensile stress as high as 6,900 kPa (1,000 lbf/in.^{2}) (Emmons, Vaysburd, and McDonald 1993). 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 bond development or a bond that is weakened by differential movements, resulting in a delaminated repair that breaks up under loading.

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; the 1- and 7-day performance requirements are 7 MPa (1,000 lbf/in.^{2}) and 10 MPa (1,500 lbf/in.^{2}), 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 a 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.

Another important property of the repair material is freeze-thaw durability. A study of the properties of repair materials found that the freeze-thaw durability of many materials is unacceptable, especially under severe exposure conditions (Smoak, Husbands, and McDonald 1997; ACI 2006). Moreover, materials with rapid strength gain characteristics may be particularly susceptible to durability problems because of the accelerated nature of the material and the reduced curing times. 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). And, 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/c, and by increasing the aggregate volume as long as workability is maintained (Van Dam et al. 2005). ASTM C666 is commonly used to assess freeze-thaw durability.

Table 5.1. Example of Opening Strength Requirements for PDRs (Frentress and Harrington 2012)

State	Compressive Strength (psi)
New York	1,527
Kansas	1,800
Missouri	1,600
Michigan	1,800
Minnesota	3,000
Colorado	2,500
Nebraska	3,624

Table 5.2. Laboratory Test Methods to Evaluate Properties of Cementitious Repair Materials

Property	Test Method
Compressive strength	ASTM C39
Free/Drying shrinkage	ASTM C157
Restrained shrinkage	ASTM C1581
Slant-shear bond strength	ASTM C882 (as specified by ASTM C928)
Tensile bond strength	ASTM C1583
Modulus of elasticity	ASTM C469
Coefficient of thermal expansion	ASTM C531
Freeze-thaw resistance	ASTM C666

Material-related factors can contribute to the premature failure of PDRs in a number of ways, including the following (Wilson, Smith, and Romine 1999):

- Incompatibilities between the climatic conditions during repair replacement and the materials or procedures used.
- Thermal incompatibility between the repair material and the pavement.
- Extreme climatic conditions during the life of the repairs that are beyond the capabilities of the repair material.
- Inadequate cure time prior to opening repairs to traffic.
- Incompatibility between the joint bond breaker and the joint sealant material.

Bonding Agents

Most concrete repair materials generally require the placement of a bonding agent to enhance the bond between the repair material and the existing pavement. Sand-cement grouts are adequate under most conditions when used as bonding agents with concrete repair materials. Epoxy bonding agents have been used with both concrete and proprietary repair materials as a means of reducing repair closure times.

A successful cement grout formulation used by many highway agencies 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. It is important that this bonding agent not be allowed to dry out, which would inhibit the bond between the two materials. If this were to occur, the dried material must be removed by sandblasting and a new application of the bonding agent applied before placing the repair material.

The Kansas DOT uses an alternative approach in their formulation of a bonding agent—a more watery mix that helps cool the existing concrete and provides a prewetting to the concrete repair area, which keeps the existing concrete from pulling moisture from the repair material (Frentress and Harrington 2012). The grout mixture typically consists of one part Type I cement and three parts water.

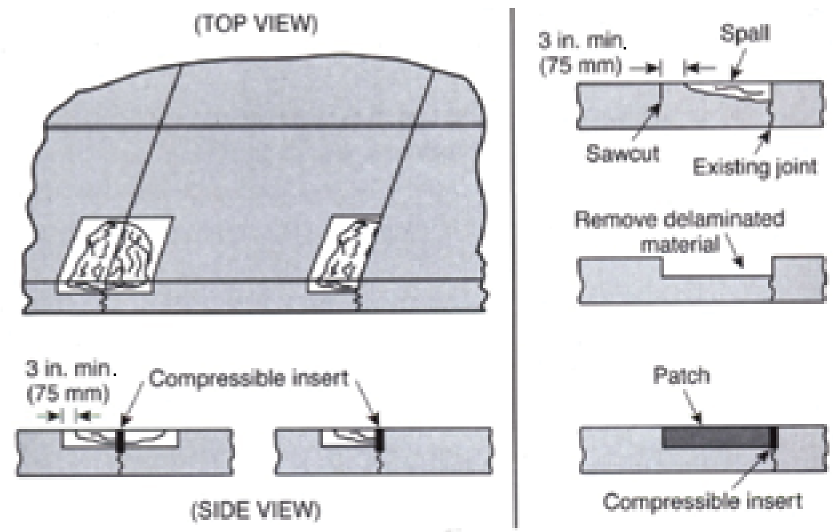
In the case of proprietary mixes, the manufacturer's instructions should be consulted to determine what type of bonding agent, if any, should be used.

8. Construction Considerations

The construction and installation of PDRs involves the following steps:

1. Mark 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 this repair process is illustrated in Figure 5.8, with a detailed description of these steps provided in the following sections. A number of manuals that describe the construction procedures for PDRs are available (Wilson, Smith, and Romine 1999; ACPA 1998; Hoerner et al. 2001; ACPA 2004; ACPA 2006; Frentress and Harrington 2012).



Make vertical saw cut 50 mm (2 in.) min. deep approx 75 mm (3 in.) beyond distressed area. Remove all material at least to the bottom of the 50 mm (2 in.) saw cut, but also at least 13 mm (0.5 in.) into sound material. Use compressible insert to reform joint and bonding agent only if required. Place, compact, finish, and cure patch material. Reseal joint after patch has cured.

Figure 5.8. Partial-depth repair details (ACPA 2006)

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 ball-peen hammer to determine unsound areas; see Figure 5.9. Areas yielding a sharp metallic ringing sound are judged to be acceptable, while those emitting a dull or hollow thud are delaminated or unsound (ACPA 2006). The repair boundaries should then be clearly marked—see Figure 5.10—keeping in mind the minimum repair dimension requirements of 250 mm (10 in.) long and 100 mm (4 in.) wide for cementitious materials; the minimum dimensions for proprietary materials should follow the manufacturer’s recommendations.

It is generally recommended that the boundaries extend at least 75 mm (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 expanded.

Figure 5.9. Sounding deteriorated concrete using hammer (top) and chain (bottom) (Frentress and Harrington 2012)

Figure 5.10. Repair boundaries marked for sawing (Frentress and Harrington 2012)

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 one-third to one-half of the slab thickness. In addition, most cementitious repairs should be at least 50 mm (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 (Types 1 and 3 repairs).
- Mill-and-patch (Types 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. The sawcut should be 50 mm (2 in.) deep; for larger repairs, this may include sawing the concrete in the interior of the repair area in a crisscross pattern to 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 7 kg (15 lb); a jackhammer with a maximum weight of 14 kg (30 lb) may be allowed if damage to sound pavement is avoided (Wilson, Smith, and Romine 1999). The jackhammer is also used to remove the polished vertical sawcut edge by chipping out concrete 50 mm (2 in.) beyond the sawcut to produce an angle between 30 and 60 degrees and create a rough surface, which promotes bonding of the repair material to the existing concrete; see Figures 5.2 and 5.11. Care must be taken to avoid fracturing the sound concrete below or cause shallow chips adjacent to the repair area, which can be difficult to repair.

The advantages of the saw-and-patch procedure are the following (Frentress and Harrington 2012):

- It is cost effective for small projects.
- Most repair crews are familiar with this method.

The drawbacks include the following:

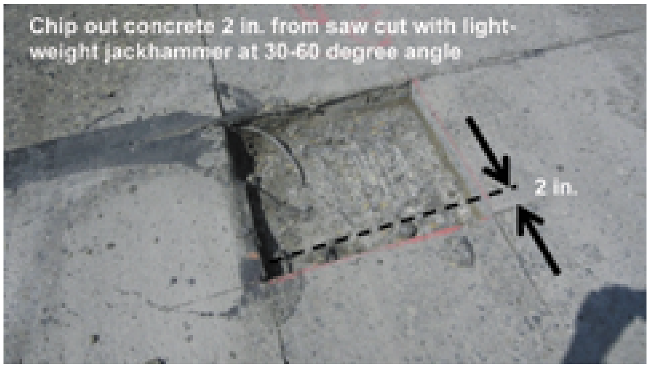


Figure 5.11. Repair area prepared using the saw-and-patch (Type 1) procedure (Frentress and Harrington 2012)

- Water from the sawing process leaves the area saturated, potentially delaying the repair process.
- Spalling can occur outside the sawcut boundaries if not careful during jackhammering operation.
- It is not cost effective for large projects.

Chip-and-Patch Procedure (Type 1 and Type 3 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 7 kg (15 lb); however, a jackhammer up to 14 kg (30 lb) may be allowed if damage to the sound pavement is avoided (Wilson, Smith, and Romine 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, and the chisel point should always be directed toward the inside of the repair (Wilson, Smith, and Romine 1999).

Figure 5.12. Repair area prepared using the chip-and-patch (Types 1 and 3) procedure

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

Cold milling is a quick and efficient method for the removal of deteriorated concrete. Standard milling machines with cutting heads of 300–450 mm (12–18 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 schematically in Figure 5.13, the milling operation can proceed either parallel to a joint or across a joint. Milling parallel to a joint is effective for spalling that occurs along an entire joint, whereas milling parallel to the centerline is effective for smaller, individual spalls. 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-trafficking bond strength testing showed that the cold milling produced the highest degree of bonding as compared to the other methods.

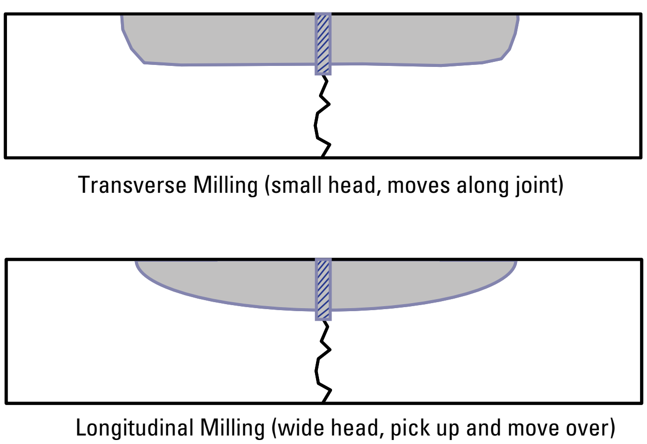


Figure 5.13. Schematic of milling options

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):

- Milling creates a standard size, which may not conform to the needs of the project site.
- 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 76 mm (3 in.) to an outside milled face.
- 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. A tapered edge with a taper angle between 30 and 60 degrees to the bottom of the joint is the preferred shape. Milling with the V-head or rounded head has been used very successfully on transverse joints without any additional sawing and with only minor chipping at the edge of the repair (Frentress and Harrington 2012).

Figure 5.14. "V" shape milling head (top) and milling pattern (bottom) (Frentress and Harrington 2012)

Figure 5.15. Rock saw capable of producing rounded milling (top) and milling pattern (bottom) (Frentress and Harrington 2012)

Vertical Edge Milling

As the name suggests, 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 chipping at the top edge, some highway agencies (such as the Kansas DOT) require sawcuts for all transverse joints repaired with partial-depth milling. The Kansas DOT, however, does not require sawcuts for longitudinal joints unless excessive chipping occurs. Debonding issues have not been reported on PDRs on longitudinal or transverse joints (Frentress and Harrington 2012).

Figure 5.16. Vertical edge mill head (top) and milling pattern (bottom) produced by the vertical edge mill head (Frentress and Harrington 2012)

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

The clean-and-patch procedure is used to perform emergency repairs under adverse conditions (Wilson, Smith, and Romine 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. Such a procedure should only be used if a spall is hazardous to highway users and the climate is so adverse that no other procedure can be used (Wilson, Smith, and Romine 1999).

Step 3: Repair Area Preparation

Following removal of the concrete, the surface of the repair area must be prepared to provide a clean, irregular surface for the development of a good bond between the repair material and the existing slab. Dry sweeping, light sandblasting, and compressed air blasting are normally sufficient for obtaining an adequately clean surface. Sandblasting, as shown in 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. High-pressure water may also be used to remove contaminants, but sandblasting usually produces better results. The compressed air used in the final cleaning must be free of oil, because contamination of the surface will inhibit bonding. This can be checked by placing a cloth over the air compressor nozzle and visually inspecting for any signs of oil.

With any cleaning method, the prepared surface must be checked prior to placing the new material. If a finger rubbed along the prepared surface picks up any loose material (e.g., dust, asphalt, slurry), the surface should be cleaned again. If there is a delay between cleaning and repair placement, the surface may also have to be cleaned again.

Figure 5.17. Sandblasting to remove loose debris

Step 4: Joint Preparation

The most frequent cause of 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 compressive forces created when the slabs expand and insufficient room is provided for the thermal expansion. Failure may also occur when the repair material is allowed to infiltrate the joint or crack opening below the bottom of the repair, resisting slab movement and thereby preventing the joint or crack from functioning. These damaging stresses may also develop along longitudinal joints or at lane-shoulder joints.

Placing a strip of polystyrene, polyethylene, asphalt-impregnated fiberboard or other compressible material between the new concrete and the adjoining slab, as shown in Figure 5.18, will reduce the risk of such failures. Such an insert is typically referred to as a bond breaker or joint reformer. This insert must be placed so that it prevents intrusion of the repair material into the joint opening. Failure to do so can result in the development of compressive stresses at lower depths that will damage the repair. The insert will also guard against damage due to deflection of the joint under traffic. It is recommended that the compressible insert extend 6 mm (0.25 in.) to 25 mm (1 in.) below the deepest removal depth and 76 mm (3 in.) beyond the repair boundaries.

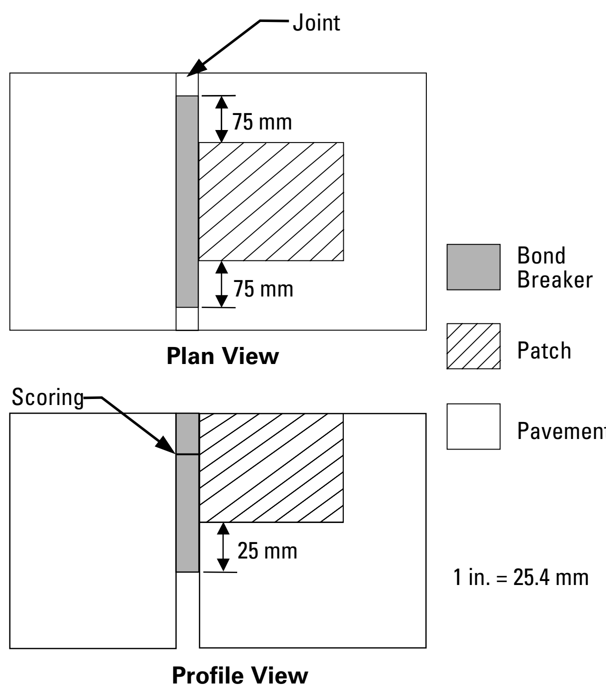


Figure 5.18. Compressible insert placement (Wilson, Smith, and Romine 1999)

Prior to its placement, the insert is typically scored at an appropriate depth. Once the scored bond breaker has been placed in the clean joint, and the repair has been installed and has cured or set, the top strip (above the scoring line) is removed. The removal of the top strip provides a clean surface and a preformed joint reservoir that is ready for the installation of the joint sealant (Wilson, Smith, and Romine 1999).

To avoid cracking, PDRs placed across joints or cracks must reestablish that joint or crack by using compression-absorbing materials or by sawing. Sawing is less commonly used, but when done it must be performed 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 (which is dependent on the materials, curing conditions, curing methods, and so on). Timing is absolutely critical in the sawing operation, because any closing of the joint before sawing will fracture the repair.

Partial-depth repairs placed at the centerline joint directly in contact with the adjacent lane frequently develop spalling because of curling stresses. This can be prevented by placing a polyethylene strip (or other thin bond-breaker material) along the centerline joint just prior to placement of the repair material. If a repair is to be placed along the outer edge of a lane, it must be formed along the lane/shoulder joint. If the repair material is allowed to flow into the shoulder, it may form a "key" that will restrict longitudinal movement and damage 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 5.19 shows PDR performed using a proprietary polymer-based repair material.

Figure 5.19. Partial-depth repair featuring a proprietary polymer-based repair material

Step 5: Bonding Agent Application

Concrete Repair Materials

After the surface of the existing concrete has been cleaned, and just prior to placement of the repair material, the surface may be coated with a bonding agent to ensure complete bonding of the repair material to the surrounding concrete. As previously noted, 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; see Figure 5.20. 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. This may be accomplished by brushing the grout onto the concrete, although spraying may be appropriate for large repair areas. 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 before it comes into contact with the repair material. As previously described,

Figure 5.20. Application of cement grout as bonding agent (Frentress and Harrington 2012)

any bonding material that has set must be removed by sandblasting and then fresh material reapplied before continuing.

The life of the cement grout in the mixing container is about 1 hour. The life of epoxy grouts may be less, depending on the characteristics of the material. In all cases, the manufacturer's preparation and application instructions should be closely followed.

Rapid-Setting Proprietary Repair Materials

Bonding agents for proprietary repair materials should be those recommended by the manufacturer for the placement conditions. Many proprietary repair materials do not require the use of a bonding agent.

Step 6: Repair Material Placement

Repair Material Mixing

The volume of material required for a PDR is usually small (0.02–0.06 m^3 [0.5–2.0 ft^3]). Small drum or paddle-type mixers with capacities of up to 0.06 m^3 (2.0 ft^3) 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 shown in Figure 5.21), ready-mix or mobile concrete trucks can produce the required amount of material in a more efficient manner (Frentress and Harrington 2012).

Figure 5.21. Placement of repair material for long joints using mobile concrete truck (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).

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 4°C (40°F). Additional precautions, such as the use of warm water, insulating covers, and longer cure times, may be required at temperatures below 13°C (55°F). Some polymer concretes and bituminous mixes may be installed under adverse conditions of low temperatures and wet substrates with reasonable success; however, even these materials will perform better when installed under more favorable environmental conditions.

Some epoxy concretes may require that the material be placed in lifts not exceeding 50 mm (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 60°C (140°F) at any time during hardening.

Almost all repair materials require consolidation during placement. Failure to properly consolidate concrete results in poor repair durability, spalling, and rapid deterioration. Consolidation provides a more dense mixture by releasing trapped air from the fresh mix, thereby contributing to the overall performance of the repair. Common methods of achieving consolidation include the following:

- Use of internal vibrators with small heads (less than 25 mm [1 in.] in diameter)
- 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 5.22) and the vibrating screed give the most consistent results. The internal vibrator is often more readily available and is used most often.

Figure 5.22. Consolidation of repair material using internal vibrator (Frentress and Harrington 2012)

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 mix stops settling, air bubbles no longer emerge, and a smooth layer of mortar appears at the surface.

On very small repairs, the mix 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.

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. At least two passes should be made to ensure a smooth repair surface. Partial-depth repairs typically cover only a small percentage of the pavement surface and have little effect on surface friction. Nonetheless, the surface of the repair should be textured to match that of the surrounding slab as much as possible. Figure 5.23 shows completed joint repairs.

Figure 5.23. Completed joint repairs (Frentress and Harrington 2012)

The repair/slab interface 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 start to occur if water at the interface freezes in cold weather (ACPA 2006). Saw-cut runouts extending beyond the repair perimeter at repair corners also can be filled with grout to help prevent moisture penetration that may negatively affect the bond (ACPA 2006). In lieu of grout, the saw-cut 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 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 that may cause the repair to fail prematurely.

Curing Methods

For concrete materials, the most common curing method is to apply a white-pigmented curing compound as soon as water has evaporated from the repair surface; see Figure 5.24. This will reflect radiant heat while allowing the heat of hydration to escape, and it will provide protection for several days. Most curing compounds adhere to the requirements of ASTM C309 or AASHTO M148. Some agencies (including Minnesota and Wisconsin) are using 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 on 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 may be required to help retain heat and ensure strength development. Curing of proprietary repair materials should be conducted in accordance with the manufacturer's recommendations.

Figure 5.24. Repair material curing operation (Frentress and Harrington 2012)

Opening to Traffic

It is important that the PDR attain sufficient strength before it is opened to traffic. As previously indicated, compressive strengths in the range of about 11 to 12.5 MPa (1,600 to 1,800 lbf/in.^{2}) are used by many agencies before the PDR is opened to traffic.

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 often desirable to blend PDRs into a concrete pavement with diamond grinding, which leaves a smooth surface that matches the surrounding pavement; see Figure 5.25.

Figure 5.25. Diamond grinding of PDR (Frentress and Harrington 2012)

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, sandblasting and air blasting both faces of the joint, inserting a closed cell backer rod, and applying the sealer. More detailed information on joint resealing can be found in Chapter 10.

9. 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 found to be unsatisfactory. Some of the common causes of failure include inappropriate use of the repair technique, poor bond, compression failure of the repair (due to failure to reestablish the joint), variability in the effectiveness of repair material, improper use of repair materials, insufficient consolidation, and incompatibility in the thermal expansion between the repair material and the original slab.

This section summarizes key portions of a checklist that has been compiled to facilitate the successful design and construction of good-performing PDRs (FHWA 2005). Although these procedures do not necessarily ensure the long-term performance of a specific repair, the checklist topics are intended to remind both the agency and contractor personnel of those specific design and construction topics that have the potential of influencing the performance of the repair. These checklist items are divided into general categories of preliminary responsibilities, equipment inspections, weather requirements, traffic control, and project inspection responsibilities.

Preliminary Responsibilities

Agency and contractor personnel should collectively conduct a review of the project documentation, project scope and intended construction procedures, and material usage and associate specifications. Such a collective review is intended to minimize any misunderstandings in the field between agency designers, inspectors, and construction personnel. 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 or checked 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.
- 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 one-third to one-half of the slab thickness. Make sure that the criteria for identifying this change are understood.

Document Review

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
- Agency application requirements
- Traffic control plan
- Manufacturer's specific installation instructions for the selected repair material(s)
- Manufacturer's material safety data sheets (MSDS)

Materials Checks

A number of material-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 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 contract documents.
- Material packaging is not damaged so as to prevent proper use (for example, packages are not leaking, torn, or pierced).
- Bonding agent (if required) meets specifications.
- Curing compound (if required) meets specifications.
- Joint/crack reformer material (compressible insert) meets specifications (typically polystyrene foam board, 12 mm [0.5 in.] thick).
- 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 checked or 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 14 kg (30 lbs).

Repair Area Cleaning Equipment

- Verify that the sandblasting unit is adjusted for correct sand 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 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 slump cone, pressure-type air meter, cylinder molds and lids, rod, mallet, ruler, and 3-m [10-ft] straightedge).

Placing and Finishing Equipment

- Verify that a sufficient number of concrete vibrators (25-mm [1-in.] diameter or less) is available on-site and 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 Limitations

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.
- Ensure that air and surface temperature meets manufacturer and contract requirements (typically 4°C [40°F] and above) for concrete placement.
- Ensure that 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 presented in the contract documents.
- Verify that the set-up complies with the Federal or local agency MUTCD or local agency 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.
- Ensure that traffic is not opened to the repaired pavement until the repair material meets strength requirements presented 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 can greatly increase the chances of obtaining PDRs that are durable and perform well. Specifically, the following checklist items (organized by construction activity) summarize the recommended project inspection items.

Repair Removal and Cleaning

- Ensure that the area surrounding the repair is checked for delamination and unsound concrete using steel chain, rod, or hammer.
- Ensure that the boundaries of unsound concrete area(s) are marked at least 75 mm (3 in.) beyond the area of deterioration.
- Verify that concrete is removed by either (1) sawcutting the boundaries and jackhammering interior concrete, or (2) using a cold milling machine.
- Verify that concrete removal extends at least 50 mm (2 in.) deep and does not extend below one-third to one-half of the slab thickness and that load transfer devices are not exposed.
- Verify that, after concrete removal, the repair area is prepared by light sandblasting.
- 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 repairs are left exposed for a period of time longer than that specified in the contract documents.

Repair Preparation

- Ensure that the repair is effectively sandblasted 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. Joint inserts are typically required to extend both below and outside the repair area by 12 mm (0.5 in.).
- When a repair abuts a bituminous shoulder, ensure that a wooden form is used to prevent the repair material from entering the shoulder joint.
- Ensure that the bonding agent (epoxy- or cement-based) is placed on the clean, prepared surface of existing concrete immediately prior to the placement of 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 sandblasting, 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 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 hand-held 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 4°C (40°F). Maintain blanket cover until 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.
- Verify that mixing, placement, and finishing equipment is properly cleaned for the next use.

10. Troubleshooting

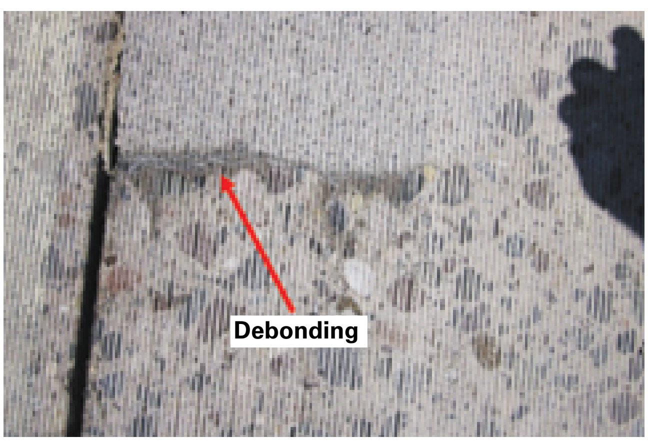
As mentioned previously, poor-performing PDRs are typically attributed to inappropriate use, improper design, or improper construction and placement techniques. Although paying close attention to the checklist items in the previous section attempts to minimize any design or construction-related problems, 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 (FHWA 2005; ACPA 2006; Frentress and Harrington 2012)

Problem	Typical Cause(s)	Typical Solution(s)
Deterioration found to extend beyond the original repair boundaries	This is an unforeseen problem because the true amount of deterioration is not actually known until the concrete is removed.	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., one-third to one-half of the slab thickness), however, an FDR should be placed instead of the PDR.
Repair failures associated with inadequate compression relief provision	Compression relief is not provided, compression relief material is not deep or wide enough to accommodate joint movement below repair, or compression relief does not extend to end of repair area.	The typical solution is to replace the repair, being sure to provide adequate compression relief.
Dowel bar exposed during concrete removal	Concrete deterioration extends deeper than originally believed or improper concrete removal techniques are being used.	An FDR should be used instead of the planned PDR.
Reinforcing steel exposed during concrete removal	If the steel is located in the upper third of the slab, exposing the steel is most likely unavoidable. If steel is exposed below the upper third of the slab, this indicates that either the concrete deterioration extends deeper than originally believed or improper concrete removal techniques are being used.	If the steel is in the upper third of the slab, the steel should be removed to the edges and the placement of the repair placement should continue as planned. If the exposed steel is below the upper third of the slab, however, an FDR should be used instead of the planned PDR.
Repair material flows into joint or crack	When the repair material flows into the joint or crack, it is most commonly the result of one of the following: • The joint insert is not extending far enough into the adjacent joint/crack and below repair. • There is an incorrectly selected insert size for the joint/crack width.	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 mix. If repair material is allowed to infiltrate a crack, it should be removed and replaced.
Shrinkage cracking and surface scaling due to improper finishing and/or curing	These issues are common when the repair material is not cured properly or adequately or if extra water is added to the surface during finishing; see Figure 5.26a on the following page.	Minor surface scaling and shrinkage cracking is typically not a major issue; however, the repair must be monitored for signs of additional deterioration. If excessive scaling and cracking is observed, the repair must be replaced.
Repair cracking or debonding of repair material	Premature cracking or debonding of a PDR is typically attributed to one of the following causes: • The joint insert is not used or is used improperly; see Figure 5.26b on the following page. • The incorrect joint insert size for joint/crack width or insert is not installed correctly. • The repair area was not cleaned immediately prior to grouting/concrete placement. • The grout material dried out before concrete placement; see Figure 5.26c on the following page. • The curing compound is not being applied adequately. • The repair material is susceptible to shrinkage. • The repair is being placed during adverse environmental conditions.	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 and determine the cause of the premature failure, however, in order to avoid repeating the same mistake on future repairs.

(a)

(b)



(c)

Figure 5.26. Repair failures associated with (a) poor compression relief, (b) improper curing/finishing, and (c) improper grout placement resulting in debonding (Frentress and Harrington 2012)

11. 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. Each material will call for different handling and mixing steps. All of the 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. Learning Outcomes

This chapter describes procedures for cast-in-place FDR of existing concrete pavements. Recommended techniques for JPCPs, JRCPs, and CRCPs are discussed. In addition, the use of precast slabs for repair of concrete pavements is presented, along with guidelines for performing utility cut restoration of concrete pavements. Upon successful completion of this chapter, the participant will be able to accomplish the following:

- List benefits of FDRs.
- Describe primary design considerations for FDRs in terms of dimensions, load transfer, and materials.
- Describe recommended construction procedures.
- List advantages of precast concrete repairs.
- Describe procedures for performing utility cut repairs.
- Identify typical construction problems and remedies.

2. Introduction

Concrete pavements exhibiting various types of structural distresses may be candidates for FDRs. When appropriately used, FDRs are an effective means of restoring the rideability and structural integrity of deteriorated concrete pavements and, therefore, extending their service life. Typical distresses that can be addressed using FDRs include transverse cracking, corner breaks, longitudinal cracking, deteriorated joints, blowups, and punchouts. 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 focuses on proper techniques that can be used to design and construct well-performing concrete FDRs on both jointed concrete pavements (JCPs) and CRCPs.

3. Purpose and Project Selection

Full-depth repairs consist of either cast-in-place concrete or precast slabs to repair deterioration that extends through the full thickness of an existing concrete pavement. They can be applied to composite pavements structures (concrete pavements with an asphalt overlay) as well as to bare concrete pavements. Because FDR involves complete removal and replacement of deteriorated areas, this technique can be used to address a wide variety of concrete pavement distresses.

JCP

Table 6.1 provides a summary of the JCP distresses and severity levels that can be successfully remedied using FDRs. In determining the need for FDRs, consideration must be given to the extent of distress within a project. 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. Concrete pavements exhibiting severe structural distresses over an entire project are more suited for a structural overlay or reconstruction.

Table 6.1. JCP Distresses Addressed by FDRs (Hoerner et al. 2001).

Distress Type	Severity Levels That Require 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 one-half of the pavement slab.

[2] If the pavement has a severe material problem (such as D-cracking or reactive aggregate), FDRs may only provide temporary relief from roughness caused by spalling. Continued deterioration of the original pavement is likely to result in redevelopment of spalling and roughness.

NOTE: Highways with low traffic volumes may not require repair at the recommended severity level.

CRCP

Table 6.2 provides a summary of the CRCP distresses and severity levels that can be successfully remedied using FDRs. Punchouts are the most common structural distress on CRCP that are addressed with FDRs.

Table 6.2. Candidate CRCP Distresses Addressed by FDRs (Hoerner et al. 2001).

Distress Type	Severity Levels That RequireFDR
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 one-half of the pavement slab.

[3] If the pavement has a severe material problem (such as D-cracking or reactive aggregate), FDRs may only provide temporary relief from roughness caused by spalling. Continued deterioration of the original pavement is likely to result in redevelopment of spalling and roughness.

NOTE: Highways with low traffic volumes may not require repair at the recommended severity level.

4. Limitations and Effectiveness

Although FDRs can be designed and constructed to provide good long-term performance, the performance of FDRs is very much dependent on their appropriate application and the use of effective design and construction practices. Many performance problems can be traced back to inadequate design (particularly poor load transfer design), poor construction quality, or the placement of FDRs on pavements that are too far deteriorated. Thus, project selection is very important to obtain the desired performance. Important points for consideration in selecting this repair technique 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 may only provide temporary relief from roughness caused by spalling. Continued deterioration of the original pavement is likely to result in redevelopment of spalling and roughness.
- Additional joints introduced by FDRs add to the pavement roughness. Diamond grinding should be considered after the repairs are made to produce a smooth-riding surface.
- Nondeteriorated transverse cracks in JPCP may be repaired by retrofitting dowel bars.

Again, the effectiveness of FDRs depends strongly on the installation of the repairs at the appropriate time in the life of the pavement and on the proper design and installation of the FDR (particularly the load transfer system).

5. Materials and Design Considerations

This section presents the materials and design considerations for FDRs of JCP, as well as special 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 pavement to traffic.

Selecting Repair Locations and Boundaries

JCP

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. This is accomplished by a trained crew performing a condition survey for the entire project in all lanes. A follow-up survey should be performed immediately prior to construction to verify the quantity of repair work needed because additional pavement deterioration may have occurred since the previous pavement inspection.

Jointed reinforced concrete pavement often exhibits deteriorated joints and mid-panel cracks that deteriorate under repeated heavy traffic loadings. Additionally, some intermediate cracks deteriorate because of "frozen" or locked doweled joints, which force the cracks to absorb the movements the doweled joints are designed to accommodate. These cracks soon lose their aggregate interlock under repeated heavy traffic loadings. Some projects will actually have joints with very little deterioration but one or more intermediate cracks in each slab opened wide and essentially acting as joints.

On JPCP, all structural cracks are candidates for FDR. The rate at which the cracks deteriorate depends on traffic, climate, and pavement structure. The types of JCP distresses that can be successfully addressed through FDRs are presented in Table 6.1. Each agency should examine these recommendations and modify them as needed to develop an approach that more closely reflects local conditions.

Sizing the Repair

After the repair locations are identified, the boundaries of each repair must be determined. This is typically performed by the project engineer at, or just before, construction. It is important that the repair boundaries extend to include all of the significant deterioration in the slab and underlying layers (including the subgrade). The extent of deterioration beneath the slab surface may be identified through coring and deflection studies. Where the pavement has an MRD (such as D-cracking), the deterioration at the bottom may extend as much as 0.9 m (3 ft) or more beyond the visible boundaries of deterioration at the surface; see Figure 6.1.

Engineering judgment is required in selecting repair boundaries, and it should be based on performance history, production efficiencies, and economics. The following guidelines are recommended for establishing repair boundaries on JCP (Correa and Wong 2003; ACPA 2006):

- **Repair Length**
  - For doweled repairs, a minimum repair length of 1.8 m (6 ft) (in the longitudinal direction) is recommended to minimize rocking, pumping, and breakup of the slab (Correa and Wong 2003). It should be noted, however, that a few highway agencies use shorter repair lengths (on the order of 1.2 m [4 ft]) with good results.
  - For nondoweled repairs, which are recommended only for pavements exposed to low truck traffic volumes, the recommended minimum repair length is 1.8–3.0 m (6–10 ft) (in the longitudinal direction).

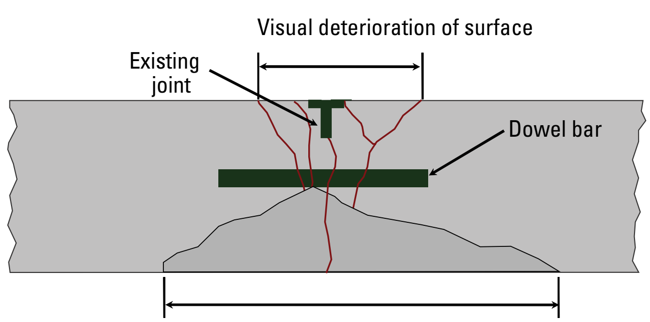


Figure 6.1. Illustration of potential extent of deterioration beneath a joint

- When the FDR exceeds about 4.6 m (15 ft), the placement of intermediate transverse joints is recommended.

- **Repair Width**—Although partial-lane-width slab replacements are used by a few agencies, full-lane-width repairs are generally recommended because boundaries are well defined and the repair is more stable.

- **General Considerations**
  - Saw full depth a minimum of 0.6 m (2 ft) from any joints.
  - Use straightline sawcuts, forming rectangles in line with the jointing pattern.
  - Extend the repair boundary to the joint if the boundary is within 1.8 m (6 ft) of an existing joint.
  - Make one large repair if the individual repairs are 2.4–3.6 m (8–12 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 guidelines for determining the maximum distance between FDRs to maintain cost effectiveness.

Figure 6.2 illustrates an example of how to select repair boundaries when multiple distresses of different severities are present. Note that not all distresses require an FDR.

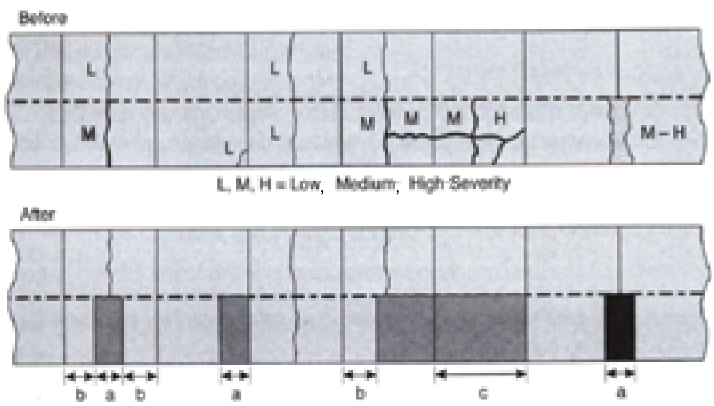
Large Area Removal and Replacement

In some situations, the existing distress is so extensive that the repair of every individual deteriorated area within a short distance (e.g., 3–9 m [10–30 ft]) is either very expensive or impractical. In this case, it is more cost effective and production efficient to replace the entire pavement panel (or series of panels, if all are significantly deteriorated). For these cases, a separate pay item should be set up to designate removal of an entire slab, the length of which could vary depending on the joint spacing used on the project. Generally referred to as "slab replacement," this pay item will have a lower unit cost than that of several small repairs. Some agencies directly consider this in their governing specifications because they have established FDR categories by size of repair. Intermediate transverse joints are recommended when long FDRs exceed 4.6 m (15 ft).

Table 6.3. Maximum Distance between FDRs to Maintain Cost Effectiveness (Correa and Wong 2003; ACPA 2006).

Pavement Thickness, mm (in.)	Patch or Lane Width, m (ft)	
	3.3 (11)	**3.6 (12)**
150 (6)	4.9 (16)	4.6 (15)
175 (7)	4.3 (14)	4.0 (13)
200 (8)	3.6 (12)	3.3 (11)
225 (9)	3.3 (11)	3.0 (10)
250 (10)	3.0 (10)	2.7 (9)
275 (11)	2.7 (9)	2.4 (8)
300 (12)	2.4 (8)	2.4 (8)

Note: if patches are closer than the distances listed, they should be combined into one repair.



NOTES
a – Minimum length is 1.8 m (6 ft).
b – Check distance between patches and nearby joints.
c – Replace the entire slab if there are multiple intersecting cracks.

Figure 6.2. Example of selection of FDR boundaries on JCP (ACPA 2006)

Multiple-Lane Repairs

On multiple-lane highways, 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 other lanes. When two or more adjacent lanes contain distress, one lane is generally repaired at a time so that traffic flow can be maintained.

Matching joints in adjacent lanes are generally not necessary, as long as a fiberboard has been placed along the longitudinal joint to separate the lanes. 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 to avoid small offsets and to maintain continuity. If blowups occur during the repair of one lane, it may be necessary to perform the work at night or delay it until cooler weather prevails.

CRCP

The types of CRCP distresses that can be addressed through FDRs are identified in Table 6.2. Again, these recommendations should be evaluated by each agency and modified for use under their local conditions.

Sizing the Repair

As illustrated in Figure 6.3, subsurface deterioration accompanying structural distresses of CRCP can be quite extensive. Subbase deterioration is particularly prevalent near punchouts and wherever there is settlement or faulting along the longitudinal lane joint. The results of coring and deflection studies provide information on the extent of deterioration beneath the slab surface, and such studies are recommended on projects of any magnitude.

As described later, FDRs in CRCP generally include the provision of new longitudinal steel bars to maintain the continuity of the reinforcement through the repair area. These new bars can be affixed to the existing steel by tying, by welding, or through mechanical connections. The method of attachment influences the selection of the repair boundaries, as listed below (Gagnon, Zollinger, and Tayabji 1998; Gulden 2013):

- A minimum repair length of 1.2–1.8 m (4–6 ft) is generally recommended, with the shorter length recommended if the steel is mechanically connected or welded and the longer length recommended if the reinforcing steel is tied.
- The repair boundaries should not be closer than 460 mm (18 in.) to adjacent nondeteriorated cracks; however, if cracks are very closely spaced, it may be necessary to place the repair as close as 150 mm (6 in.) to an existing tight transverse crack.
- Full-lane-width repairs are generally recommended, although a half-lane width (1.8 m [6 ft]) may be used when all of the pavement distress is contained within that width.

These criteria are given to provide adequate lap length and cleanout, as well as to minimize repair rocking, pumping, and breakup. Figure 6.4 illustrates these construction recommendations.

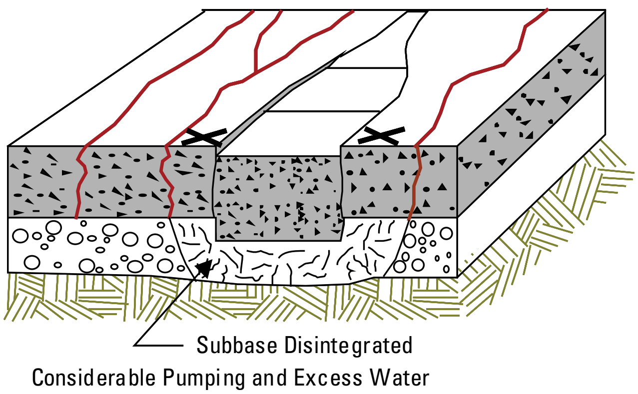


Figure 6.3. Schematic illustration (left) and photograph (right) of CRCP punchout distress

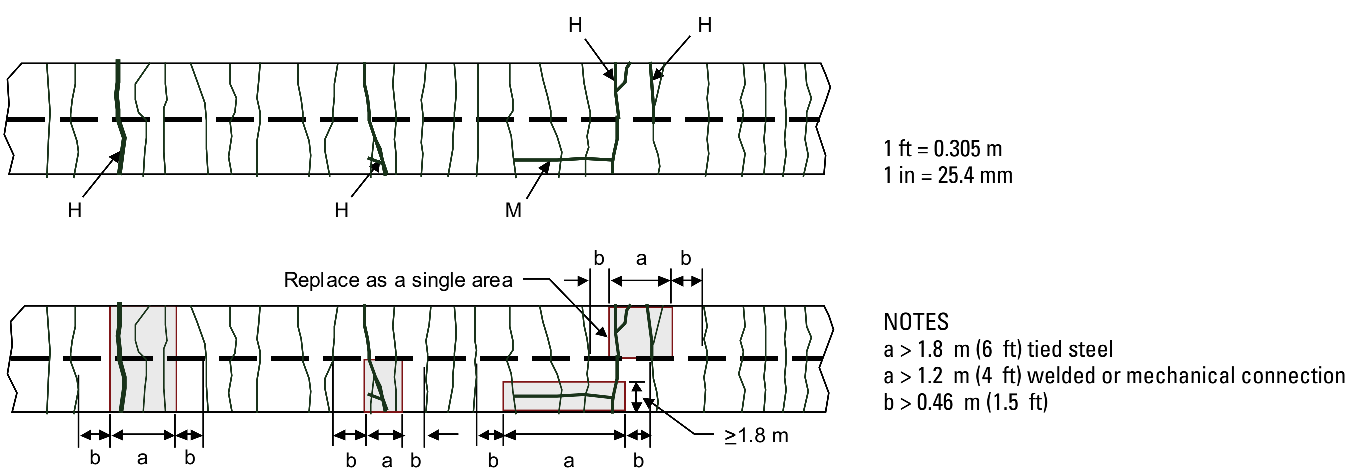


Figure 6.4. Example of repair recommendations for a CRCP

Multiple-Lane Repair Considerations

If a distress such as a wide crack with ruptured steel occurs across all lanes, special considerations are necessary because of the potential for the following:

- Blowups in the adjacent lane.
- Crushing of the new repair during the first few hours of curing by the expanding CRCP.
- 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 avoid being crushed by the expanding CRCP slab. In addition, it is recommended that the lane with the lowest truck traffic be repaired first.

Selecting Repair Materials

The repair material 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 less than 1 hour to 24 hours or more), using either conventional concrete or a proprietary material. Faster-setting mixes, however, generally have higher costs and special handling requirements. A good rule of thumb in selecting the material for concrete pavement repair projects is to use the least exotic (i.e., most conventional) material that will meet the opening requirements.

The most widely used repair materials for FDRs are conventional concrete mixtures. Different constituent materials can be used to meet a range of opening times, as shown in Table 6.4. 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, Cable, and Taylor 2009). In addition, the long-term durability of these mixtures is also potentially at risk. Guidelines are available that summarize the state of practice for high early strength concrete repairs, including the identification of material properties impacting their performance, the selection of materials and mixture design properties for high early strength concrete, and the identification of performance-related tests of fresh and hardened concrete (Van Dam et al. 2005).

Laboratory testing of proposed repair materials (using the aggregates that will be used in the project mix) must be conducted to ensure that the opening requirements are met. To ensure adequate durability of hardened concrete, the concrete mix should have between 4.5 and 7.5 percent entrained air, depending on the maximum coarse aggregate size and the climate (ACPA 1995). The slump should be between 50 and 100 mm (2 to 4 in.) for overall workability and finishability.

In addition to conventional concrete, a number of specialty cements and proprietary materials have also been used successfully in FDRs. For example, California has used a calcium-sulfoaluminate-based cement for highway panel replacement since 1994; the product gains structural strength in approximately 1 hour at standard placement temperatures and exhibits good durability and low shrinkage (Ramseyer and Perez 2009).

Anticipated climatic conditions should 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 temperature gradient. When the ambient temperature is in excess of 32°C (90°F), it may be very difficult to place some of the rapid-setting materials because they harden 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 time of placement have been linked to premature failures (Yu, Mallela, and Darter 2006).

Table 6.4. Common Ranges of Constituent Materials for High Early Strength Concrete (compiled from Whiting et al. [1994] and Van Dam et al. [2005])

Mix Characteristic	4- to 6-Hour Concrete	6- to 8-Hour Concrete	20- to 24-Hour Concrete
Cement Type	I or III	I or III	I or III
Cement Content	385–530 kg/m^3 (650–895 lb/yd^3)	425–525 kg/m^3 (715–885 lb/yd^3)	400–475 kg/m^3 (675–800 lb/yd^3)
w/c ratio	0.38–0.40	0.36–0.40	0.40–0.43
Accelerator	Yes	Yes	None to Yes

Load Transfer Design in JCP

Transverse joint load transfer design is one of the most critical factors influencing the performance of FDRs. 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 of 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.

The use of smooth dowel bars is highly recommended for all FDRs because they provide better performance (less faulting, rocking, and other joint-related distresses) than other means of load transfer. The only exception may be residential streets that carry fewer than 100 trucks or buses per day, for which aggregate interlock joints may be sufficient. Table 6.5 summarizes dowel bar-related design details for different pavement thickness ranges (ACPA 2006).

Round, solid steel dowels conforming to AASHTO M31 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 M284 (ASTM A775) or ASTM A934. Although the AASHTO M254 specification requires coating thicknesses of 7 ± 2 mils, recent recommendations call for an average epoxy coating thickness of 10 mils or more (Snyder 2011).

Highway agencies vary in the number of dowel bars included in the FDR designs. Some specifications require three, four, or five dowels per wheelpath, whereas others require dowels across the entire lane

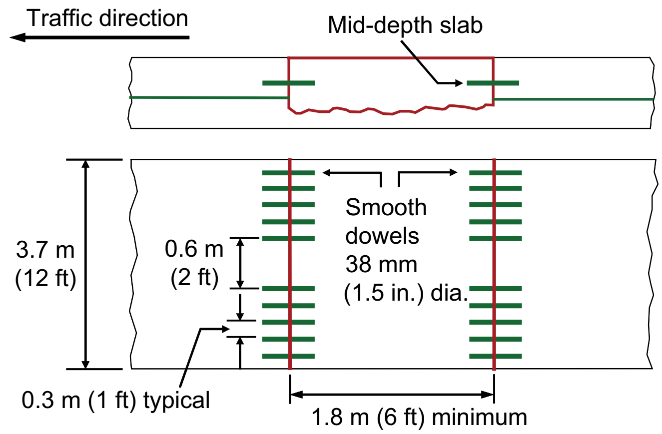


Figure 6.5. Example of dowel bar layout

width (ACPA 2006). Figure 6.5 shows one recommended layout, with five dowel bars clustered in the wheelpath to provide effective load transfer.

Longitudinal Joint Considerations

When the repair length is less than 4.5 m (15 ft), a bondbreaker board is typically placed along the length of the longitudinal joint to isolate it from the adjacent slab. The bondbreaker, commonly a 5-mm (0.2-in.) thick fiberboard, should be configured for the full length of the repair and extend to the top height of the slab. Tiebars are generally not required for these shorter repairs.

When the FDR is longer than 4.5 m (15 ft), tiebars are recommended for installation in the face of the adjacent slab at the longitudinal joint (ACPA 2006). Tiebars should also be provided at the lane-shoulder joint if the existing shoulder is concrete. The tiebars should be epoxy coated and installed at mid-depth of the pavement slab; they are 13–16 mm (0.5–0.625 in.) in diameter and are typically spaced on 762- to 914-mm (30- to 36-in.) intervals.

Table 6.5. Dowel Requirements for FDRs in JCPs (ACPA 2006)

Pavement Thickness, mm (in.)	Dowel Diameter, mm (in.)	Drilled Hole Diameter, mm (in.)		Min. Length, mm (in.)	Spacing, mm (in.)
		Grout	Epoxy		
≤150 (≤6)	19 (0.75)	24 (0.95)	21 (0.83)	350 (14)	300 (12)
<200 (6.5–8)	25 (1.0)	20 (1.2)	27 (1.08)		
200–240 (8–9.5)	32 (1.25)	37 (1.45)	34 (1.33)		
250+ (10+)	38 (1.5)	43 (1.7)	40 (1.58)		

Restoring Reinforcing Steel in CRCP

As previously mentioned, it is important on 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 it can be affixed to the existing reinforcing through a number of methods (ACPA 1995; Gagnon, Zollinger, and Tayabji 1998; Gulden 2013):

- **Tied Splice**—In this method, tie wires are used to attach the new steel to the existing reinforcement. Longer lengths of the existing steel must be exposed (commonly up to 610 mm [24 in.]), and the tied splices should be lapped 406 mm (16 in.) to 508 mm (20 in.), depending on the bar diameter. This is the most common method of attaching the new steel to the existing steel.
- **Welded Splice**—This method uses a 6-mm (0.25-in.) continuous weld made either 100 mm (4 in.) long on both sides or 200 mm (8 in.) long on one side. Because of the weld length, shorter lengths of steel are required to be exposed (typically about 200 mm [8 in.]). To avoid potential buckling of bars on hot days, the reinforcement must be lapped at the center of the repair. This allows movement of the CRCP ends without damaging the steel. Although this procedure has been used successfully, some problems have resulted from poor workmanship or weldability of the steel.
- **Mechanical Connection**—This procedure uses special couplers to join the two pieces of steel and, similar to the welded splices, requires that about 200 mm (8 in.) of existing steel reinforcement be exposed. The mechanical connection has a lap length of about 50 mm (2 in.). Again, it is recommended that the reinforcement be lapped at the center of the repair to allow movement of the CRCP. Some agencies have reported performance issues with mechanical connections.

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; it is desired that the unsupported length not exceed 1.2 m (4 ft). For all connection types, a 50-mm (2-in.) clearance is required between the end of the lap and the existing pavement, and a minimum of a 65-mm (2.5-in.) cover should be provided over the reinforcing steel. And, as noted previously, different minimum repair lengths are recommended based on the type of splicing technique used. Figure 6.6 summarizes the sawing and repair details for CRCP repairs.

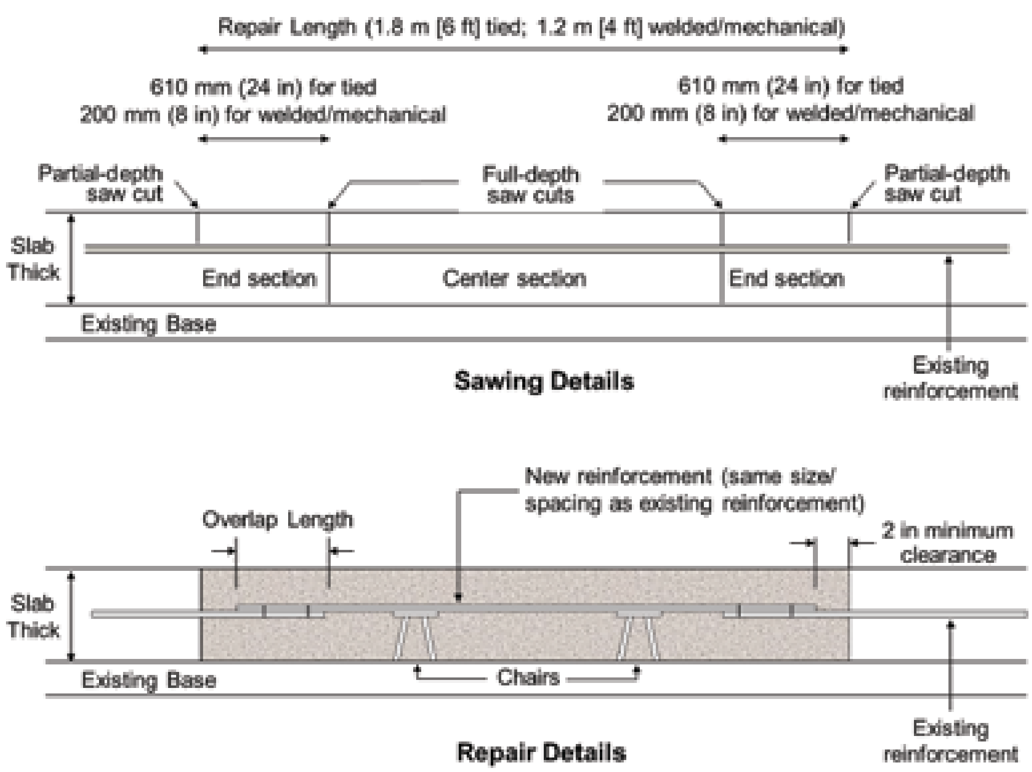


Figure 6.6. Summary of sawing and repair details for CRCP repairs

In addition to longitudinal steel, a few agencies also place transverse steel in the repair to guard against longitudinal cracking and punchouts (Gulden 2013). These transverse bars are tied to the longitudinal bars and are typically placed on 300-mm (12-in.) spacings. As with FDRs on jointed pavements, FDRs on CRCP should be tied to the adjacent slabs when the repair length exceeds about 4.6 m (15 ft).

Several highway agencies have adopted slightly alternative approaches to the conventional practice of carrying the steel through the repair area. These include the following:

- The Texas DOT installs tiebars in drilled holes in both exposed transverse joint faces in the existing CRCP slab. The grouted tiebars are then connected to new longitudinal bars that are carried through the repair area. This reduces the need for two sawcuts, allows for the restoration of the base within the repair area, and significantly reduces the labor requirements and overall installation time (Tayabji 2011). This has been a specification item in Texas since 1994. Experience has shown that the proper installation of the tiebars is critical to the performance of these types of repairs, largely because they (along with the base) are relied upon to provide the load transfer at the joints (the smooth joint face has limited aggregate interlock load transfer capabilities) (Gulden 2013).
- The South Carolina DOT 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, with no attempts to maintain the continuity of the longitudinal steel (Tayabji 2011). These are working well in a number of projects in South Carolina. Details of the South Carolina DOT method for panel lengths of 1.8–3.6 m (6–12 ft) are shown in Figure 6.7.

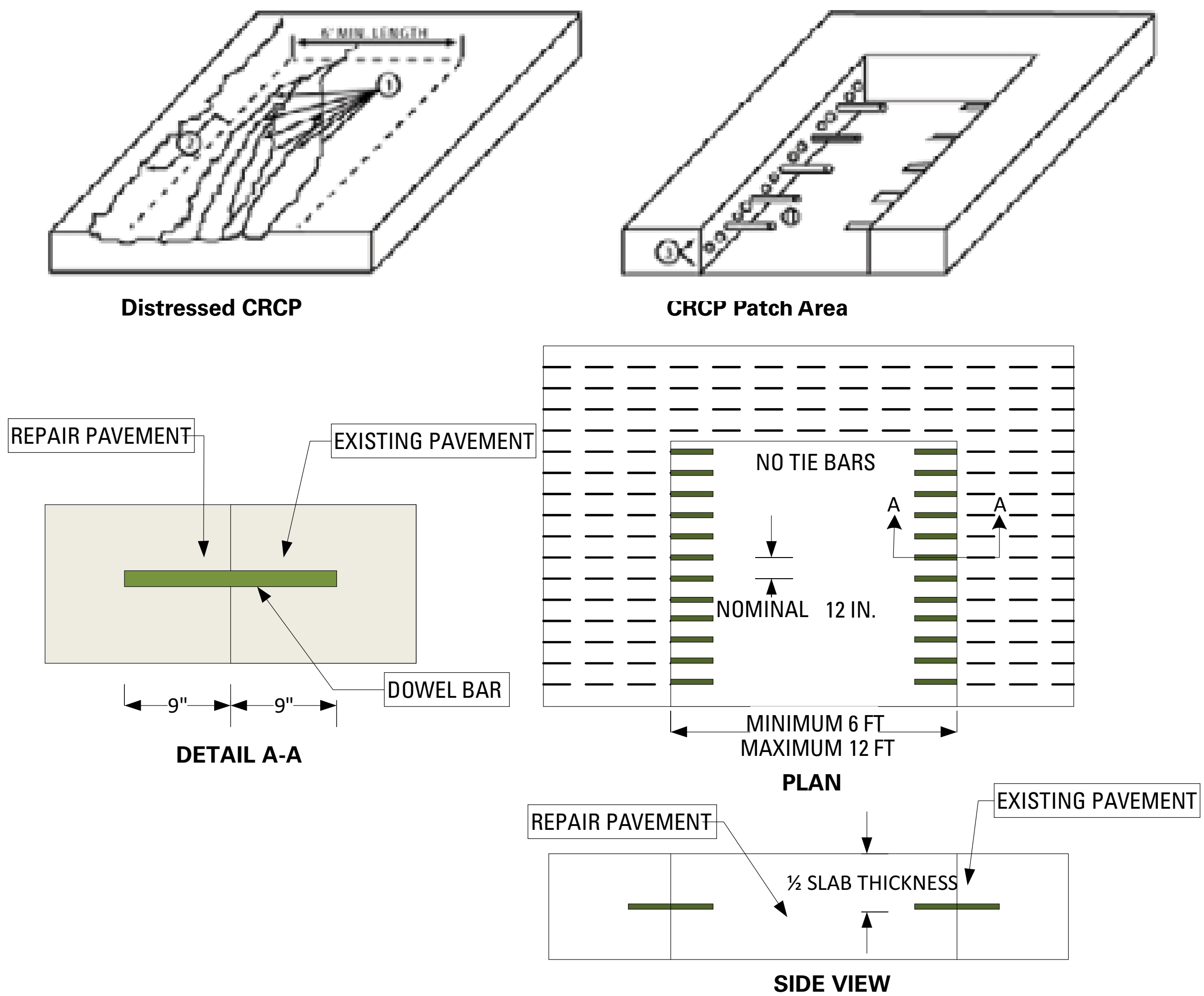


Figure 6.7. Details of the South Carolina jointed FDR of CRCP (Tayabji 2011)

Opening to Traffic

There is not a clear consensus on what strength is required for opening fast-track concrete pavements to traffic. Factors such as the type of application (FDRs with 1.8-m [6-ft] slabs compared to a localized reconstruction pavement with 4.6-m [15-ft] slabs), expected traffic loadings, and expected edge loading conditions may all affect the required minimum strength.

A review of state highway practices suggests a range of values are often specified for the opening of FDRs, from compressive strength values of 13.8–20.7 MPa (2,000–3,000 lbf/in.^{2}) and from flexural strength values of 2.0–2.8 MPa (290–400 lbf/in.^{2}) (third-point loading) (Van Dam et al. 2005). Those limits, however, are conservative values based on the ultimate strength of the 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, Cable, and Taylor 2009). Moreover, thicker repairs have a greater load carrying capacity, so they would require a lower strength. This is reflected in the opening requirements suggested by the ACPA in Table 6.6 for various sizes and thicknesses of FDRs (ACPA 2006).

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

- 13.8 MPa (2,000 lbf/in.^{2}) for concrete pavement slabs containing 38-mm (1.5-in.) dowel bars.
- 17.2 MPa (2,500 lbf/in.^{2}) for concrete pavement slabs containing 32-mm (1.25-in.) dowel bars.

As such, consideration should be given to the cracking criteria provided in Table 6.6 and the dowel bearing stress criteria given above, with the highest value controlling the opening time.

A few highway agencies, including Georgia, have employed opening strengths lower than 13.8 MPa (2,000 lbf/in.^{2}) without any adverse effect of performance. Accelerated load testing of 229-mm (9-in.) thick slabs on a stiff base course supports the use of lower values (in the range of 11.0 MPa [1,600 lbf/in.^{2}]) (Tia and Kumara 2005). Agencies are encouraged to explore the appropriateness of using lower opening strength values on their FDR projects. The use of maturity meters or pulse-velocity devices for monitoring the in-place concrete strength is recommended as part of that process (ACPA 1995).

The HIPERPAV computer software program (www.hiperpav.com) may be helpful in identifying the conditions under which special care is needed to avoid random cracking of FDRs. Developed under contract with the FHWA, the software takes key environmental, structural design, mix design, and construction inputs, and it generates a graph showing the development of concrete strength and stress over the first 72 hours after placement. 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, HIPERPAV III, was released in 2009 and features an improved software interface and enhanced modeling capabilities (Xu et al. 2009).

Table 6.6. Minimum Opening Strengths for FDRs (ACPA 2006)

Slab Thickness, mm (in.)	Strength for Opening to Traffic, MPa (lbf/in.^{2})			
	Repair Length <3 m (10 ft)		Slab Replacements	
	Compressive	3rd-Point Flexural	Compressive	3rd-Point Flexural
150 (6.0)	20.7 (3000)	3.4 (490)	24.8 (3600)	3.7 (540)
175 (7.0)	16.5 (2400)	2.6 (370)	18.6 (2700)	2.8 (410)
200 (8.0)	14.8 (2150)	2.3 (340)	14.8 (2150)	2.3 (340)
225 (9.0)	13.8 (2000)	1.9 (275)	13.8 (2000)	2.1 (300)
250+ (10.0+)	13.8 (2000)	1.7 (250)	13.8 (2000)	2.1 (300)

FDR of Composite Pavements

Full-depth repairs may also be used to address deterioration in existing composite pavements (asphalt overlays of concrete pavements). Typically, these 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.

6. Construction

The construction and installation of FDRs involves the following steps:

- Concrete sawing
- Concrete removal
- Repair area preparation
- Restoration of load transfer in JCP or reinforcing steel in CRCP
- Treatment of longitudinal joint
- Concrete placement and finishing
- Curing
- Diamond grinding (optional)
- Joint sealing on JCP

Each of these steps is described for both JCP and CRCP; further guidance can be found in other publications (ACPA 1995; ACPA 2006).

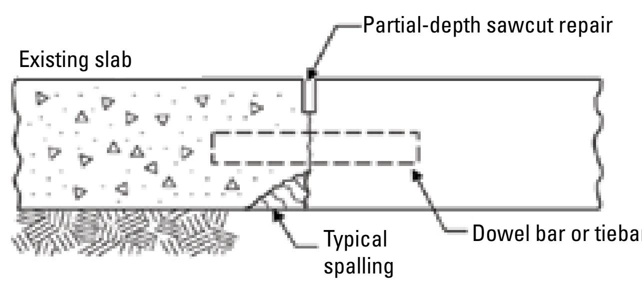
Step 1: Concrete Sawing

JCP

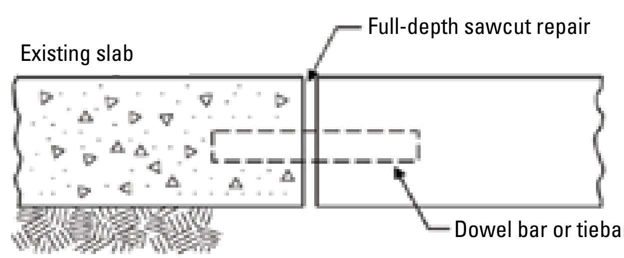
Two types of sawed transverse joints have been used for FDR: rough-faced and smooth-faced (shown in Figure 6.8). The smooth-faced joint, in which sawcuts are full depth, is recommended. Although smooth-faced joints will not contribute to aggregate interlock load transfer, they are easier to construct and do not contribute to secondary deterioration. Dowels are recommended for all smooth-faced joints.

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 repairs, there is no need to provide reinforcing steel within the repair. Reinforcing steel may only be required within repairs that are more than 4.6 m (15 ft) long, but a better approach is to install an intermediate joint any time the length of the repair exceeds 4.6 m (15 ft).

Repair boundaries should be sawed full depth with diamond saw blades. On hot days, it may not be possible to make such cuts without first making a wide, pressure-relief cut within 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. The wheel sawcuts produce a ragged edge that promotes excessive spalling along the



(a) Rough-faced type joint



(b) Smooth-faced type joint

Figure 6.8. Types of sawed transverse joints: (a) rough-faced; (b) smooth-faced

joint. Hence, if wheel sawcuts are made, diamond sawcuts must be made just outside the wheel sawcuts. To prevent damage to the subbase, the wheel saw must not be allowed to penetrate more than 13 mm (0.5 in.) into the subbase. The longitudinal joint (and concrete shoulder, if it exists) should be cut full depth using diamond-bladed saws.

Figure 6.9 illustrates the sawing pattern for JCP. The slanted cut shown in the bottom figure is a pressure-relief cut that may be necessary to prevent spalling of the adjacent concrete during concrete removal. This cut should be made when the sawed joint closes up (because of hot weather) before the concrete can be removed. Alternatively, a contractor may elect to saw at night during cooler temperatures (ACPA 1995).

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 actually installing the repairs. 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 2 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 up against the existing shoulder and fill the area with concrete repair material.

CRCP

For CRCP, two sets of sawcuts are required to provide a rough joint face at repair boundaries. To ensure good repair performance, the joint faces must be rough and vertical, and all underlying deteriorated material must be removed and replaced with concrete. The rough joint faces and continuity of reinforcement (which is re-established during the repair process) provide the load transfer across the repair joints.

The rough joint faces are created by first making a partial-depth cut at each end of the repair area, to a depth of about one-fourth to one-third of the slab thickness, as shown previously in Figure 6.6. The partial-depth sawcuts should be located at least 460 mm (18 in.) from the nearest tight transverse crack. They 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 lap length required (which is dictated by the type of method used to splice the reinforcing steel).

After the partial-depth cuts, two full-depth sawcuts are then made at a specified distance in from the partial-depth cuts (see Figure 6.6). The recommended distance is 610 mm (24 in.) for tied laps and 200 mm (8 in.) for mechanical connections or welded laps.

As alluded to previously, 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 (ACPA 1995; Tayabji 2011). Instead, holes are drilled in the faces of the concrete slab and new reinforcing steel (either

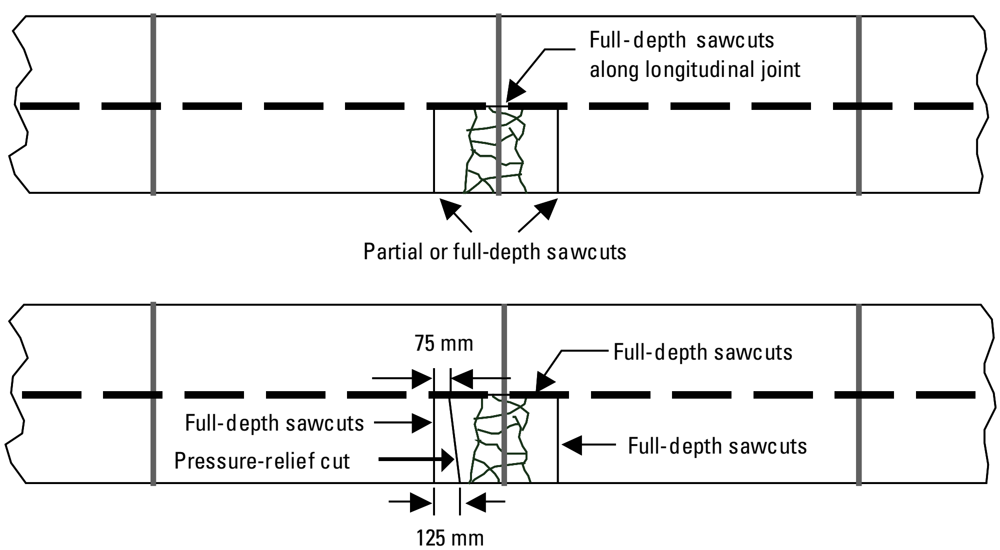


Figure 6.9. Sawcut locations for FDR of JCP

tiebars 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 tiebars and through the repair, while the South Carolina method adds no new steel. These procedures reduce the amount of hand chipping and greatly increase productivity.

Step 2: Concrete Removal

JCP

Two methods have been used to remove deteriorated concrete from the repair area (Darter, Barenberg, and Yrjanson 1985; ACPA 1995):

- **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; see Figure 6.10.
- **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 it is 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 allowed 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 6.7. The lift-out method is generally recommended in order to minimize disturbance to the base, which is critical to good performance. This method generally provides the best results and the highest production rates for the same or lower cost.

Figure 6.10. Lift-out method of slab removal

Table 6.7. 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.	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 with teeth can rapidly remove the broken concrete and load it onto trucks.	This method usually greatly disturbs the subbase/subgrade, requiring either replacement of subbase material or filling with concrete. It also has some potential to damage the adjacent slab.

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 underbreaking of the concrete on the bottom of the 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.

CRCP

The procedure for removing concrete from the center section (between the inner full-depth sawcuts) of the repair area is the same as for JCP. The deteriorated concrete must be carefully removed to avoid damaging the reinforcement and to prevent spalling concrete at the bottom of the joint (beneath the sawcut). This can be accomplished by using jackhammers, prying bars, picks, and other hand tools. 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. Any significant underbreaking that occurs will require a new partial-depth sawcut outside of 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 percent 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 6.11 shows a photo of a CRCP repair with the reinforcing steel exposed.

Figure 6.11. Prepared CRCP repair area with exposed reinforcing steel (Gulden 2013).

Step 3: Repair Area Preparation

All subbase and subgrade materials that have been disturbed or that are loose should be removed and replaced either with similar material or with concrete. If excessive moisture is present in the repair area, as determined by the project engineer, it should be dried out before placing new material. Placement of a lateral drain (consisting of a trench cut through the shoulder and the placement of a lateral pipe or open-graded crushed stone) may be necessary where there is standing water in the repair area.

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 percent of the total concrete repair area for this estimate.

It is very difficult to adequately compact granular material in a confined repair area. Hand vibrators generally do not produce adequate compaction to prevent settlement of the repair. Consequently, replacing the damaged portion of a disturbed subbase with concrete is often the best alternative.

Step 4: Restoration of Load Transfer in JCP or Reinforcing Steel in CRCP

Restoring Load Transfer in JCP

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 of the exposed face of the existing slab. Tractor-mounted gang drills can be used to drill several holes simultaneously while maintaining proper horizontal and vertical alignment at the same time; see Figure 6.12. The existing concrete joint face should be inspected prior to drilling to ensure that it is sound and without any signs of deterioration. Some pavements with large and particularly hard aggregates may also tend to spall out 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 6 mm (0.25 in.) larger than the dowel diameter

Figure 6.12. Example of gang drill used for dowel bar installation

(ACPA 2006). A plastic grout mixture provides better support for dowels than a very fluid mixture. If an epoxy mortar is used, smaller hole diameters (2 mm [0.08 in.]) are required because this type of material can often ooze out through small gaps and is somewhat more flexible than cement grout. Figure 6.13 shows a schematic of the dowel bar anchoring 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). The following procedure is recommended for anchoring dowel bars (Snyder et al. 1989; ACPA 1995):

1. Remove debris and dust from the dowel holes by blowing them out with air. If the holes are wet, they should be allowed to dry before installing dowels. Check dowel holes for cleanliness before proceeding.
2. 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.
3. 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.
4. Optionally, place a grout retention disk (a thin, donut-shaped plastic disk) over the dowel and against the slab face, as illustrated in Figure 6.13. This 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).

Some agencies are exploring alternative methods of anchoring dowel bars, including the use of grout bags or grout capsules that contain a cementitious, non-shrink grout material that is pre-mixed dry and encapsulated in a water permeable wrapping. The grout capsule is saturated in water, and then placed in a dry clean hole and the dowel bar is inserted, which breaks the capsule and distributes the fast-setting grout material around the dowel bar.

After placement, the protruding end of the dowels should be lightly greased to facilitate movement. If steel reinforcement is to be provided within the repair (typically in longer repairs), the steel should be placed between concrete lifts with a minimum of a 75-mm (3-in.) cover and 65-mm (2.5-in.) edge clearance.

Restoring Reinforcing Steel in CRCP

As mentioned previously, the continuity of reinforcement must be maintained through FDRs. The splicing of the reinforcement bars should be conducted using the detailed design information presented previously. Some agencies also require the provision of transverse steel. Figure 6.14 shows a CRCP repair with longitudinal and transverse steel.

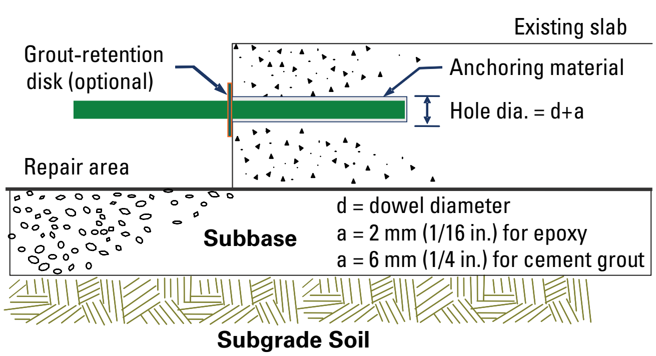


Figure 6.13. Illustration of dowel bar anchoring in slab face

Figure 6.14. CRCP repair with longitudinal and transverse steel.

Step 5: Treatment of Longitudinal Joint

As described previously, a bondbreaker board or the addition of tiebars may be required as dictated by the length of the repair. When the repair length is less than 4.5 m (15 ft), a bondbreaker board is typically placed along the length of the longitudinal joint to isolate it from the adjacent slab. Generally, a fiberboard or similar material is used and configured to match the repair area depth and length and sit flush with the longitudinal face of the repair. For longer repairs, tiebars should be installed along the face of the adjacent slab using procedures similar to those used for installing dowel bars. The tiebars are typically spaced at 762–914 mm (30–36 in.) intervals.

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 (Darter, Barenberg, and Yrjanson 1985; Snyder et al. 1989). 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 4 and 32°C (40 and 90°F) for any concrete placement (ACPA 2006). The addition of extra water at the construction site should not be allowed because this will decrease the strength and increase shrinkage.

For repairs less than 3 m (10 ft), the surface of the concrete should be struck off with a screed perpendicular to the centerline of the pavement, whereas repairs longer than 3 m (10 ft) should be struck off with the screed parallel to the centerline of the pavement; see Figure 6.15. 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.

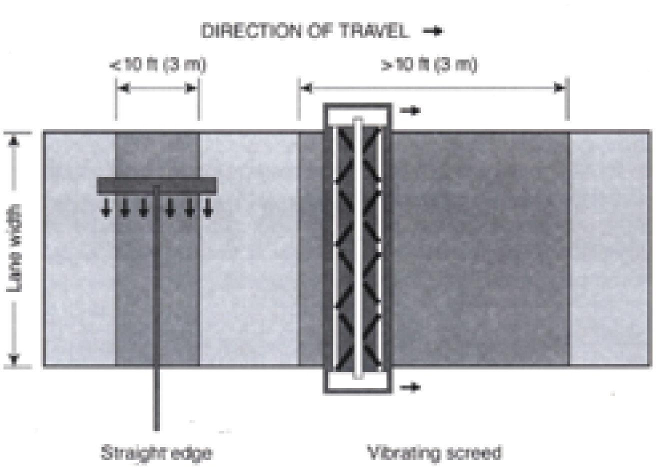


Figure 6.15. Recommended finishing direction depending on size of repair (ACPA 2006)

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 should be sawed as soon as possible without causing significant raveling.

On CRCP repairs, it may be necessary to restrict the time of placing concrete to late in the afternoon, depending on the climatic and pavement conditions. On some patching projects where concrete has been placed in the mornings, expansion of the adjacent slab in the afternoon has resulted in crushing of the repair concrete. This is especially true when the failure extends across all lanes.

Step 7: Curing

Moisture retention and temperature during the curing period are critical to the ultimate strength of the concrete. Proper curing is even more important when using set accelerating admixtures. Therefore, as soon as the bleed water has disappeared from the surface of the concrete (typically within one-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 membranes (compounds), and polyethylene sheeting. In general, a normal application of the pigmented curing compound (typically 4.9 m^2/liter [200 ft^2/gal]) gives the best results. A recent FHWA report provides more detailed guidelines on curing (Poole 2005).

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. 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.

Step 8: Diamond Grinding (Optional)

Rehabilitation techniques such as FDRs may result in increased roughness if not finished properly. In particular, differences in elevation between the repair areas and the existing pavement can create an uncomfortable ride. Restoration of a smooth ride may also be an issue when using precast panels. If needed, the best method to blend repairs into a concrete pavement is with diamond grinding. The smooth surface results in improved rideability of the construction project.

Step 9: Joint Sealing on JCPs

Experience has shown that both the transverse and longitudinal repair joints must be sawed or formed and then sealed as soon as possible after concrete placement. This will reduce spalling (by lowering the initial point-to-point contact between the existing slab and newly placed repair) and will minimize the infiltration of water. The joint sealant shape factor is the primary factor to consider. Chapter 10 discusses procedures and materials for sealing these joints.

7. FDR Using Precast Slabs

During the last decade, a number of highway agencies have implemented precast paving technologies for the repair, rehabilitation, and reconstruction of roadway pavements. This innovative technology uses prefabricated concrete panels that are fabricated or assembled at a plant, transported to the project site, and then installed on a prepared foundation (existing pavement or regraded foundation) (Tayabji and Hall 2010). The specific advantages of using precast pavement systems for the repair and rehabilitation of concrete pavements include the following (Tayabji, Ye, and Buch 2012):

- **Better-Quality Concrete**—There are no issues related to the quality of fresh concrete 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 the precast panels occurs under controlled conditions at the precast concrete plant.
- **Minimal Weather Restrictions on Placement**—The construction season can be extended because 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**—Issues related to late or shallow sawing do not develop.

Precast repairs offer an attractive alternative to cast-in-place repairs, particularly in situations where high traffic volumes and consideration of user delay costs favor more expeditious rehabilitation solutions (Tayabji and Hall 2010). Precast pavement slabs have been employed by a number of agencies in intermittent repair applications, in which precast panels are placed as FDRs at isolated joints or cracks (or even as full slab replacements). For example, Michigan, New Jersey, New York, Ontario, and the Illinois Tollway are a few of the highway agencies 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, Ye, and Buch 2012). 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 (paralleling the requirements for effective, cast-in-place repairs) (Tayabji, Ye, and Buch 2012).

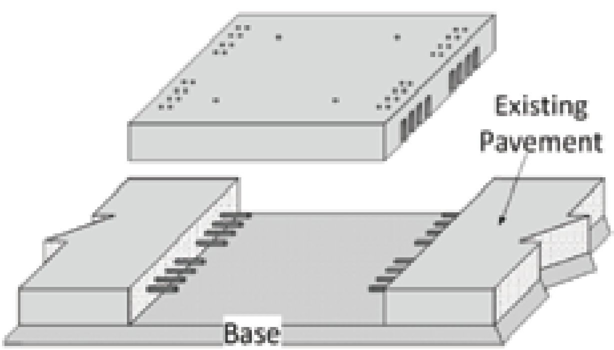
Precast Systems

There are a number of different systems available for FDR using precast slabs (Tayabji, Ye, and Buch 2012). Each of these systems essentially shares the same components, consisting of the preparation of the slab at an off-site precast plant, preparing the repair area (including proper sizing and preparing the base), installation of load transfer devices (system dependent), panel placement, and grout undersealing (system dependent). The load transfer provisions are often what differentiates these various systems, with common methods described below.

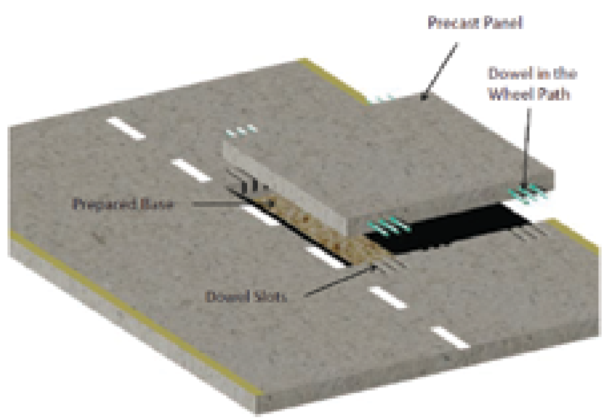
- **Drilling and Installing Dowel Bars in the Existing Pavement**—This is similar to what is done for conventional FDRs, but it requires precast slabs with slots at the bottom to accommodate the dowel bars;

see Figure 6.16a. A porthole in the precast slab is then used to pump grout beneath the slab to anchor the dowel bars.

- **Using Dowel Bars and Slots at the Surface**—In this method, conventional slots (typically 63 mm [2.5 in.] wide) are cut in the surface to accept dowel bars. Two options are available for providing the load transfer (Tayabji, Ye, and Buch 2012):
  - *Partial Dowel Bar Retrofit Technique*—Dowel slots are cut in the existing pavement before panel placement in order to accommodate dowel bars cast in the slab; see Figure 6.16b. The dowel slots are then patched using approved patch material (similar to dowel bar retrofit).
  - *Full Dowel Bar Retrofit Technique*—After the panel has been placed, 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 dowel bar retrofit procedures.



(a) Dowels drilled in existing slab



(b) Dowels fabricated in precast slab

Figure 6.16. Schematic illustrations of two precast FDR alternatives (Tayabji, Ye, and Buch 2012)

General Construction Steps

Although different precast systems exist for 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, Ye, and Buch 2012).

1. **Panel Installation Staging and Lane Closures**—Accessibility to the project site must be identified 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 planning of the preinstallation and installation activities. The work zone requires closure of two lanes, the lane undergoing repairs and an additional lane for construction traffic (especially for trucks delivering the panels). In some cases, two separate closures may be required, in which the existing concrete is removed and the new repair panel is placed on the first day and the remaining activities (such as load transfer provision and undersealing) are performed on the second day.

2. **Removal of Distressed Concrete Sections**—As with conventional FDRs, the repair boundaries are identified and full-depth sawcuts are made to isolate the deteriorated concrete so that it may be removed. The repair should be full width, and the sawcutting of the distressed area should be carried out as close as possible to the installation time of the precast panels. Highway 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 10–13 mm (0.38–0.5 in.). The lift-out method of pavement removal is preferred, being careful to avoid damage to the base and to the adjacent slabs that are to be left intact.

3. **Base Preparation and Bedding Materials**—The base may need to be regraded or reworked to provide a uniform level of support to the precast repair slab and to meet established tolerance levels. Because of time constraints, it is very unlikely that a new base will be used for a precast repair. The base treatment options may include the following:
   - The existing base is regraded to the specified grade and compacted if granular. A thin finer-grained granular bedding material may be used to provide a

smoother grade. Additional base material will be needed if the base grade needs to be adjusted to accommodate the thickness of the precast repair slab. In many cases, the precast slab is fabricated slightly thinner than the existing pavement, so any thickness deficiencies must be made up with the base course.

- A thin granular bedding layer may be placed over the base if the existing base is stabilized (asphalt treated or cement treated). The use of stone dust-like material or manufactured or river sand to achieve a smooth base surface or as filler should be carefully considered, however, because these materials cannot be compacted well and are not structurally stable. The stone dust or sand bedding thickness should be limited to a maximum of 6 mm (0.25 in.).
- A fast-setting bedding grout (flowable fill) or polyurethane material may be used. These may be used with existing asphalt- and concrete-treated base courses, and they can be used to establish the smooth grade and uniform support needed to accept the precast panel. The flowable fill or polyurethane material should be at least 13–25 mm (0.5–1 in.) thick.

4. **Load Transfer Provisions**—Load transfer is critical to the performance of any type of FDR (cast in place or precast). As previously described, load transfer in precast systems is provided by dowel bars that are drilled into the existing slab prior to panel placement, or by systems that use either a partial or full dowel bar retrofit technique. For the first method, the dowel bars will slip into slots located on the bottom of the precast slab, which will require the insertion of grout material beneath the slab and into the slots to provide anchoring for the dowel bars; for the latter methods, the installation of dowel bars on the surface essentially follows the methodology outlined for dowel bar retrofit; see Chapter 8. In either case, a minimum of three dowel bars is provided per wheelpath.
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 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 handled by a crane and carefully lowered into position so that it is centrally located within the repair area and 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 next 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 (required for all systems to ensure that full support exists beneath the slab), surface grinding (as required by the system for rideability), and joint sealing. The repair 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 6.17 shows some installation photos of the various systems.

For intermittent precast repairs within a given lane closure area, the typical production rate is about 14 to 18 panels per 6- to 8-hour lane closure, or about one panel per 20 to 25 minutes. Ideally, two crews should be used for repair installations—one crew preparing the repair area, including drilling and epoxy grouting the dowel bars, and the second crew installing the panels (Tayabji, Ye, and Buch 2012).

The costs of precast slabs for repair activities has come down during the last few years as contractors become more familiar with the technology. Typical pricing in 2010 was about \$300–\$600/m^2 (\$250–\$500/yd^2) (Tayabji, Ye, and Buch 2012).

(a) Dowel bars drilled in existing slab

(b) Dowel bars fabricated in precast panel

(c) Precast panel using full dowel bar retrofit installation

Figure 6.17. Placement of different precast repair systems (Tayabji, Ye, and Buch 2012)

8. FDR of Utility Cuts

Different types of utilities (e.g., storm and sanitary sewers, water mains, telecommunication lines, gas mains and service lines, and power conduits) must periodically be accessed for repair or maintenance. This requires cutting into the street to gain that access, which can disrupt the uniformity and continuity of the pavement and even compromise its overall structural integrity. Thus, after utility access, the use of effective materials and construction procedures is critical to help restore the pavement to like-new condition and ensure long-term performance.

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 information provided herein focuses on permanent, long-term repairs constructed using cast-in-place concrete.

Key Factors in Restoration Success

Studies on utility cut restoration techniques indicate that there are three major modes of failure associated with utility repairs (Schaefer et al. 2005; Suleiman et al. 2010; SUDAS 2013):

- One mode of failure is the 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 second mode of failure is a “bump” forming over the restoration, resulting from the uplift/heaving of the backfill soil caused by frost action or from the settlement of the surrounding soil.
- The third mode of failure is the weakening of the surrounding soils as a result of the stress-state change created by the trench excavation. This weakening causes the adjacent pavement to settle and fail, which then leads the repair itself to fail. During trenching, native soil surrounding the perimeter of the trench is subjected to a loss of lateral support/confinement, which leads to loss of material under the pavement and bulging/sloughing of the soil along the trench sidewall. This weakened zone, known as the “zone of influence,” cannot be fully restored.

Key factors cited as significantly affecting restoration performance are as follows (Suleiman et al. 2010; SUDAS 2013):

- Proper compaction of trench backfill materials (ensured by quality control measures) and use of appropriate backfill materials (materials that can achieve relatively high density without a significant amount of compaction) in order to minimize future settlement/consolidation.

- Use of backfill materials that are not significantly susceptible to frost to prevent heaving action.
- Placement of granular backfill materials at a moisture content above the bulk moisture content to lessen the collapse potential of soil and thereby reduce trench settlement.
- Restricting large equipment from the utility cut edges in order to minimize damage to the surrounding pavement surface and to minimize weakening of soil in the "zone of influence."

General Construction Steps

The recommended process for utility cut restoration in concrete pavement is summarized in the following ten steps (ACPA 2009; Suleiman et al. 2010; SUDAS 2013):

1. Planning the Utility Cut Location, Size, and Shape.
    - Utility cuts in existing concrete pavements should be made at least 150–300 mm (6–12 in.) beyond the edges of the required trench to prevent the existing concrete from being undermined during utility repair/installation and to provide support for the restoration patch; some agencies recommend a minimum cutback of 900 mm (3 ft). An example of a trench for a utility cut restoration is shown in Figure 6.18.

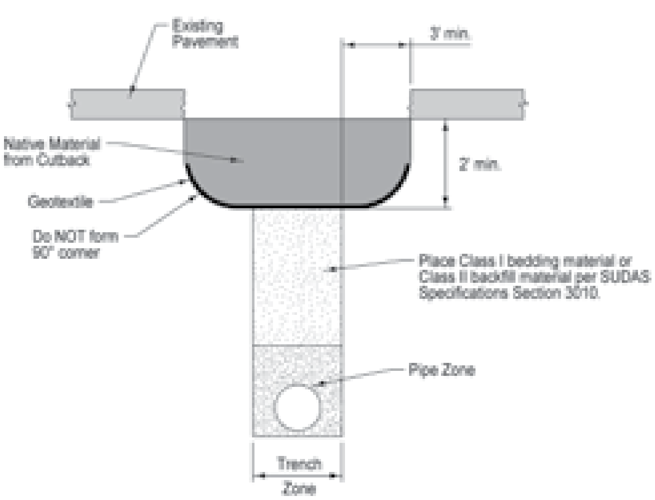


Figure 6.18. Example of trench for utility cut restoration (SUDAS 2013)

    - Utility cuts in the slab interior should be located at least 0.6 m (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."
2. Creating the Utility Cut.
    - For concrete pavements thinner than 176 mm (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 slab thickness) and jackhammering through the remaining depth (the jackhammer chipping produces a roughened face).
    - For concrete pavements 176 mm (7 in.) or thicker, dowel bars are required for load transfer across transverse joints. Since aggregate interlock is 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.
    - If used, the typical breakup and clean-out method of concrete removal must not damage the adjacent pavement or overbreak/undercut the slab bottom.
    - The alternative lift-out procedure should employ a vertical lift to prevent the slab from binding and spalling the adjacent pavement.
4. Creating the Trench.
    - During trench construction, excavation equipment should be kept as far away from the trench area as possible to minimize trench wall sloughing; see Figure 6.19. The smallest equipment that can satisfactorily perform the job should be used to further minimize loading effects.
    - The constructed trench should include beveled sidewalls (i.e., walls slightly angled outward toward the top of the trench) to facilitate compaction of the backfill material.
5. Repairing/Upgrading or Installing the Utility.
    - Install shoring (as necessary) along trench sidewalls to prevent cave-ins.

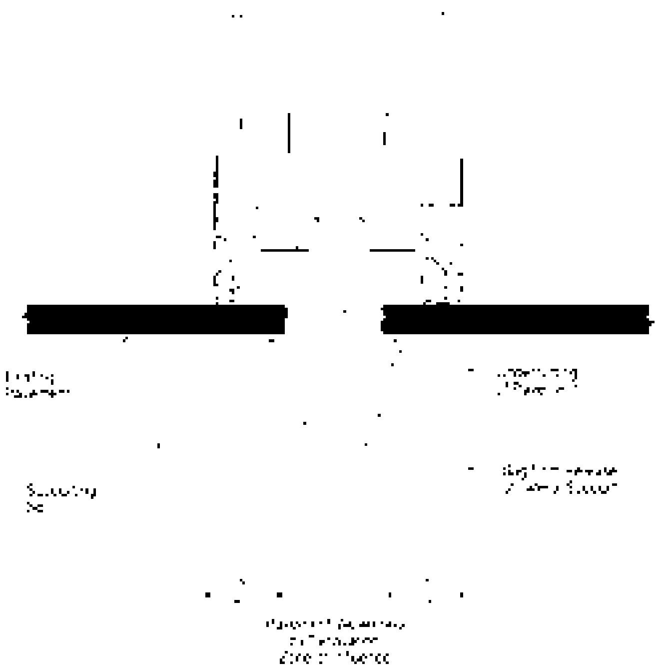


Figure 6.19. Overstressing of the pavement and supporting soil adjacent to the trench (SUDAS 2013)

6. Backfilling the Trench.
   - Soils excavated from the trench should not be mixed with granular backfill unless previous lab testing yielding a range of recommended moisture content and densities to be achieved in the field are conducted. Also, saturated material from the excavation should not be used.
   - Granular backfill material should be placed in 200- to 300-mm (8- to 12-in.) lifts and compacted to at least medium (≥65 percent) relative density (95 percent is ideal) with moisture contents above the bulk moisture content. If cohesive soils are used in the top 0.6 m (2 ft) to match existing subgrade materials, the soil should be placed in 203-mm (8-in.) lifts and compacted to at least 95 percent of Standard Proctor Density and within 2 percent of optimum moisture content for that soil. Quality control procedures should be implemented to ensure that compaction requirements are met.
   - Consideration should be given to using 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 rather than to produce a hardened soil-cement.
   - Consideration should also be given to using flowable fill, a controlled low-strength, self-leveling material made with cement, supplementary cementitious materials, and water that easily flows and fills the utility trench area and then gains strength.
7. Installing Necessary Embedded Steel.
   - All necessary subgrade/subbase and/or backfill compaction should be completed prior to installing dowel bars and/or tiebars into the existing concrete pavement.
   - Recommended dowel bar sizes and drilled hole diameters for utility cut restorations are provided in Table 6.8. The grout (cementitious or epoxy) should be placed in the back of each drilled hole to ensure that the material flows out around the bar as it is inserted.
8. Placing, Finishing, Texturing, and Curing the New Concrete Surface.
   - Prior to placing concrete into the utility cut area, any loose subgrade/subbase material should be firmly compacted with hand or pneumatic tools.
   - All concrete placement, consolidation, and finishing techniques should follow standard procedures. Final surface texturing should match the existing concrete pavement. Effective curing techniques should be followed to ensure proper strength and durability.
9. Jointing and Joint Sealing.
   - Sawed joints should be one-fourth the slab thickness for any interior 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.

Table 6.8. Dowel Size Recommendations for Utility Cut Restoration (ACPA 2009)

Adjacent Pavement Thickness, in. (mm)	Dowel Diameter, in. (mm)	Drilled Hole Diameter, in. (mm)	
		Grout	Epoxy
≤7 (175)	No dowel	–	–
7–8 (175–200)	1.0 (25)	1.2 (30)	1.08 (27)
8–9.5 (200–240)	1.25 (32)	1.45 (37)	1.33 (34)
10+ (250+)	1.5 (38)	1.7 (43)	1.58 (40)

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 6.9 provides recommended minimum compressive strengths for opening to traffic. A summary of the overall utility cut restoration process is illustrated in Figure 6.20 (ACPA 2009).

Table 6.9. Minimum Opening Strength for Utility Cuts (ACPA 2009)

Utility Cut Thickness, in. (mm)	Compressive Strength for Opening to Traffic, psi (MPa)	
	Utility Cut Length < 10 ft (3.0 m)	Utility Cut Length > 10 ft (3.0 m)
6 (150)	3,000 (20.7)	3,600 (24.8)
7 (175)	2,400 (16.5)	2,700 (18.6)
8 (200)	2,150 (14.8)	2.150 (14.8)
9+ (225+)	2,000 (13.8)	2,000 (13.8)

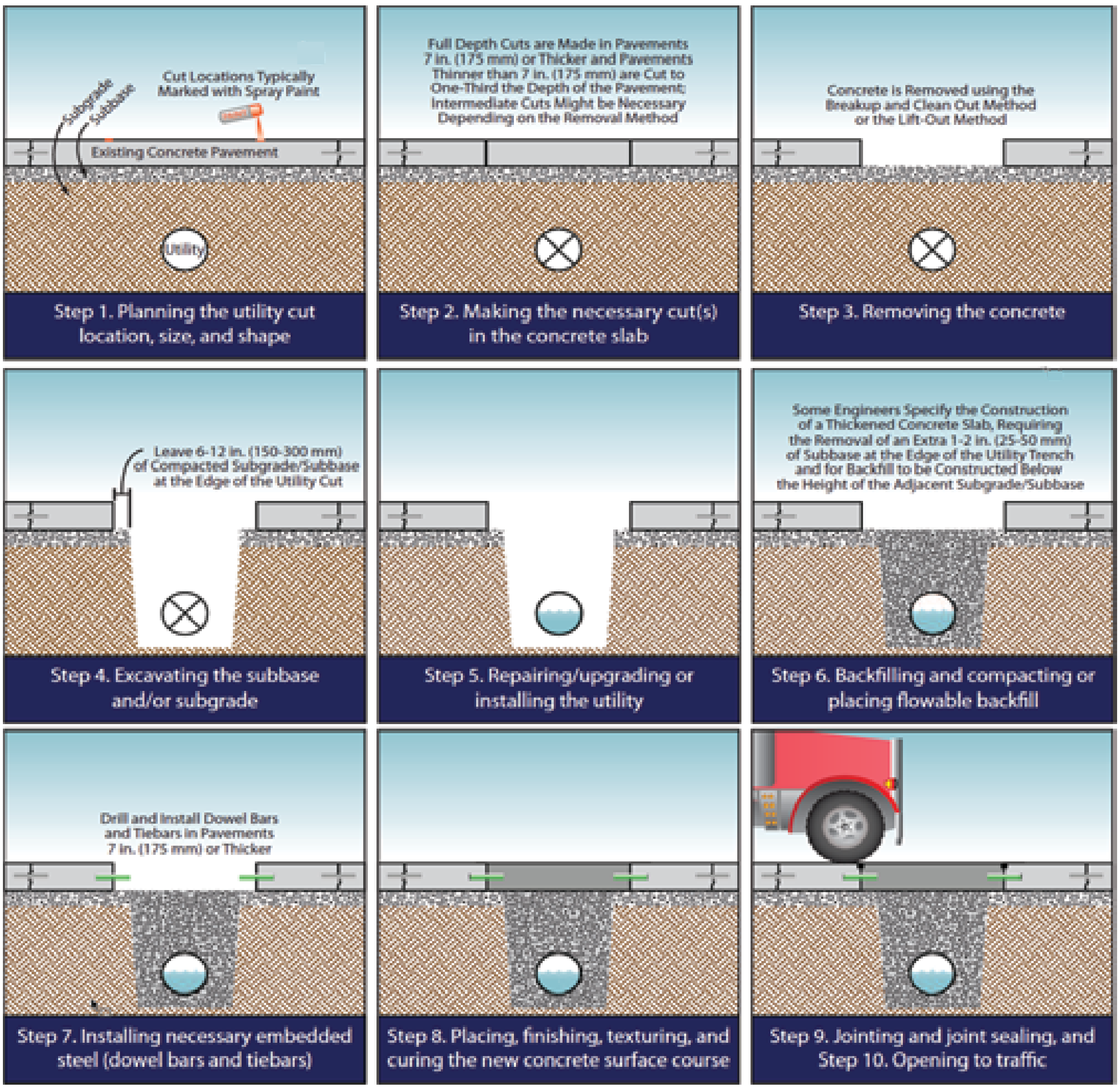


Figure 6.20. Summary of utility cut repair process (ACPA 2009)

9. Quality Assurance

Quality assurance practices for FDRs mirror those for the placement of conventional concrete pavement. Paying close attention to the quality of the construction procedures and material handling during construction greatly increases the chances of minimizing premature failures on FDR projects. This section summarizes key portions of a recently developed checklist that has been compiled to facilitate the successful design and construction of well-performing FDRs (FHWA 2005). Although these procedures do not necessarily ensure the long-term performance of a specific repair, the checklist topics are intended to remind both the agency and contractor personnel of those specific design and construction topics that have the potential of influencing the performance of the repair. These checklist items are divided into 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, material usage, and associated specifications. Such a collective review is intended to minimize any misunderstandings in the field between agency designers, inspectors, and construction personnel. 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 FDR. Specifically, the following items should be verified or checked 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 that the estimated number of FDRs agrees with the number specified in the contract.
- Agree on quantities to be placed, but allow flexibility if deterioration is found below the surface.

Document Review

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 MSDS.

Materials Checks

A number of material-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 used, verify the development of maturity curves for the specific mixture for determining opening times.
- The load transfer units (dowels) meet specifications and dowels are properly coated with epoxy (or other approved material) and free of any minor surface damage in accordance with contract documents.
- Dowel-hole cementing grout meets specifications.
- Bondbreaking board meets specifications (typically asphalt-impregnated fiberboard).
- Joint sealant material meets specifications.
- Sufficient quantities of materials are on hand for completion of the project.
- All material 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 checked or 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 repair boundaries as required by the contract documents.
- Verify that required equipment used for concrete removal is all on site, in proper working order, and of sufficient size, weight, and horsepower to accomplish the removal process (including front-end loader, crane, forklift, backhoe, skid steer, and jackhammers).

Repair Area Preparation Equipment

- Verify that the plate compactor is working properly and capable of compacting the subbase material.
- Verify that the gang drills are calibrated, aligned, and sufficiently heavy and powerful enough to drill multiple holes for dowel bars.
- Verify that air compressors are equipped with and using properly functioning oil and moisture filters/traps. This can be accomplished by passing the air stream over a board and then examining for contaminants.

Testing Equipment

- 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 slump cone, pressure-type air meter, cylinder molds and lids, rod, mallet, ruler, and 3-m [10-ft] straightedge).
- Ensure that sufficient storage area on the project site is specifically designated for the storage of concrete cylinders.
- Verify that hand-held concrete vibrators are the proper diameter and operating correctly.
- Verify that all floats and screeds are straight, free of defects, and capable of producing the desired finish.
- Verify that sufficient polyethylene sheeting is readily available on site for immediate deployment as rain protection of freshly placed concrete, should it be required.

Weather Requirements

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

- Verify that air and surface temperature meets manufacturer and contract requirements (typically 4°C [40°F] and above) for concrete placement.
- Repairs should not be performed if rain is imminent. Repairs that have been completed should be covered with polyethylene sheeting to prevent rain damage.

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 pre- and postconstruction traffic-related items should be verified:

- Verify that the signs and devices used match the traffic control plan presented in the contract documents.
- Verify that the setup complies with the Federal or local agency MUTCD or local agency 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.
- Ensure that traffic is not opened to the repaired pavement until the repair material meets strength requirements presented 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 can greatly increase the chances of obtaining well-performing FDRs. Specifically, the following checklist items (organized by construction activity) summarize the recommended project inspection items.

Concrete Removal and Cleanup

- Verify that the boundaries of the removal areas are clearly marked on the pavement surface and the cumulative area of the pavement to be removed is consistent with quantities in the contract documents.
- Verify that the repair size is large enough to accommodate a gang-mounted dowel drilling rig, if one is being used. Note: the minimum longitudinal length of repair is usually 1.8 m (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 and that disturbance of the base or subbase is minimal (note that the lift-out method is preferred).
- Verify that after concrete removal, disturbed base or subbase is recompacted and additional subbase material is added and compacted if necessary.
- Verify that concrete adjoining the repair is not damaged or undercut during concrete removal.
- Ensure that removed concrete is disposed of in the manner described in the contract documents.

Repair Preparation

- Verify that the dowel holes are drilled perpendicular to the vertical edge of the remaining concrete pavement using a gang-mounted drill rig.
- Verify that the holes are thoroughly cleaned using compressed air.
- Verity that approved cement grout or epoxy is placed in 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 (parallel to the centerline and perpendicular to the vertical face of the sawcut excavation) in accordance with contract specifications. Typical tolerances are 6 mm (0.25 in.) misalignment per 300 mm (12 in.) of dowel bar length.
- If used, verify that tiebars 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 4.5 m (15 ft) or greater, tiebars are typically installed in the face of the longitudinal joint. When the length of the repair is less than 4.5 m (15 ft), a bondbreaker board is placed along the length of the repair to isolate it from the adjacent slab.
- Ensure that tiebars are checked for location, depth of insertion, and orientation (perpendicular to centerline and parallel to slab surface).

Placing, Finishing, and Curing Repair Material

- Concrete is typically placed from ready-mix trucks or mobile mixing vehicles in accordance with contract specifications.
- Verify that the fresh concrete is properly consolidated using several vertical penetrations of the surface with a hand-held vibrator.
- Verify that the surface of the concrete repair is level with the adjacent slab using a straightedge in accordance with contract documents.
- Verify that the surface of the repair material is finished and textured to match the adjacent surface.
- Verify that adequate curing compound is applied to the surface of the fresh concrete immediately following finishing and texturing in accordance with contract documents. 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 4°C (40°F). Maintain blanket cover until concrete attains the strength required in the contract documents.

Resealing Joints and Cracks

- Verify that the repairs have attained adequate strength to support concrete saws and that repair perimeters and other unsealed joints are sawed off to specified joint reservoir dimensions.
- 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.
- Verify that mixing, placement, and finishing equipment is properly cleaned for the next use.
- Verify that all construction-related signs are removed when opening the pavement to normal traffic.

10. Troubleshooting

Table 6.10 summarizes some of the more common problems that a contractor or inspector may encounter in the field during the construction of FDRs, whereas Table 6.11 presents some of the performance problems that may be observed later. Recommended solutions for these issues are provided in each of the tables.

Table 6.10. Potential FDR Construction Problems and Associated Solutions (FHWA 2005; ACPA 2006)

Problem	Typical Solutions
Undercut spalling (deterioration on bottom of slab) is evident after removal of concrete from patch area	• Saw back into adjacent slab until sound concrete is encountered. • Make double sawcuts, 150 mm (6 in.) apart, around patch area to reduce damage to adjacent slabs during concrete removal. • Use a carbide-tipped wheel saw to make pressure-relief cuts 100 mm (4 in.) wide inside the area to be removed.
Saw binds when cutting full-depth exterior cuts	• Shut down saw and remove blade from saw. • Wait for slab to cool, then release 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 saw cuts when the pavement is cool. • Use a carbide-tipped wheel saw to make pressure-relief cuts 100 mm (4 in.) wide inside the area to be removed.
Lifting out a patch for an FDR damages adjacent slab	• Adjust lifting cables and position lifting device to ensure a vertical pull. • Resaw and remove broken section of adjacent slab. • Use a forklift or crane instead of a front-end loader.
Slab disintegrates when attempts are made to lift it out	• Complete removal of patch area with backhoe or shovels. • Angle the lift pins and position the cables so that fragmented pieces are bound together during liftout. • 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 subbase to a density consistent with contract documents, adding material as necessary. • Permit the use of aggregate dust or fine sand to level small surface irregularities (12 mm [0.5 in.] or less) in surface of subbase before patch material is placed.
Grout around dowel bars flows back out of the holes after dowels are inserted	• Pump grout to the back of the hole first. • Use a twisting motion when inserting the dowel. • Add a grout retention disk around the bar to prevent grout from leaking out.
Dowels appear to be misaligned once they are inserted into holes	• If misalignment is less than 6 mm (0.25 in.) per 300 mm (12 in.) of dowel bar length, do nothing. • If misalignment is greater than 6 mm (0.25 in.) per 300 mm (12 in.) of dowel bar length on more than three bars, resaw patch boundaries beyond dowels and redrill holes. • Use a gang-mounted drill rig referenced off the slab surface to drill dowel holes.

Table 6.11. Potential FDR Performance Problems and Prevention Techniques

Problem	Typical Causes	Typical Solutions
Longitudinal cracking in the patch	• Patch not long enough. • Insufficient isolation from adjacent slabs. • Inadequate curing for ambient conditions. • Expansion of adjacent slabs on young concrete pavements.	• Verify patch dimensions. • Use proper material to isolate FDR along longitudinal joints. • Avoid patching in extreme climate conditions. • Use appropriate protection against rapid moisture loss (double application of curing compound, curing blankets).
Transverse cracking in the patch	• Patch too long. • Misaligned dowel bars. • Tiebars instead of dowel bars. • Inadequate curing for ambient conditions.	• Verify patch dimensions. • Check dowel size and location. • Use tiebars at only one joint. • Use appropriate curing methods.
Surface scaling	• Poor mix design. • Adding water during placement or finishing. • Overfinishing the surface. • Inadequate curing for ambient conditions.	• Check mix design and adjust if necessary. • Do not add additional water at site. • Do not overfinish surface. • Use appropriate curing methods.
Spalling in patch at the transverse or longitudinal joint	• "Point" load causing high compressive stress. • Incompressibles in joint. • Locked load transfer device.	• Isolate longitudinal joints and ensure transverse joints are clean. • Install all transverse dowels and tiebars in line with the longitudinal joint and perpendicular to the transverse joint.
Deterioration adjacent to the patch	• Inadequate material removal. • Less than full-depth sawcuts. • Poor removal technique.	• Identify removal boundaries outside the area of deterioration. • Sawcut removal areas full depth. • Use removal technique that does not damage adjacent pavement.
Settlement of the patch	• Inadequate load transfer. • Poor base preparation. • Lack of sealant. • Subsurface moisture.	• Follow guidelines for tiebars and load transfer devices. • Prepare subsurface layers properly. • Remove source of any subsurface water. • Seal joints following construction.

11. Summary

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 replacing it with an appropriate repair material that meets the durability and opening-time demands of the project. Full-depth repairs are necessary to address significant structural 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 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 provides guidance on recommended design and construction procedures to administer effective FDRs on both jointed and continuously reinforced pavements.

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Chapter 7

Retrofitted Edgedrains

1. Learning Outcomes

This chapter discusses the installation of retrofitted edgedrains to improve the drainage of existing concrete pavements. After completion of this chapter, the participant should be able to accomplish the following:

- List the benefits and issues associated with retrofitted edgedrains.
- List the components of edgedrain systems.
- Describe recommended installation procedures.
- Identify typical construction problems and remedies.

2. Introduction

Subsurface drainage systems are commonly believed to contribute to the improved performance of both asphalt and concrete pavements (Hall and Crovetti 2007). Although previous research indicates that drainage can effectively extend concrete pavement life (see, for example, Darter et al. 1985; Cedergren 1987; Smith et al. 1998; Christopher 2000), other studies suggest that certain design and construction factors may have a bigger effect on performance than drainage (NCHRP 2002). For instance, a permeable base in a doweled JPCP was observed to have minimal contribution to performance, whereas the same permeable base in a nondoweled JPCP significantly improved performance (NCHRP 2002). In that 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, and stabilized or nonerodible bases (Hall and Crovetti 2007).

The above information notwithstanding, positive drainage may still be required for existing concrete pavements that were constructed without those design features and that are exposed to excessive moisture throughout the year. Although the ideal time to address drainage concerns is during the initial design and construction phases of the project, a number of 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 have begun to show signs of moisture-related distresses, such as pumping and joint faulting. Agencies typically install the drains in an effort to delay or slow the development of those moisture-related distresses, but it is important that the right pavement be targeted and that effective installation procedures be followed to obtain the anticipated benefits.

This chapter presents information regarding the process of retrofitting existing concrete pavements with edgedrains. Included are discussions of key definitions, guidance on project selection, limitations and effectiveness of the method, design considerations, and construction considerations. Also included is a summary of recommended maintenance activities to help ensure the effectiveness of the drainage system.

3. Purpose and Project Selection

Purpose of an Effective 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 is to remove excess water that infiltrates the pavement structure in an effort to reduce, or eliminate, the development of moisture-related damage. The overall goal is to reduce the amount of time that the water is beneath the pavement and the period that the underlying pavement layers are in a saturated condition.

When an existing pavement begins showing signs of moisture-related damage, the agency generally has two options for improving the pavement's drainage: (1) wait and 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 in order 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; NHI 1999; Christopher, Schwartz, and Boudreau 2006; Arika, Canelon, and Nieber 2009), and a comprehensive computer program, DRIP (*Drainage Requirements in Pavements*), is available to perform detailed drainage analyses (Mallela et al. 2002).

In rehabilitation projects where retrofitted edgedrains are to be installed, pavement layers are already in place and little can be done to improve their individual abil-

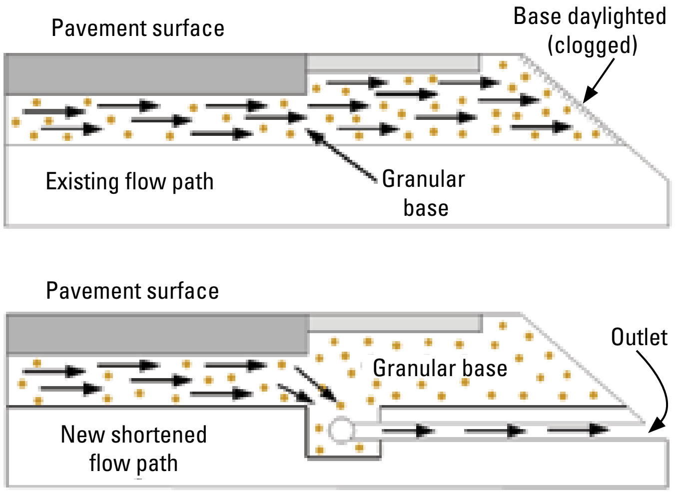

Figure 7.1. Addition of edgedrain to shorten flow path

ity to drain. Because of this, the goal is to shorten the drainage path (i.e., the distance that water must travel to get out from beneath the pavement structure) in order to improve subsurface drainage. Figure 7.1, above, presents a pavement cross section that shows how the presence of retrofitted longitudinal edgedrains can improve the drainability of the pavement by shortening that flow path. In addition, the presence of the edgedrain can help intercept water that infiltrates at the lane-shoulder joint. It is important to recognize, however, that the permeability of the granular base will still play a significant role in determining how quickly some of the water can be removed from the pavement structure.

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 the addition of subsurface drainage may be too late. Furthermore, it is not always clear if retrofitted edgedrains are an appropriate rehabilitation option for a given project. As a first step in identifying 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 only exhibiting a minimal amount of cracking. On the other hand, pavements exhibiting any of the following condition characteristics are considered poor candidates for retrofitted edgedrains (Wells 1985; FHWA 1992; NHI 1999; ITD 2007):

- More than 10 percent of the slabs exhibit cracking.
- A high number of transverse joints are spalled.
- There are significant levels of pumping (unless the voids under the pavement are to be corrected).
- There is a presence of other significant distress (such as edge punchouts, transverse cracking, longitudinal cracking, and corner breaks) that would require extensive patching to return the pavement to an adequate level of service.
- There are pavements where the existing base contains greater than 15 percent fines (material passing the 0.075-mm [No. 200] sieve). Base materials with a high percentage of fines may be too impermeable for an effective retrofitted subdrainage installation.

In sum, retrofitted edgedrains are <u>not</u> 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 acceptable geometrics (longitudinal and transverse slopes) and adequate depth and condition of roadside ditches. It is important that these pavement characteristics be adequate (or improved during the edgedrain installation) so that water can effectively be removed. In addition, consideration should be given to providing edgedrains only in critical drainage areas (such as on curves or low areas) and not necessarily during the entire length of a project.

4. 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 some noted reductions in faulting on some projects and no such reduction on others (NCHRP 2002). In some cases, retrofitted edgedrains have been found to have even contributed to the further deterioration of the pavement structure (Gulden 1983; Wells and Nokes 1993); this was due to the drains actually removing base and soil material from beneath the slabs (leading to poor support conditions) or because of the clogging of the outlets (saturating the pavement and leading to reduced support conditions). Such clogging is not uncommon—a recent study in Iowa found that about 65 percent of drainage outlets on concrete pavements were blocked (Ceylan et al. 2013).

Overall, this inconsistent performance of retrofitted edgedrains has been mostly attributed to a combination of improper usage (project selection), improper design, damage during installation, lack of postinstallation maintenance, or the failure to provide other pavement repairs that are needed at the time of retrofitting the edgedrains. Indeed, a study of edgedrains in California found that more than 70 percent of the edgedrains were not performing efficiently or as designed, but the overall poor performance was attributed to design flaws, improper construction, and lack of maintenance (Bhattacharya et al. 2009). Additionally, as noted earlier, it is believed that many concrete pavements are less vulnerable to the effects of poor drainage, given the more widespread use of effective design features (dowel bars, nonerodible bases) in modern designs (Hall and Crovetti 2007). Thus, the entire pavement must be viewed as a system, not only from the perspective of whether or not the pavement will benefit from retrofitted drainage, but also in ensuring that the drains are properly designed, constructed, and maintained.

In considering subsurface drainage on an existing pavement, the design engineer is forced to deal with the existing pavement 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 for water to migrate from beneath the pavement to the edgedrains. It is often suggested that the base course contain no more than about 15 percent fines, ideally even less to provide some permeability. There still can be some benefit, however, to the use of retrofitted edgedrains in removing surface infiltration water that enters at the lane-shoulder joint (Christopher, Schwartz, and Boudreau 2006). Since this is a primary entry point of surface infiltration, retrofitted edgedrains—by virtue of their installation at that location—can remove that water, regardless of the permeability or gradation of the 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 Rehabilitation Pavement Structures* 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" (NCHRP 2004). This in essence places the burden on the individual highway agency to assess the value of providing subsurface drainage based on their local climatic and subsurface conditions, pavement designs, and design practices (AASHTO 2008). And, as previously indicated, if determined to be appropriate, the proper installation, construction, and maintenance of the systems is critical to ensure their functionality and performance (Daleiden 1998).

5. 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 (PGED).
- Aggregate drains (sometimes called "French" drains).

For each of these, 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 these various 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 the roadway. Water is discharged from the pipe edgedrains into the ditches through regularly spaced transverse outlet pipes connected to the longitudinal drainage pipe. Perforated corrugated plastic is commonly used for the longitudinal collector pipe, although rigid, smooth-walled plastic pipe is being used more widely because it lays flat in the trench and is less susceptible to crushing. The trench is partially lined with geotextile fabric (in areas where it comes in contact with either the subbase or subgrade materials) to prevent the infiltration of fines, and then it is filled 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.

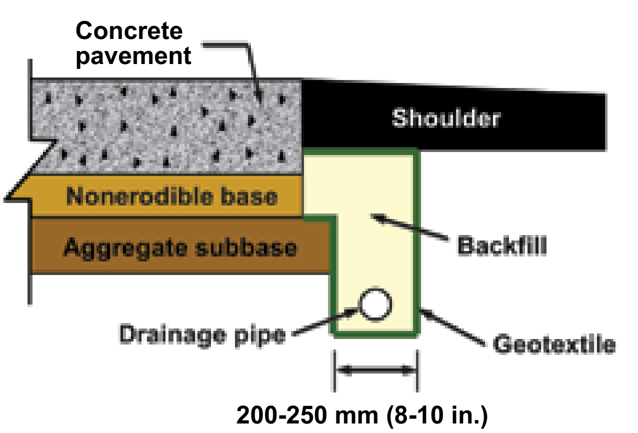

Figure 7.2. Recommended design for retrofitted pipe edgedrains (NHI 1999)

PGEDs

Prefabricated geocomposite edgedrains, also known as "panel" or "fin" drains, consist of a flat, extruded plastic drainage core wrapped with a geotextile filter. Figure 7.3 provides typical geocomposite edgedrain details, whereas Figure 7.4 depicts a recommended installation detail.

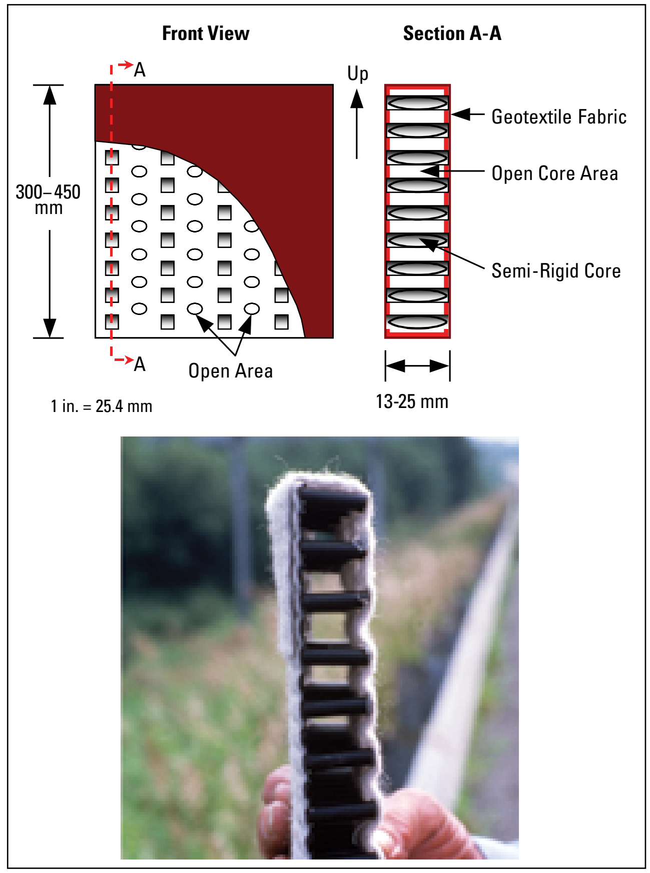

Figure 7.3. Typical prefabricated geotextile edgedrain configuration (Fleckenstein, Allen, and Harison 1994; NHI 1999)

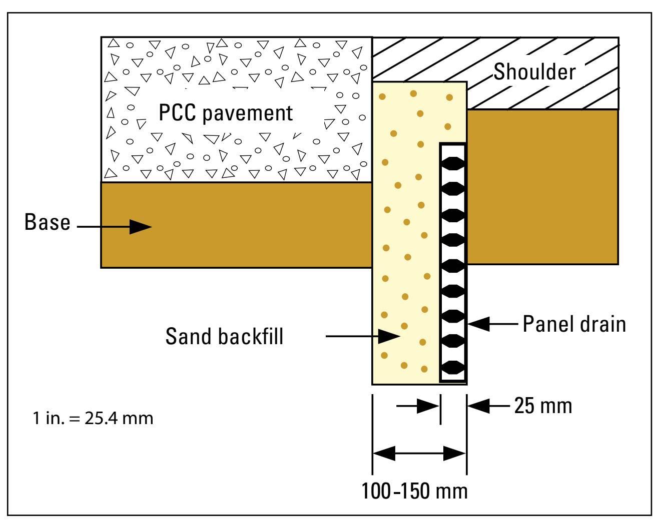

Figure 7.4. Recommended installation of geocomposite edgedrains (Koerner et al. 1994)

Prefabricated geocomposite edgedrains are typically 13–25 mm (0.5–1 in.) thick and are manufactured in long strips that are coiled into rolls. Their size and the incorporation of a geotextile filter in their design means that they can be placed in narrower trenches than conventional pipe edgedrain installations. Although geocomposite edgedrains 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 of the typically lower permeability of the existing base course materials).

The main advantage of PGEDs is that they are easier and cheaper to install than conventional pipe edgedrains. One disadvantage of geocomposite edgedrains, however, is their susceptibility to damage during construction. For example, if proper care is not taken during the backfilling operations, crushing, bending, or buckling of the drainage core may occur (Koerner et al. 1994). This can lead to siltation and clogging issues in the PGED, which has prompted several highway agencies to no longer allow their use. Nevertheless, an NCHRP study found generally good performance from PGEDs, and it reported that most failures were predictable and related to a poor drainage design, a misapplication of the treatment, or improper construction techniques (Christopher 2000). Furthermore, installing the PGED on the shoulder side of the trench (as shown in Figure 7.4) helps 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, Allen, and Harison 1994; Koerner et al. 1994).

Aggregate Drains

Aggregate drains—consisting of a free-draining aggregate trench constructed at the edge of the pavement—have not typically been recommended because they have a relatively low hydraulic capacity and cannot be maintained (FHWA 1992). Some agencies (for example, Missouri and Ohio) have used these systems effectively on pavements without subsurface drainage, however, particularly on lower volume roadways or to provide localized, spot improvements. These drains are physically cut into the edge of the pavement and configured such that the bottom of the aggregate drain is at or below the bottom of the pavement's aggregate base. Figure 7.5 shows a schematic of an aggregate underdrain system that is used in Missouri.

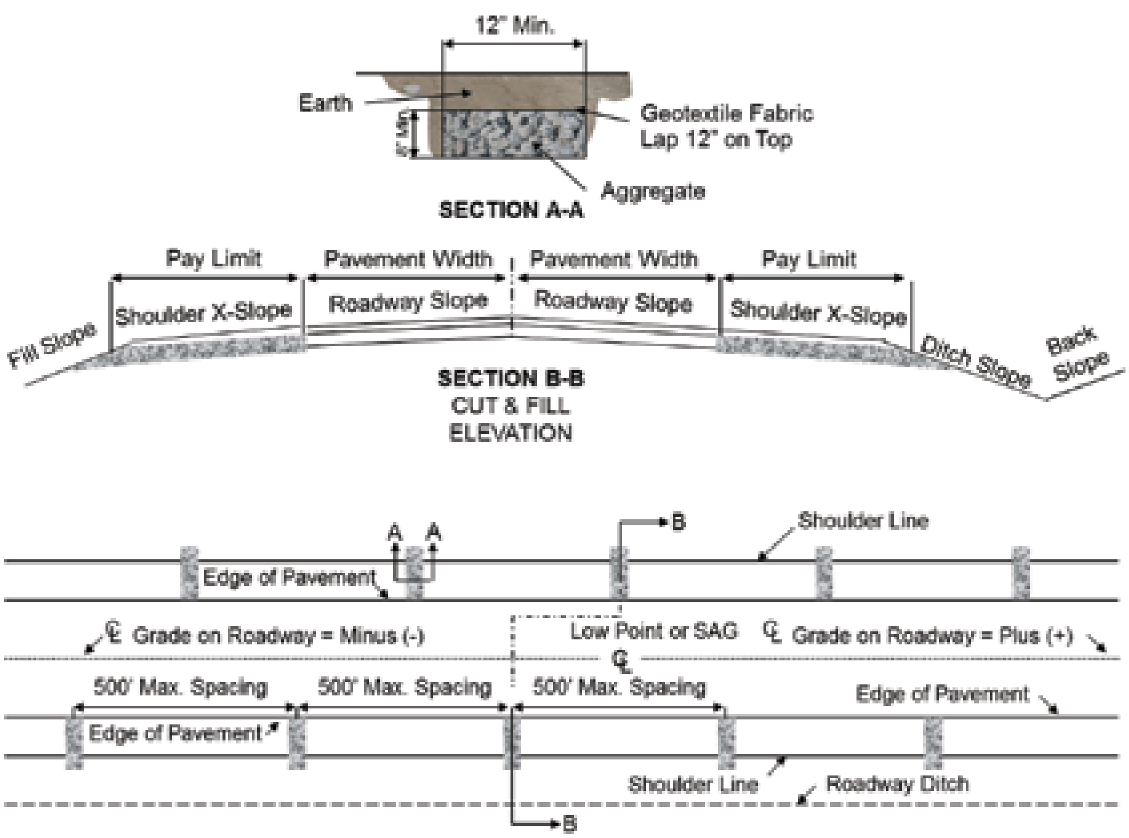

Figure 7.5. Aggregate underdrain system used in Missouri

Backfill Material

The backfill/filler material placed in the trench around the pipe or alongside the geocomposite serves the following functions:

- It acts as a drainage medium to provide a means by which water is moved from the pavement layers to the drainage pipe.
- It acts as a filter system that prevents or restricts fines from moving into and clogging the drainage system (although the effectiveness may be reduced over time).
- It supports and confines the drain pipe or geocomposite, providing protection both during construction and while in service.
- It 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 feature, be it a pipe or geocomposite, does not become clogged with fines. Recommended gradations for the backfill/filler material are found in numerous references (FHWA 1992; NHI 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. AASHTO No. 57 gradation generally provides sufficient permeability and stability for use as nonstabilized backfill material. Nonstabilized pea gravels are not recommended as the backfill material because they cannot be compacted satisfactorily. Proper compaction of the backfill material is important to avoid settlement over the edgedrain, yet overcompaction should 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 allow the drainage system to effectively remove the water from the pavement. The general philosophy is that each segment of the drainage system is adequately sized as water moves toward the outlet, as shown in Figure 7.6. In addition to sizing the components of the drainage system, however, it is important to design filters (geotextile or aggregate) that are effective at preventing fines from entering the edgedrain (not clogging) over the life of the system (Christopher 2000). The grade of the invert 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).

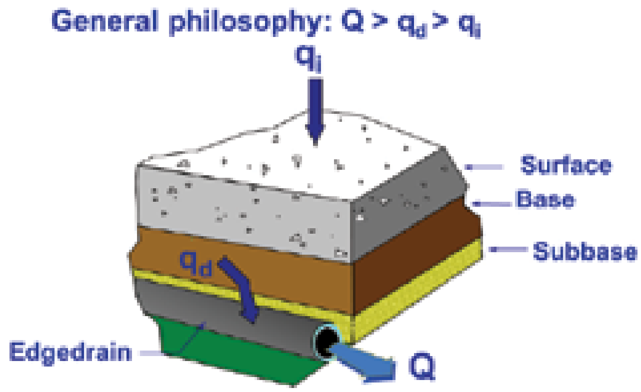

Figure 7.6. Sizing of each element of the drainage system (NHI 1999)

This section provides an abbreviated explanation of the major considerations associated with designing effective retrofitted edgedrains, with more detailed information provided elsewhere (Moulton 1980; FHWA 1992; NHI 1999; Arika, Canelon, and Nieber 2009). As previously noted, the DRIP computer program is available from the FHWA and can be used to perform detailed drainage analyses (Mallela et al. 2002).

Estimate Design Flow Rate

The first step in the design of retrofitted edgedrains is the determination of the net inflow of water. 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 often ignored in the analysis, but if these items are determined to be critical on a particular project, then drainage treatments specific to those sources will be required.

The amount of 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. Consequently, the actual infiltration will be the lesser of two values: (1) the amount of water that could enter through cracks and joints; or (2) the amount of water that the base course is able to accept.

The *design flow rate* is an 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 by knowing detailed information about the base (e.g., width, thickness, permeability) and encountered slopes (cross slope and longitudinal edgedrain slope). Details of the available methods for computing this design flow rate are described elsewhere (Moulton 1980; FHWA 1992; NHI 1999) and are automated in the DRIP computer software.

Edgedrain (Collector) Type

As mentioned previously, two types of longitudinal edgedrains are commonly used for retrofitted drainage projects: pipe edgedrains and PGEDs. It is important that the selected collector type be compatible with the existing pavement structure as well as the surrounding materials.

For pipe edgedrains, several types of drainage pipe of various lengths and diameters have been used successfully in collector systems. Highway agencies use 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. As previously mentioned, CPE pipe has been commonly used, but many agencies are moving toward the use of rigid pipe because it lies flat in the trench and is less susceptible to crushing. For geocomposite edgedrains, product selection should consider an evaluation based on the test procedures outlined in ASTM D 6244-11, *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 diameter of pipe edgedrains is often selected as the minimum diameter that facilitates maintenance (cleaning) activities and allows a reasonable distance between outlets (Christopher 2000). Pipe diameters typically range from 38 to 203 mm (1.5 to 8 in.), with 102 mm (4 in.) being the most common. The larger sizes are commonly preferred because of their ability to be easily cleaned and maintained. A typical cross section for a geocomposite edgedrain has a width of 13–25 mm (0.5–1.0 in.) and a height of 300–450 mm (12–18 in.) (see Figure 7.3) (Fleckenstein, Allen, and Harison 1994).

The computation of the actual flow capacity (required to determine drain sizes) is fairly complicated and is beyond the scope of this chapter. A detailed explanation of these computation methods is available elsewhere (Moulton 1980; FHWA 1992; NHI 1999; Arika, Canelon, and Nieber 2009), and the computations themselves are completely automated in the DRIP software program.

Edgedrain Location

The design depth for the collector pipes should consider the down elevation available for outletting the water, 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 50 mm (2 in.) below the subbase/subgrade interface. When significant frost penetra-

Figure 7.7. Corrugated (left) and rigid (right) pipe edgedrains

tion 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.

The location of the drain within the trench is also a major concern for retrofitted geocomposite edgedrains. As described previously, the recommended approach is to place geocomposite edgedrains on the shoulder side of the trench (see Figure 7.4). Studies have shown that this approach will minimize voids within the trench, alleviate the problem of soil loss through the geotextile filters, and avoid bending and buckling of the geocomposite edgedrain (Koerner et al. 1994).

Grade Considerations

In most cases, the 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. One solution is to increase the grade of the edgedrain; previous guidance recommends grades of at least 1 percent for smooth pipes and at least 2 percent for corrugated pipes (Moulton 1980). This solution, however, can be impractical for very flat areas. For instance, using a 1 percent grade over a flat section of 200 m (660 ft), the edgedrain will have to be 2 m (6.6 ft) deep on the low side. A more practical solution may be to use smooth pipe and decrease the outlet spacing where flat grades exist.

Trench Width

The required width of trench is a function of construction requirements, drainage requirements, and the permeability of the trench material. Depending on the size of the pipe, many agencies use a trench width of 200–250 mm (8–10 in.) to allow proper placement of the pipe and compaction of the backfill material around the pipe. A narrower trench of 100–150 mm (4–6 in.) is typical for geocomposite edgedrains.

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 these functions throughout the life of the drainage system (Koerner et al. 1994). For both pipe and PGED systems, geotextiles are recommended to line the trench wherever the backfill material comes into contact with the subgrade.

Geotextiles consist of either woven or nonwoven mats of polypropylene or nylon fibers. The fabrics are used in place of graded filter material, permitting greater use to be made of locally available gradations without special processing. To be effective, the selected geotextile must have the following three characteristics (Koerner et al. 1994):

- The voids must be sufficiently open to allow water to pass through the geotextile and into the downstream drain without building excessive pore water pressures in the upstream soil.
- The voids must be sufficiently tight to adequately retain the upstream soil materials so that soil loss does not become excessive and clog the downstream drain.
- The geotextiles must perform the previous two conflicting tasks (that is, open voids versus tight voids) over the anticipated lifetime of the retrofitted drainage system without excessively clogging.

Geotextiles should be designed considering both the subbase and subgrade soils using the filter criteria in the FHWA geosynthetics design manual (Holtz, Christopher, and Berg 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 the edgedrain can result when the geotextile material is not selected based on the properties of the surrounding soil (Bhattacharya et al. 2009). Aggregate bases with significant fines may be prone to clogging the geotextile material.

Outlet Considerations

The outlet pipe should be a 100-mm (4-in.) diameter stiff, nonperforated smooth-walled PVC or high-density polyethylene (HDPE) pipe with a minimum slope of 0.03 percent (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 150 mm (6 in.) above the 10-year ditch flow line and protected with a headwall and splash block that is blended into the slope. Figure 7.8 illustrates the recommended outlet pipe configuration.

The location of outlets is controlled in part by topography and highway geometrics, in that the locations must permit free and unobstructed discharge of the water. It is particularly important to accommodate low points and sags of vertical curves. In general, the recommended outlet spacing is between 76 and 91 m (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 outlet pipes (FHWA 1992). These can be either cast in place or precast and should be placed flush with the slope to facilitate mowing operations. 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). If high ditch flows are expected, flap valves can be used to prevent backflow into the drainage system. A schematic of a precast headwall with a removable rodent screen is shown in Figure 7.9.

If pipe edgedrains are used, the outlet pipes should be connected with the collector pipe through elbows with minimum radii of 305–457 mm (12–18 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 7.10.

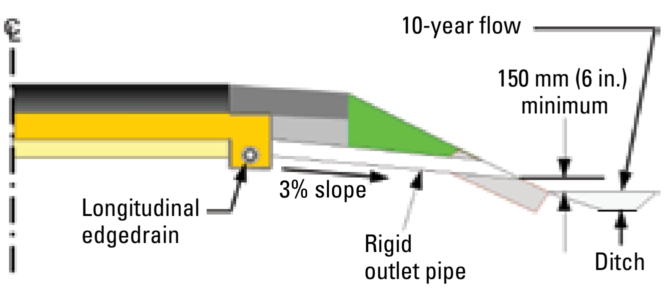

Figure 7.8. Outlet pipe configuration (FHWA 1992; NHI 1999)

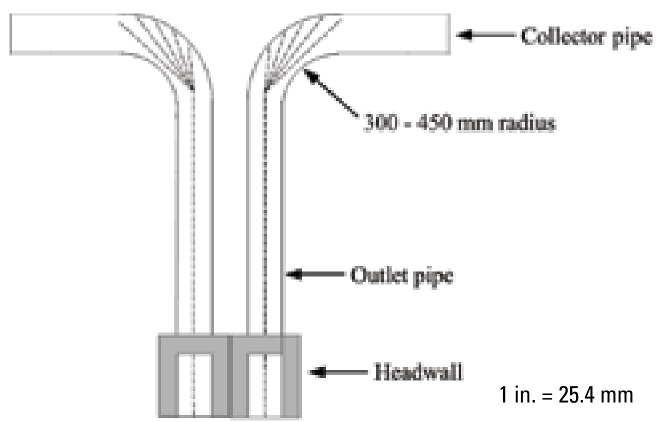

Figure 7.10. Recommended outlet detail to facilitate cleaning and inspections (FHWA 1992)

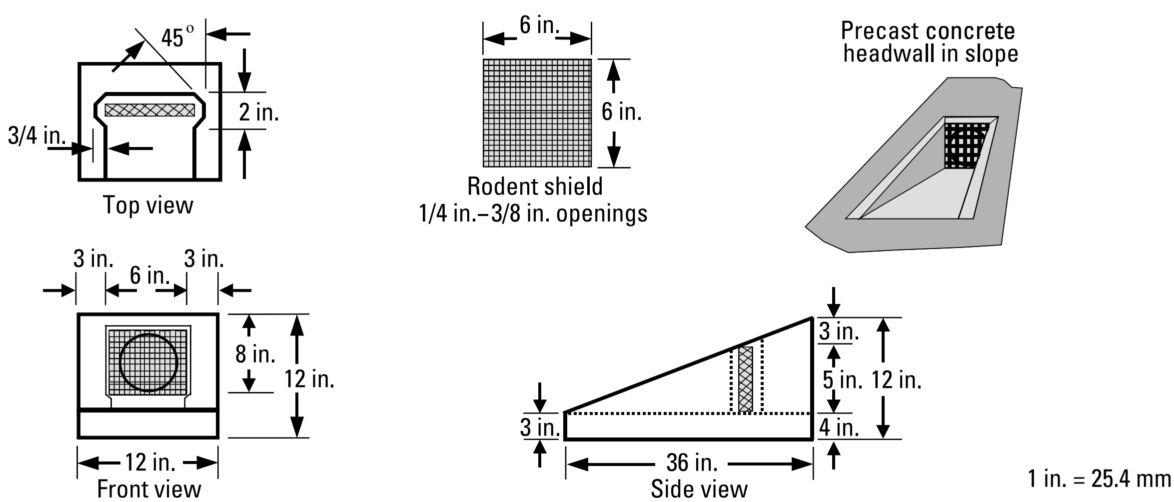

Figure 7.9. Precast headwall with rodent screen (FHWA 1992)

Other Repair Considerations

It is critical that other repairs to the existing pavement also be considered when designing a retrofitted edgedrain project. If the pavement does not receive these needed repairs prior to (or at the same time as) the installation of the retrofit drains, the effectiveness of the retrofitted edgedrains will be limited (NHI 1999). For instance, concrete pavements that exhibit visible pumping and noticeable faulting should be evaluated for possible undersealing prior to the installation of edgedrains. Joint resealing, dowel bar retrofit, and diamond grinding should also be considered as appropriate. Without these repairs, continued pumping, faulting, and loss of support can be expected, even with the addition of the retrofitted edgedrain system.

6. Construction Considerations

Proper construction and maintenance are extremely important to ensure the effectiveness of the edgedrain system. Inconsistent performance of edgedrains, resulting from construction or maintenance problems, has hampered the ability to determine the effectiveness of edgedrains in improving pavement performance. The construction steps involved in retrofitting edgedrains on an existing pavement differ slightly depending on the type of edgedrain being used.

Pipe Edgedrains

Trenching

It is important to maintain correct line and grade when installing longitudinal underdrains. 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 50 mm (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 150 mm (6 in.) above the ditch flowline to preclude 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 150 mm (6 in.) above the 10-year expected water level in the storm drain system (see Figure 7.8).

Placement of Geotextile

When either pipe edgedrains or PGEDs are used, the trench should be lined with a geotextile to prevent migration of fines from the surrounding soil into the drainage trench; however, the top of the trench adjacent to the permeable base should be left open 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, Christopher, and Berg [1998]). Figure 7.11 shows the placement of a geotextile in a trench, with the CPE pipe laid as well.

Figure 7.11. Geotextile-lined trench with CPE pipe (courtesy of John Donahue, Missouri DOT))

Placement of Drainage Pipes and Backfilling

If a layer of bedding material will be placed prior to placing the drainage pipes, the grooving of the trench bottom has to be done after placing the bedding material. 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 percent (NHI 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 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 150 mm (6 in.) of cover over the drainage pipe is recommended before compacting (NHI 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 to avoid the settlement problem. Generally, several passes of an approved vibrating pad, plate, or compactor are used to consolidate the backfill material, generally seeking a minimum density of 95 percent Standard Proctor (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 Vermeer vibratory wheel (Ford and Eliason 1993). Each pass of the vibratory wheel is effective in achieving the target density to a depth of 300 mm (12 in.). The Minnesota study also showed that the degree of compaction can be verified easily using a DCP.

Automated equipment has been developed that can be used to install either smooth-walled or corrugated plastic pipes. Figure 7.12 shows a schematic of a piece of equipment for placing longitudinal edgedrains, whereas Figure 7.13 shows the equipment in an actual installation process. Productivity for this equipment is about 5 km (3.1 mi) per day.

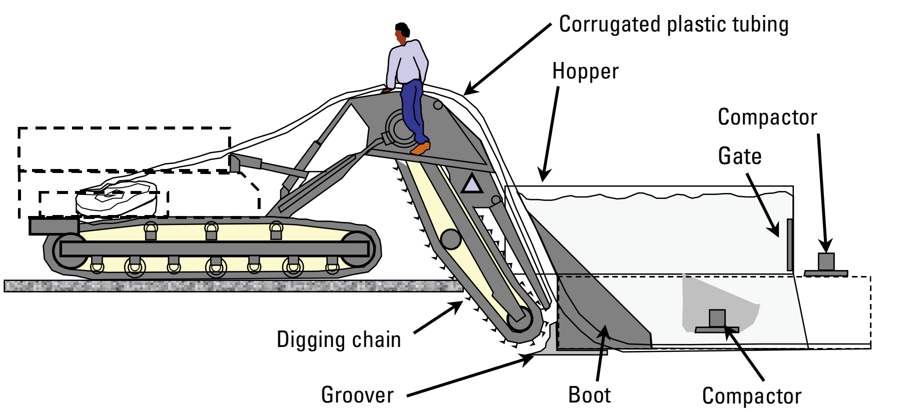

Figure 7.12. Schematic of automated equipment for installing pipe edgedrains (NHI 1999)

Figure 7.13. Automated equipment installing CPE edgedrains (courtesy of Kevin Merryman, Iowa DOT)

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 underdrain pipe. 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. Several example outlets are shown in Figure 7.14.

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 7.15 shows several different types of headwall installations.

Figure 7.14. Lateral outlet pipes (courtesy of John Donahue [left] and Kevin Merryman [right])

Geocomposite Edgedrains

Trenching

The trench should be cut 100–150 mm (4–6 in.) wide and deep enough to place the top of the panel drain 50 mm (2 in.) above the bottom of the pavement surface layer. Typical dimensions for a geocomposite edgedrain consist of an inside cross sectional thickness of 13–25 mm (0.5–1 in.) and a depth of 300–450 mm (12–18 in.).

Installation of the Geocomposite Edgedrain

The drain 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 important (Elfino, Riley, and Baas 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 through the splice, and the splice should not allow infiltration of backfill or any fine material.

Figure 7.15. Various headwall installations

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.

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 152-mm (6-in.) lifts, has been successfully used as backfill material and compacted by flushing or puddling with water (Koerner et al. 1994). To enhance placement around the PGED, the maximum aggregate size should be limited to 19 mm (0.75 in.). 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, Allen, and Harison 1994).

Aggregate Underdrains

Aggregate underdrains are physically cut in at the pavement-shoulder interface using a trencher or backhoe. The dimensions for this type of underdrain vary, but the trench is often about 305 mm (12 in.) wide and at least 203 mm (8 in.) deep, although the depth will depend on the pavement and shoulder thickness and underlying base course thickness. Moreover, 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 will be sloped to the ditch at a grade of at least 8 percent. Figure 7.16 shows a schematic of an aggregate underdrain design.

After cutting the trench, it is recommended that it be lined with an appropriate geotextile material to prevent migration of fines. Sufficient geotextile material should be distributed so that it can totally encapsulate the aggregate material. The specified aggregate underdrain material is then placed in the trench in no more than about 152-mm (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.

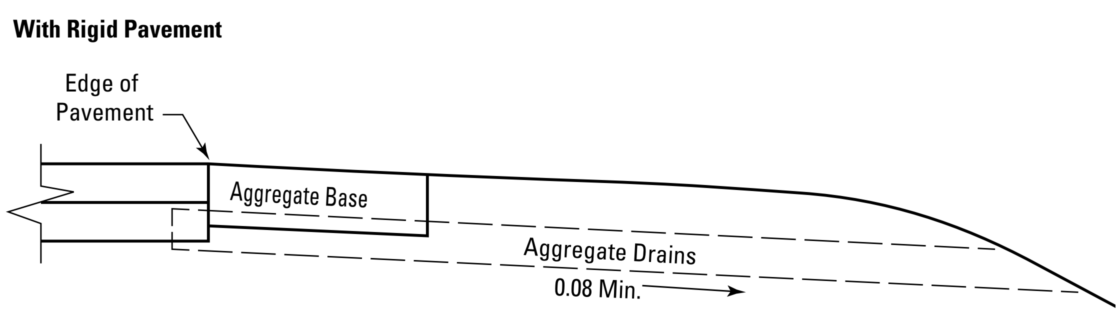

Figure 7.16. Example of aggregate underdrain design from Ohio DOT

7. 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 in order 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 sediment; see Figure 7.17.
- Edgedrains (both pipe drains and fin drains) that are filled with sediment, especially at slopes of less than 1 percent.
- Missing rodent screens at outlets.
- Missing outlet markers.

Figure 7.17. Clogged outlet pipes (courtesy of John Donahue)

- Erosion around outlet headwalls.
- Shallow ditches that have inadequate slopes and that are clogged with vegetation.

Adequate maintenance actually begins in the design stage, when a system is constructed so that it can be adequately maintained. This includes the placement of outlet markers 610–914 mm (24–36 in.) above the ground and suitably marked to locate transverse outlets, using 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 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, 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 the roadside ditches be 0.9–1.2 m (3–4 ft) wide, have a depth 1.2 m (4 ft) below the surface of the pavement, and have a minimum longitudinal slope of 1 percent.)

A major advancement in this area has been the use of video cameras to inspect the condition of drainage systems. Since first promoted by the FHWA in the late 1990s, more than 17 highway agencies report 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, Schwartz, and Boudreau 2006). A study by Daleiden (1998) conducted video inspections of in-service edgedrains to assess their performance and revealed that only 30 percent of the in-service edgedrains were fully functional. The common causes for poor performance of retrofitted pipe edgedrains were discovered to be improper installation, pipe clogging due to fines, and pipe crushing, whereas for PGEDs the causes were due to improper installation (crushed or buckled geocomposite panels) and clogging caused by caking of fines on the geotextile material. After implementing a video camera inspection quality control program, the Kentucky Department of Highways determined that the number of edgedrain failures decreased from 20 percent to less than 2 percent (Fleckenstein and Allen 2000). Figure 7.18 shows photos of a video camera system.

Even when all design parameters are properly evaluated and included in the design, the effect of retrofitted subdrainage on pavement performance may not be as expected, and the potential benefits discussed earlier may not be attainable. An evaluation program that provides feedback data will help the design engineer to determine if there are any aspects of the design that may be detrimental to long-term performance. These programs cannot be short-term evaluations because many moisture-related distresses take time to develop.

Figure 7.18. Video system (top) and camera head (bottom) (NHI 1999)

8. Summary

Pavement engineers are often faced with older concrete pavements that are displaying moisture-related damage, which may be attributed to a combination of inadequate initial drainage, subsurface drainage system damage, 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 reduced pavement deterioration to a detrimental effect on a few projects. The cases of poor performance have generally been attributed to inappropriate use, improper construction or installation, or lack of maintenance. Complicating matters is that design engineers must work with existing pavement materials, which may have limited drainability, but 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 using 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 water affecting pavement performance is surface infiltration.
- The pavement is less than 15 years old.
- The base material has less than 15 percent material passing the 0.075-mm (#200) sieve.
- The pavement is in relatively good condition (i.e., there are limited signs of severe moisture damage and the pavement contains less than 10 percent cracked slabs).

A variety of edgedrain systems has been used on retrofitted drainage projects, with each having slightly different characteristics. Prefabricated geocomposite edgedrains and aggregate underdrains are less expensive to install but can be difficult to maintain (i.e., they are nearly impossible to clean if they become clogged). Typically geocomposite drains have lower hydraulic capacities than pipe drains, although newer materials are changing this trend. Pipe edgedrains, on the other hand, have higher hydraulic capacities but are more expensive. Aggregate underdrains have been used by some agencies for localized drainage improvements on pavement projects. Proper construction and installation of these systems is important to ensure their long-term effectiveness.

The performance experiences of a number of highway agencies highlight the need for regular maintenance of edgedrain systems. This begins with regular inspection and monitoring and also includes such items as installing and maintaining reference markers at outlet locations, clearing debris and vegetation at outlets, and flushing/rodding the edgedrain system. 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

Element	Consideration
Project Selection	• This is most appropriate on existing pavements with moisture-related distresses (pumping, faulting) but little cracking or other signs of structural deterioration (less than 10 percent of slabs exhibit cracking). • The existing base should have less than 15 percent fines (material passing #200 sieve). • The geometrics of the project must be acceptable (in terms of the transverse and longitudinal slopes). • This may also be used only in localized areas (and not over entire project) where specific moisture problems exist.
Design	• Anticipated water outflow levels that can be realistically removed from pavement must be determined (FHWA DRIP program). • Geotextile material is selected based on base and subgrade materials. • Edgedrains must be properly sized and placed in the proper location (horizontal offset and vertical location). • Effective backfill material must be selected with proper gradation for existing pavement. • Outlets spacing is determined for projected outflow and slopes of project (typically 250 to 300 ft). • Proper elbow radii are selected for outlet pipes to facilitate cleaning.
Construction	• Pipe drains are placed in proper vertical and horizontal location, while PGEDs are placed against the shoulder side of the trench. • Aggregate drains are placed at or below the bottom of the pavement base. • Backfill material is placed to avoid damaging pipe drains or PGEDs and carefully compacted. A minimum 6-inch cover is recommended over the drainage pipe before compacting. • Rigid outlet pipes are installed, hooked up to collector pipes, and placed at least 6 inches above the 10-year ditch flow line (or 10-year water level in the storm drain system). • Headwalls are installed for each outlet location.
Maintenance	• Drainage outlets are marked, mowed around, and inspected for condition and functionality. • Headwalls are inspected and maintained. • Video inspection of pipe drains is performed regularly, and the system is flushed as needed. • Vegetation and debris are removed from pipe outlets, daylighted edges, and ditches. • Ditches should have adequate slopes and depths. It is generally recommended that the roadside ditches be 0.9–1.2 m (3–4 ft) wide, have a depth 1.2 m (4 ft) below the surface of the pavement, and have a minimum longitudinal slope of 1 percent.

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Chapter 8

Dowel Bar Retrofit, Cross Stitching, and Slot Stitching

1. Learning Outcomes

This chapter presents information on dowel bar retrofit (DBR), cross stitching, and slot stitching of joints and cracks in concrete pavements. Upon completion of this chapter, the participants will be able to accomplish the following:

- List benefits and applications of DBR, cross stitching, and slot stitching.
- Describe recommended materials and mixtures used in the DBR, cross-stitching, and slot-stitching processes.
- Describe recommended construction procedures.
- Identify typical construction problems and remedies.

2. Introduction

Dowel bar retrofit is the installation of dowel bars at existing transverse joints or cracks in order to effectively transfer wheel loads across slabs and reduce deflections. In this process, dowel bars are retrofitted into the joints of existing concrete pavements either that do not have load transfer devices or in which the existing devices are no longer functional. Dowel bar retrofit is also an effective means of providing positive load transfer across random transverse cracks.

Doweled concrete pavements normally exhibit adequate load transfer, but nondoweled JPCPs typically show lower levels of load transfer because they rely on aggregate interlock of the abutting joint faces for load transfer. Aggregate interlock is only effective if the opposing joint faces remain in close contact, with openings of less than 0.6 mm (0.025 in.) (Kelleher and Larson 1989). 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. Furthermore, it is also possible that the load transfer devices on an existing pavement may have become ineffective, and the pavement would benefit from the addition of new dowel bars at the transverse joints (and placed between the existing dowels).

Restoration of load transfer by installing dowel bars is expected to enhance pavement performance by reducing pumping, faulting, and corner breaks, and also by retarding the deterioration of transverse cracks. In most instances, the pumping and faulting mechanism can be corrected by DBR. Diamond grinding of the pavement surface is often done in conjunction with DBR to minimize dynamic impact loading caused by faulting and to restore rideability.

Two 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 longitudinal cracks and longitudinal joints 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 tiebar into the drilled hole. Slot stitching, on the other hand, is similar to DBR in that a deformed tiebar is grouted into slots cut across a nonworking longitudinal joint or crack. Currently, cross stitching is the more commonly used treatment.

This chapter presents information associated with using DBR, cross stitching, and slot stitching as effective pavement preservation techniques for concrete pavements. 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.

3. Purpose and Project Selection

LTE

In order to select good candidate projects for DBR, it is first important to understand the concept of LTE and how it is measured. Load transfer efficiency is a quantitative measurement of the ability of a joint or crack to transfer load from one side to the next. 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 on existing pavements with an FWD. The most common mathematical formulation for expressing deflection LTE is

$$LTE = \frac{\Delta_{UL}}{\Delta_L} \times 100 \qquad (8.1)$$

where:

LTE = Load transfer efficiency

Δ_{UL} = Deflection stress on the unloaded side of the joint

Δ_L = Deflection stress on the loaded site of the joint

The concept of deflection load transfer is illustrated in Figure 8.1. 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 zero percent. 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 percent.

Load transfer efficiency should be measured during cooler temperatures (ambient temperatures less than 21°C [70°F]) and during the early morning when the joints will not be 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. Load transfer efficiency is generally measured either in the outer wheelpath, which is subject to the higher repeated truck traffic load applications, or at the slab corner, which is the location with the highest deflection potential and often considered the more critical location. Deflection measurements for the determination of LTE should be taken with sensors placed as close to the joint or crack as possible.

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 lead to pumping of the underlying base course material, faulting, and perhaps even corner breaks. A useful parameter to help assess this is the differential deflection (DD), which is the relative displacement between the loaded and unloaded sides of the joint and is computed as follows:

$$DD = \Delta_L - \Delta_{UL} \tag{8.2}$$

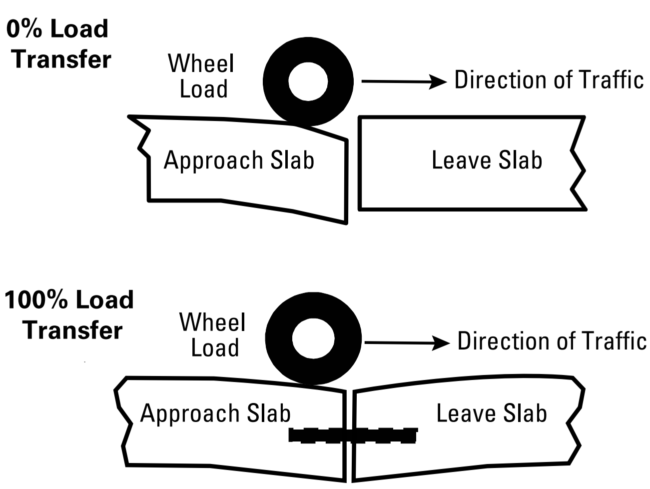


Figure 8.1. Illustration of deflection load transfer concept

The DD should be computed along with the LTE over a project to gain a more complete understanding of the load transfer characteristics of a joint or crack. It is suggested that DDs be limited to 0.13 mm (5 mils) or less (Odden, Snyder, and Schultz 2003; Snyder 2011) and that peak corner deflections be limited to 0.63 mm (25 mils) or less (Snyder 2011).

Selecting Candidate Projects for DBR

The following are general characteristics associated with good candidate pavements for DBR (FHWA/ACPA 1998):

- Pavements with structurally adequate slab thickness, but exhibiting low load transfer due to lack of dowels, poor aggregate interlock, or base/subbase/subgrade erosion.
- Relatively young pavements that, because of insufficient slab thickness, excessive joint spacing, inadequate steel reinforcement at transverse cracks, and/or inadequate joint load transfer, are at risk of developing faulting, working cracks, and corner cracks unless load transfer is improved.

In general, the pavement should be in relatively good condition with a limited amount of structural cracking (Bendaña and Yang 1993). Pavements exhibiting significant slab cracking, joint spalling, or MRD (such as ASR or D-cracking) should not be considered candidates for DBR (Larson, Peterson, and Correa 1998).

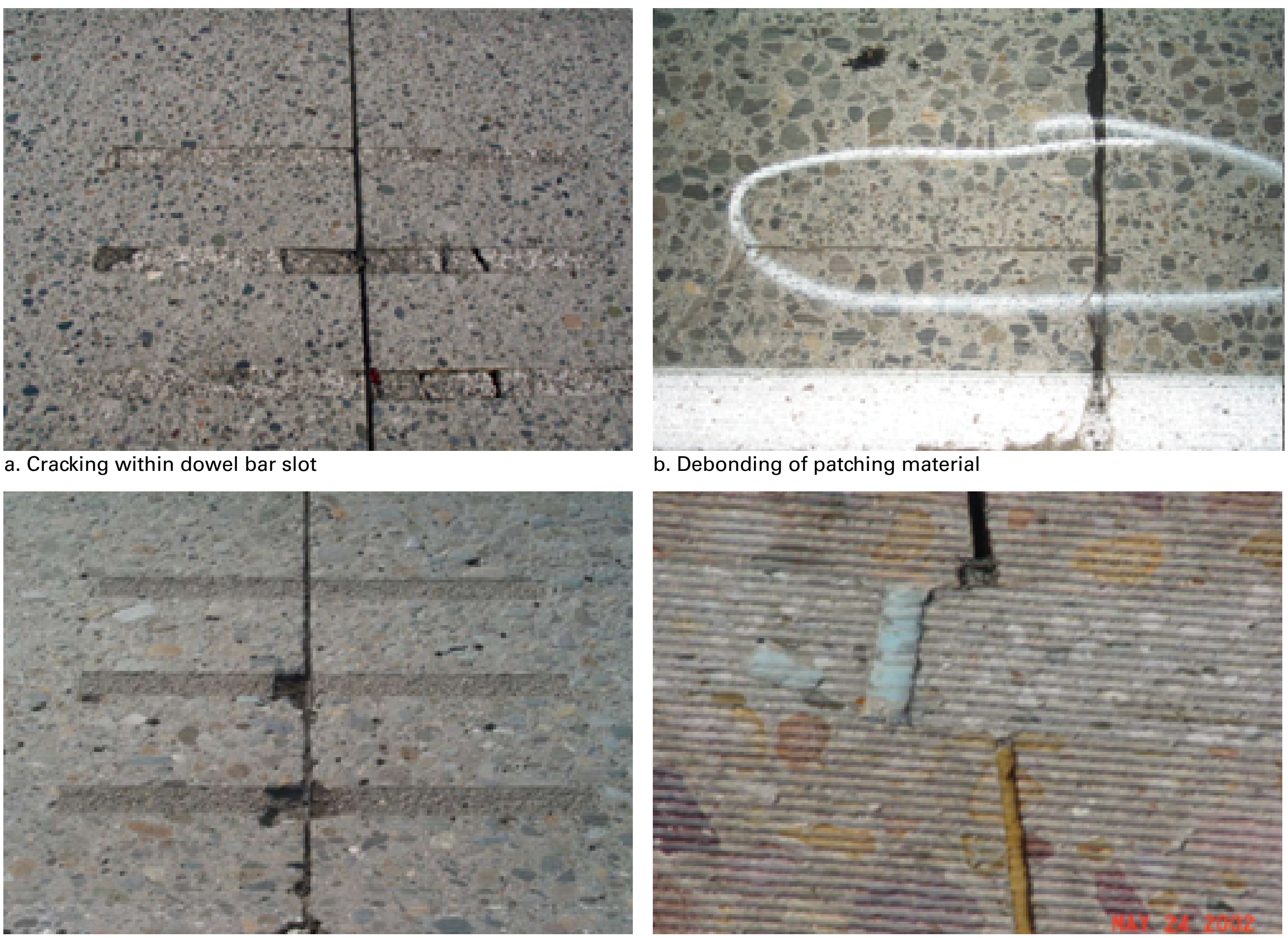
One set of recommendations on the condition of a joint or crack suitable for DBR is that it exhibits a deflection load transfer of 60 percent or less, faulting greater than 2.5 mm (0.10 in.) but less than 6 mm (0.25 in.), and differential deflection of 0.25 mm (0.01 in.) (FHWA/ACPA 1998). The recommendation from the Washington State DOT is that DBR should be considered on pavements that have an average faulting between 3 mm (0.125 in.) and 13 mm (0.5 in.) and when the number of panels with multiple cracks is 10 percent or less (Pierce et al. 2003). Caltrans (2006) has similar requirements as the Washington State DOT, and it also includes DD (0.25 mm [10 mils] or more) and IRI (levels between 2.3 and 3.2 m/km [150 and 200 in./mi]) as additional considerations.

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) and in preparation for an overlay. In the former DBR helps to maintain structural integrity and improves ride quality, whereas in the latter DBR can help reduce the incidence and severity of reflection cracking, spalling, and deterioration of the overlay (and may also result in a thinner overlay thickness).

4. Limitations and Effectiveness

Dowel bar retrofit is not a new rehabilitation technique. As early as 1980, the Georgia DOT evaluated a number of different load transfer restoration devices (e.g., dowels, figure eight, Georgia Split Pipe, vee, and double vee) on a project on I-75, the results of which clearly indicated that DBR was the better-performing technique (Gulden and Brown 1985; Gulden and Brown 1987). Additionally, Puerto Rico started using dowel bars as early as 1983 and has had good performance (Larson, Peterson, and Correa 1998). In the ensuing years, many highway agencies have implemented DBR as an effective pavement preservation technique, and at present at least 19 state highway agencies have standard specifications and standard plans for DBR (IGGA 2012).

The Washington State DOT has a DBR program that dates back to 1992 (Pierce 1994; Pierce 1997), and it has retrofitted more than 451 lane-km (280 lane-mi) of concrete pavement (Pierce 2009). In a review of approximately 290 lane-km (180 lane-mi) of retrofitted concrete pavement (representing approximately 380,000 dowel bar slots), it was noted that less than 10 percent of the DBR slots exhibited any form of distress (after 2 to 14 years of service) (Pierce and Muench 2009). Figure 8.2 illustrates some of the more common types of distresses observed on the DBR installations.



a. Cracking within dowel bar slot

b. Debonding of patching material

c. Spalling within dowel bar slot

d. Misaligned foam core board

Figure 8.2. Typical dowel bar slot distress (Pierce and Muench 2009)

From 1994 to 1999, the Minnesota DOT 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 faulting of mid-panel cracks and in extending the service life of nondoweled concrete pavements (Burnham and Izevbekhai 2009).

Although there has been good documented success with this technique, a few states have experienced some problems with their initial DBR trials. For example, on some of its early projects, the Wisconsin DOT found that the patching material used to backfill the slots was deteriorating at some of the joints (Bischoff and Toepel 2002). In response to these observed material problems, the Wisconsin DOT developed modified patching materials to reduce unwanted shrinkage (Bischoff and Toepel 2004). Because of the sensitivity of patching materials to loading and environmental conditions, it is extremely important to test and modify (as necessary) patching materials in the laboratory and on test sections prior to using them on a wider scale in the field.

The North Dakota DOT constructed several DBR test sections and found that distress within the dowel bar slot appears 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 board vertical within the dowel bar slot, the North Dakota DOT requires the use of a notched foam core board insert; see Figure 8.3. The notched foam core board is also required by the California and Idaho DOTs.

Based on these and other highway agency experiences, it is important that the DBR treatment be targeted to the proper pavement (one that is not exhibiting significant structural deterioration). Furthermore, the installation of DBR must be viewed as a system, and each part of that system—from the slot preparation to the materials to the patching and consolidation—is critical to the long-term performance of DBR projects. Additionally, for maximum effectiveness, the application of other preservation treatments, such as crack sealing, joint resealing, and slab stabilization, may need to be considered.

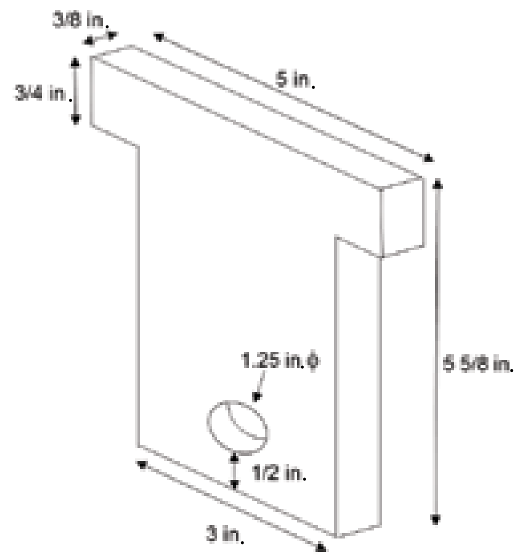


Figure 8.3. Typical notched foam board insert (Dunn and Fuchs 1998)

5. Materials and Design Considerations

When designing a DBR project, it is important to determine what dowel bar size (diameter and length) will be used, what patching material will be used to fill the slots, and where and in what configuration the dowel bars should be placed. This section provides recommendations for some of these key factors.

Patching Material

The patching material is the substance used to encase the dowel bar in the existing pavement. 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 in a reasonable period of time. To aid in this process, many agencies maintain a qualified product list of suitable patching materials.

The patching material is one of the most critical factors in successful DBR projects (ACPA 2006). Generally, materials found to work well for partial-depth repairs also work well as a patching material for DBR (FHWA/ACPA 1998). One of the most important factors to control is the water content of the patching material in order to reduce the probability of shrinkage cracks and debonding (Rettner and Snyder 2001). Table 8.1 summarizes recommended tests and material properties for DBR patching materials, based on a summary of agency specifications.

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 meet local concrete aggregate quality requirements (IGGA 2013). In addition, the coarse aggregate gradation should meet the following sieve size requirements (IGGA 2013):

- 100 percent passing the 3/8 sieve.
- 0 to 15 percent passing the #4 sieve.
- 0 to 5 percent passing the #8 sieve.
- 1.0 percent (maximum) passing the #200 sieve.

Concrete

Concrete can be used as a patching material for DBR. It is cheaper than other 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 9.5 mm (0.375 in.) maximum size are commonly used to extend the yield of the mix.

Fast-Setting Proprietary Materials

Several proprietary materials are available for use as a patching material for DBR. Fast-setting proprietary materials are the predominant patching material used by state highway agencies for DBR projects. The main advantage of these types of materials is that they are quick setting, thereby allowing earlier opening times to traffic. State highway agencies typically maintain a list of approved proprietary products for use in pavement construction. It is strongly recommended that any patching material without an acceptable history of performance under similar conditions of load and environment be tested in the laboratory for specification compliance before being used in the field. Also, it is critical that all manufacturer's instructions be followed when working with these proprietary materials (ACPA 2006).

Table 8.1. Recommended Properties for Patching Materials (IGGA 2013)

Property	Test Procedure	Recommended Value
Compressive Strength	AASHTO T 160	>20.7 MPa (3,000 lbf/in.^{2}) at 3 hours
	ASTM C109	>34.5 MPa (5,000 lbf/in.^{2}) at 24 hours
Scaling	ASTM C672	Visual rating of 2 or less
Shrinkage	ASTM C157	<0.13 percent @ 4 days
Durability Factor	AASHTO T 161 ASTM C666A	>90 percent @ 300 cycles
Bond Strength	ASTM C882	>6.9 MPa (1,000 lbf/in.^{2}) at 24 hours

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 M235, and the manufacturer's recommendations should be closely followed for application and placement.

Dowel Bar Design and Layout

Round, solid steel dowels conforming to AASHTO M31 or ASTM A615 are commonly used for load transfer in concrete pavements. These bars commonly have a fusion-bonded epoxy coating (typically between 0.2 to 0.3 mm [0.008 to 0.012 in.] thick) that provides corrosion protection by acting as a barrier against moisture and chloride intrusion, although other coatings (e.g., plastic) or dowel materials (e.g., stainless steel, fiber-reinforced polymer) have also been used. In addition, the dowels must be coated with a bondbreaker to allow the joint to open and close with changing temperatures; paraffin-based lubricants or form oils are typically specified as bondbreakers, and these can be applied either in the field or at the factory.

The required size of the dowel bars is dependent on the pavement thickness. A minimum dowel bar length of 350 mm (14 in.) is recommended to allow for at least 150 mm (6 in.) of embedment on each side of the joint or crack, adequate room for an expansion cap on each end of the dowel bar, and reasonable placement tolerances (ACPA 2006). A summary of the recommended dowel size requirements for DBR projects is presented in Table 8.2.

In order for the retrofitted dowel bars to be effective, they must be of sufficient size and placed in a suitable configuration. Currently it is recommended that three or four dowels (spaced 300 mm [12 in.] apart) be used in each wheelpath, with the outermost dowel being 300 mm (12 in.) from the lane edge, except where tiebars from adjacent lanes or shoulders are encountered (ACPA 2006). Although the number of dowel bars varies by agency, most specify the use of three dowel bars per wheelpath. An illustration of the recommended dowel bar configuration is presented in Figure 8.4.

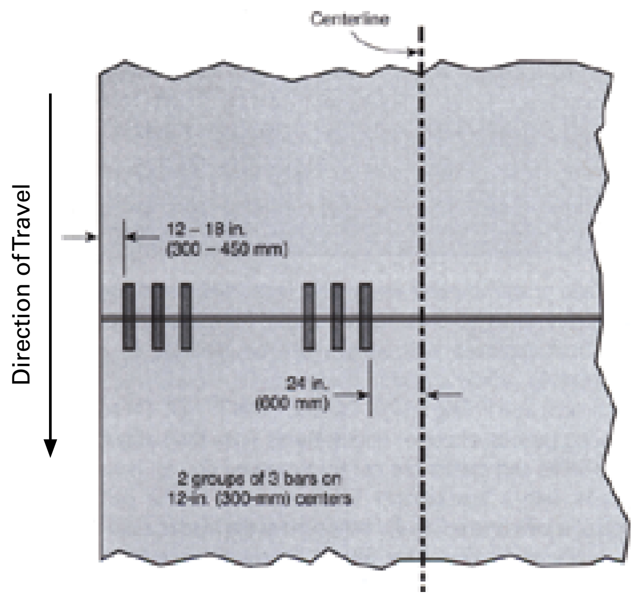


Figure 8.4. Recommended dowel bar configuration (ACPA 2006)

Table 8.2. Dowel Size Requirements for DBR Projects (ACPA 2006)

Pavement Thickness, mm (in.)	Diameter, mm (in.)	Minimum Length, mm (in.)	Spacing, mm (in.)
<200 (8)	25 (1.0)	350 (14)	300 (12)
200 to 240 (8 to 9.5)	32 (1.25)	350 (14)	300 (12)
≥250 (10)	38 (1.5)	350 (14)	300 (12)

A second design consideration is the dimensions of the slots themselves. The slot must be sufficiently long to enable the dowel to lie flat across the bottom without hitting the end of the slot; this typically requires the surface length of the sawcut to be 1 m (3 ft) for a 350-mm (14-in.) long dowel bar (FHWA/ACPA 1998). The width of the slot is typically 65 mm (2.5 in.). 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 13 mm (0.5 in.) beneath the dowel bar for placement on chairs. The bottom of the slot should also be flat and uniform across the joint. Figure 8.5 shows an example illustration of the slot details.

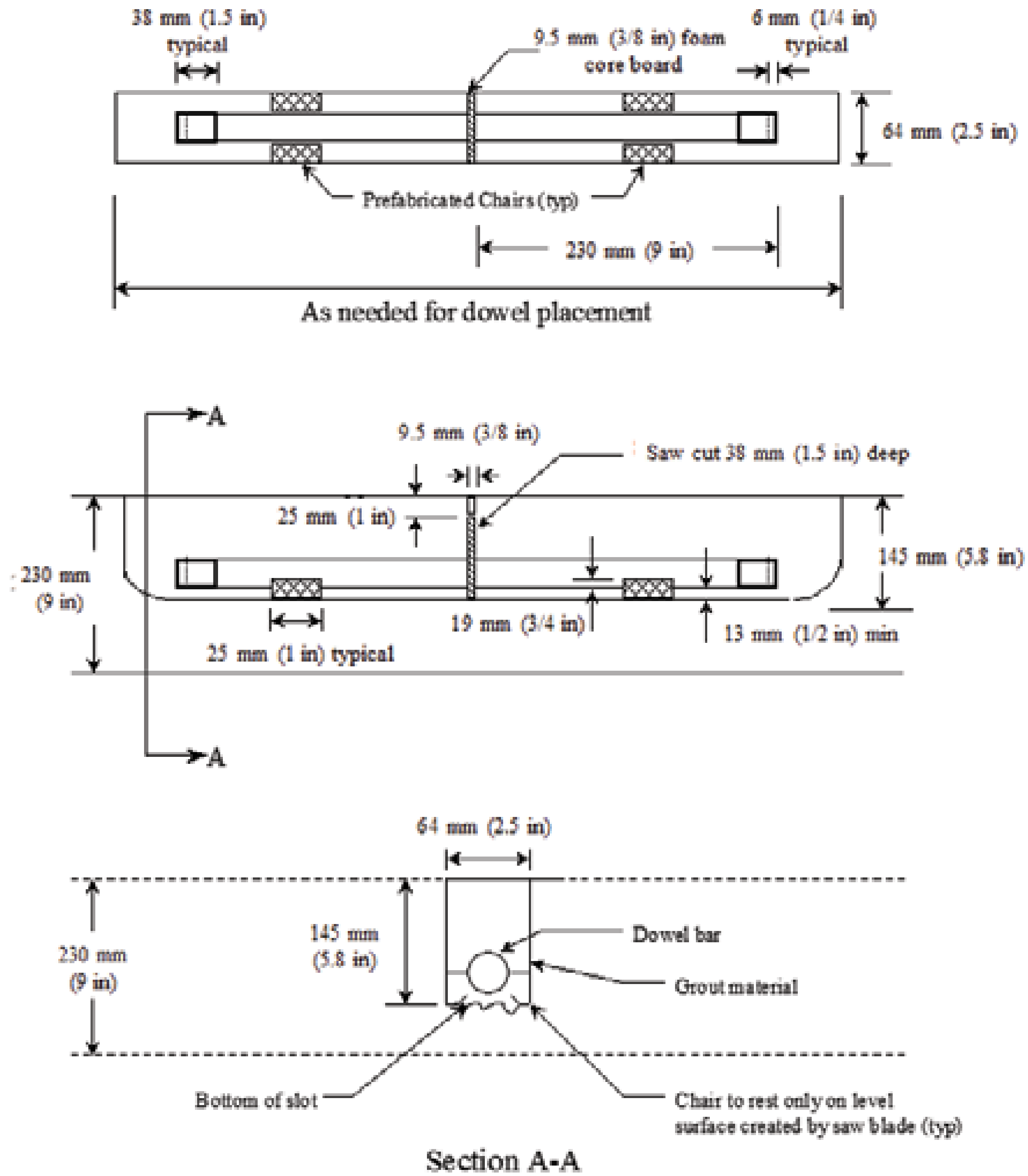


Figure 8.5. Example of DBR installation details (Pierce et al. 2003)

6. Construction Considerations

The completion of a DBR project involves the steps listed below, which are described in more detail in the following sections:

1. Test section.
2. Slot creation.
3. Slot preparation.
4. Dowel bar placement.
5. Patching material placement.
6. Diamond grinding (optional).
7. Re-establishment of joint and joint sealing.

Step 1: Test Section

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 Dimension**—The test section 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 100 mm [4 in.] diameter) to assess the completeness of slot removal, the effectiveness of the dowel bar installation, and the level of consolidation achieved on the patching material. In addition, agencies may require that FWD testing be performed to verify the effectiveness of the DBR installation.

Step 2: Slot Creation

The recommended method of creating slots for DBR projects is with a diamond-bladed slot-cutting machine; see Figure 8.6. Whereas modified milling machines have been occasionally used in the past to create slots, they produce excessive spalling and do not provide consistent slot dimensions; they are therefore not recommended (ACPA 2001a).

Diamond-bladed slot-cutting machines make two parallel cuts in the pavement for each dowel slot; the "fin" area between the cuts is then broken up with a light jackhammer. Diamond-bladed slot-cutting machines have been developed that can cut either three—see Figures 8.7 and 8.8—or six slots (in one or two wheelpaths) at the same time (FHWA/ACPA 1998). Production rates for this method of slot cutting can exceed 2,500 slots per day. It is important that the slots be parallel to the centerline of the pavement and that the resulting slots be cut to the prescribed depth, width, and length at the required spacing.

Figure 8.7. Example of diamond-bladed slot-cutting machine—three dowel bar slots per wheelpath (Pierce, Uhlmeyer, and Weston 2009)

Figure 8.6. Example of diamond-bladed slot-cutting machine (Pierce, Uhlmeyer, and Weston 2009)

a. Nonskewed transverse joints

b. Skewed transverse joints

Figure 8.8. Example of cut dowel bar slots (Pierce, Uhlmeyer, and Weston 2009)

Step 3: Slot Preparation

After the sawcuts have been made, lightweight jackhammers (less than 14 kg [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 8.9. After removing the concrete wedge, the bottom of the slot should be flattened with a small hammerhead mounted on a small jackhammer; see Figure 8.10.

Once the jackhammering operations are completed, the slots are thoroughly sandblasted to remove dust and sawing slurry and to provide a slightly roughened surface to promote bonding; see Figure 8.11a. Final cleaning through air blasting, as shown in Figure 8.11b, is performed on the slot immediately before the dowel and patching material are placed.

Figure 8.12 illustrates the dowel bar slot prior to and immediately following slot cleaning. Figure 8.12a shows the debris and slurry buildup from the saw-cutting operation on the sides of the dowel bar slot, whereas Figure 8.12b shows the cleaned vertical faces of the slot surface just prior to placement of the patching material. The side of the dowel bar slot is considered clean when wiping the sides of the slot with a clean towel results in no residue buildup (Pierce, Uhlmeyer, and Weston 2009).

b. Air blasting immediately prior to dowel bar placement

a. Sandblasting to remove residue

Figure 8.11. Preparing dowel bar slot (Pierce, Uhlmeyer, and Weston 2009)

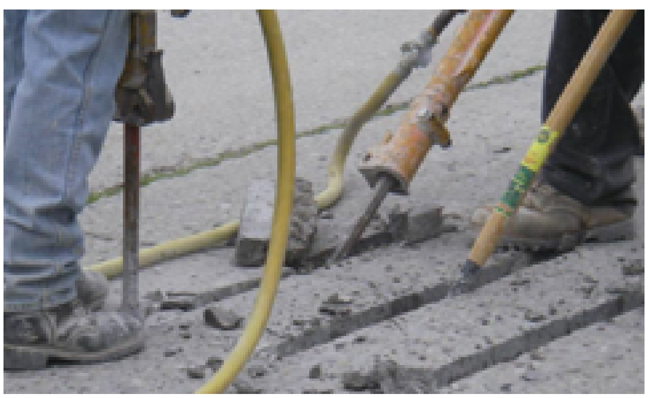


Figure 8.9. Operating jackhammers at no more than a 45-degree angle (Pierce, Uhlmeyer, and Weston 2009)

Figure 8.10. Leveling the bottom of the dowel bar slot (Pierce, Uhlmeyer, and Weston 2009)

a. Debris and saw blade slurry on side of slot

b. Final cleaned slot surface

Figure 8.12. Before and after slot cleaning (Pierce, Uhlmeyer, and Weston 2009)

After cleaning, the joint or crack in the slot is caulked with a silicone sealant prior to dowel bar placement to prevent intrusion of any patching material that might cause a compression failure, as shown in Figure 8.13. The sealant should not extend 13 mm (0.5 in.) away from the joint because this excessive sealant will detract from the patching material bonding to the sides of the slot.

Figure 8.13. Applying caulk on slot sides and bottom (Pierce, Uhlmeyer, and Weston 2009)

Step 4: Dowel Bar Placement

The dowel bars should be coated with a bondbreaking material (e.g., curing compound or a manufacturer-supplied material) along their full length to facilitate joint movement. Expansion caps are placed at both ends of the 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) and positioned in the slot so that the dowel rests horizontally and parallel to the centerline of the pavement at mid-depth of the slab. The proper alignment of the dowel bar is critical to its effectiveness. 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 8.14 illustrates the placement of the dowel bars into the slots.

Figure 8.14. Placing dowel bar assembly into the slot (Pierce, Uhlmeyer, and Weston 2009)

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 the slot and then shoveled into the slot, as shown in Figure 8.15.

A small spud vibrator (i.e., ≤25 mm [1.0 in.] in diameter) should be used to consolidate the patching material; see Figure 8.16a. After the material is consolidated, it should be finished with a trowel such that it is flush with the existing pavement surface; see Figure 8.16b. While finishing, the patching material in the dowel bar slots should not be overworked, which would otherwise 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 to minimize shrinkage; see Figure 8.16c. Depending upon the type of patching material, the pavement may be opened to traffic in a few hours. The minimum compressive strength required to open newly placed concrete to traffic is approximately 13.7 MPa (2,000 lbf/in.^{2}) for slabs 200 mm (8 in.) or thicker (FHWA/ACPA 1998).

After the patching material has gained sufficient strength, the transverse joint or crack should be re-established using saws, as shown in Figure 8.17. Typically, sawcutting of the patching material at the transverse joint or crack should occur within 24 hours after placement. Re-establishing the transverse joint will minimize the potential of spalling of the patching material.

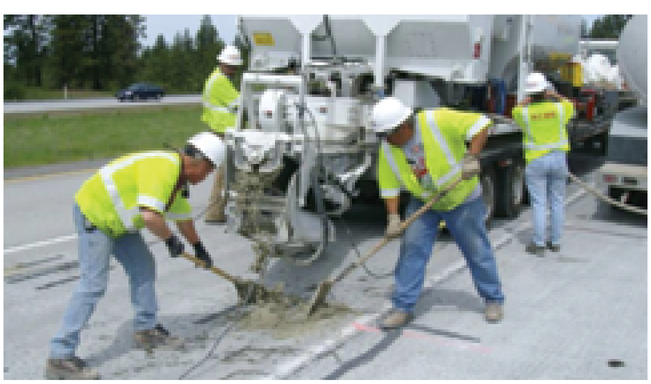
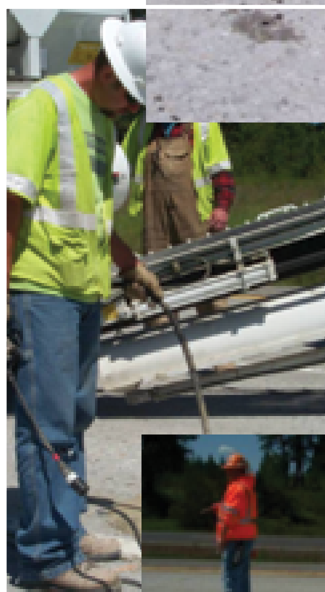


Figure 8.15. Placing patching material into the dowel bar slot (Pierce, Uhlmeyer, and Weston 2009).

b. Finishing the patching material



a. Consolidating the patching material

c. Applying curing compound

Figure 8.16. Patching material placement (Pierce, Uhlmeyer, and Weston 2009)

Figure 8.17. Re-establishing the transverse joint in patching material (Pierce, Uhlmeyer, and Weston 2009)

Step 6: Diamond Grinding (Optional)

Rehabilitation techniques such as DBR may result in increased roughness if not finished properly. This is typically due to differences in elevation between the finished dowel bar slots and the existing pavement, or perhaps 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 provide a smooth-riding surface. Chapter 9 provides detailed information on diamond grinding.

Step 7: Joint Sealing

After diamond grinding, the transverse joints should be prepared and sealed in accordance with agency policies. Chapter 10 provides detailed information on joint sealing.

7. 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 this quality during construction greatly increases the chances of minimizing premature failures on DBR projects. A comprehensive set of design and construction recommendations for DBR, based on more than 10 years of experience, is available from the Washington State DOT (Pierce et al. 2003). This section summarizes some of the critical recommendations for successful DBR projects provided in that document as well as in the FHWA's *Dowel-Bar Retrofit for Portland Cement Concrete Pavements Checklist* (FHWA 2005).

Preliminary Responsibilities

Agency and contractor personnel should collectively conduct a review of the project documentation, project scope, intended construction procedures, material usage, and associated specifications. Such a collective review is intended to minimize any misunderstandings in the field between agency designers, inspectors, and construction personnel. 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 DBR. Specifically, the following items should be verified or checked 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.
- Check estimated quantities for DBR.

Document Review

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.
- Agency application requirements.
- Traffic control plan.
- Manufacturer's installation instructions for patching materials.
- MSDS.

Review of Materials

In preparation for the construction project, the following list summarizes many of the material-related checklist items that should be checked or reviewed:

- Verify that dowel slot cementing grout meets specification requirements.
- Verify that the dowel slot cementing grout is from an approved source or listed on the agency qualified products list (QPL) (if required).
- Verify that the component materials 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, with acceptable quality.

- Verify that the material packaging is not damaged (i.e., 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 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, 10 mm [0.38 in.] thick).
- Verify that the joint sealant material meets specification requirements.
- Verify that all sufficient quantities of materials are on hand for completion of the project.
- Ensure that all material certifications required by contract documents have been provided to the agency prior to construction.

Inspection of Equipment

Prior to beginning 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, horsepower, and configuration to cut the specified number of slots per wheelpath to the depth shown on the plans.
- Verify that jackhammers for removing concrete are limited to a maximum rated weight of 14 kg (30 lb).
- Verify that the sandblasting unit for cleaning slots is adjusted for correct sand rate and has oil and moisture filters/traps.
- Verify that air compressors have sufficient pressure and volume to adequately remove all dust and debris from slots and meet agency requirements.
- 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 (e.g., mobile mixers) are kept in good condition and calibrated on a regular basis to properly proportion mixes.
- Ensure that material test equipment required by the specifications is all available on-site and in proper working condition (e.g., slump cone, pressure-type air meter, cylinder molds and lids, rod, mallet, ruler, 3-m [10-ft] straightedge).
- Verify that vibrators are the size specified in the contract documents (typically 25 mm [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 is available on the project site specifically designated for the storage of concrete cylinders.

Weather Requirements

The weather conditions at time of construction can have a large impact on the performance of the DBR technique. Specifically, the following weather-related items should be checked immediately prior to construction:

- Review manufacturer installation instructions for requirements specific to the patching material used.
- Confirm that the air and surface temperature meet manufacturer and all agency requirements (typically 4°C [40°F] and above) for concrete 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 checked or verified:

- Verify that the signs and devices used match the traffic control plan presented in the contract documents.
- Verify that the setup complies with the Federal or local agency MUTCD or local agency procedures.

- Verify that flaggers are trained/qualified according to contract documents and agency requirements.
- Verify that unsafe conditions, if any, are reported to a supervisor.
- Ensure that traffic is not opened to the repaired pavement until the patching material has attained the specified strength required by the contract documents.
- Verify that signs are removed or covered when they are no longer needed.

Project Inspection Responsibilities

Cutting Slots

During the slot creation construction step, the inspector should ensure the following:

- Slots are cut parallel to each other and to the centerline of the roadway within the maximum tolerance permitted by the contract documents, typically 6 mm (0.25 in.) per 300 mm (12 in.) of dowel bar length.
- The number of slots per wheelpath (typically three or four) is in agreement with contract documents.
- Slots are aligned to miss any existing longitudinal cracks, as well as any existing tiebars and dowel bars.
- The cut slot length extends the proper distance on each side of the joint/crack, as required by the contract documents. This is especially important for skewed joints and cracks.
- Slots are sawed sufficiently deep so that the center of the dowel bar is positioned at the mid-depth of the pavement. Slots that are cut too deep will contribute to corner cracks when traffic loads are applied.
- Slot widths should be sized to be the exact width of the dowel bar chairs.

Removing Material from Slots

It should be verified that concrete fin removal is conducted with only lightweight 14 kg (30 lb) jackhammers. During the process of removing material from the slots, the contractor should take extra care to prevent the jackhammer from punching through the bottom of the slot. The bottom of the slots should then be smoothed and leveled using a lightweight bush hammer.

Slot Cleaning and Preparation

The following should be closely inspected when cleaning the slots and the adjacent area and preparing the slots prior to the placement of the dowels:

- After concrete removal, the slots should be prepared by sandblasting. A physical check of the slot's cleanliness (using a tool such as a scraper) should be made to ensure no slurry residue remains on the sides of slots.
- After sandblasting, the slots should be cleaned using air blasting. A second air blasting may be required immediately before placement of dowel slot cementing grout if slots are left open for a duration exceeding that permitted in the contract documents.
- Concrete chunks, dirt, debris, and slurry residue should be cleaned 1–1.2 m (3–4 ft) away from the slot's perimeter. This practice minimizes the possibility of reintroducing unwanted material into the slot during subsequent operations.
- The existing joint/crack is sealed with an approved caulking filler material along the bottom and sides of the slot to prevent the patching material from entering the joint/crack. Special care must be taken to ensure that the sealant does not extend 13 mm (0.5 in.) away from the joint (i.e., into the slot).
- If an epoxy resin is used, the patching material should be placed while the epoxy resin is still tacky. If the epoxy resin sets prior to placement, the epoxy resin material should be removed, the slot recleaned, and the epoxy resin applied again.

Placement of Dowel Bars

During the placement of dowels into the cut slots, construction inspections should ensure the following:

- Plastic end caps are placed on each end of the dowel bar to account for pavement expansion as required by the contract documents.
- Dowel bars are completely coated with an approved compound prior to placing into chairs. Dowel bars that have a factory-applied coating should be free of dirt, debris, nicks, and abrasions. The factory-applied coating should be clearly visible; otherwise, an additional application of an approved material must be applied. Dowel bars should not be coated once they have been placed in the slots because the sides and bottom of the slots will become contaminated.

- Proper clearance is maintained between the supported dowel bar and the sidewalls, ends, and bottom of the cut slot in accordance with contract documents.
- Joint forming material (foam core insert) is placed at mid-point of each bar and in line with the joint/crack, to allow for expansion and to reform the joint/crack.
- The chairs placed on the dowel bars are strong enough to allow full support of the dowel bar. Chairs should allow at least 13 mm (0.5 in.) clearance between the bottom of the dowel and the bottom of the slot.
- End caps allow at least 6 mm (0.25 in.) of movement at each end of the bar. End caps placed on each end of the bar reduce the risk of dowel bar lock-up at negligible extra cost.
- Dowels are centered across the joint/crack such that at least 150 mm (6 in.) of the dowel extends on each side.
- Dowels are placed within the tolerances listed below. Dowel bars placed outside of the acceptable tolerances could potentially cause joint lock-up that leads to cracking.
    - Placed within 25 mm (1 in.) of the center of the existing pavement thickness.
    - Centered over the transverse joint with a minimum embedment of 150 mm (6 in.).
    - Placed parallel to the centerline and within the plane of the roadway surface.
    - Placed to a horizontal tolerance of ±13 mm (0.5 in.), vertical tolerance of ±13 mm (0.5 in.), and skew from parallel of ±6 mm (0.25 in.) per 300 mm (12 in.).

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:

- Patching materials are being mixed in accordance with the material manufacturer's instructions.
- Patching materials are mixed in small quantities to prevent premature setting.
- Concrete surfaces, including the bottom of the slot, are dry.
- Material is consolidated using small, hand-held vibrators, which do not touch the dowel bar assembly during consolidation. Inspectors should also ensure that the grout material is not overconsolidated. Each slot should only require two to four short, vertical penetrations of a small-diameter spud vibrator.
- Patching material is finished flush with surrounding concrete, using an outward motion to prevent pulling material away from patch boundaries. Material is finished slightly "humped" if diamond grinding is to be employed.
- Transverse joint is re-established in patching material within 24 hours of placement.
- Adequate curing compound is applied immediately following finishing.

Cleanup

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

If diamond grinding is specified for use in combination with a DBR project, the grinding should be completed within 30 days of the placement of the patching material.

Resealing Joints/Cracks

Inspectors should ensure that the joints/cracks are resealed after diamond grinding (if specified) in accordance with agency specifications.

8. Troubleshooting

Some of the more common problems that a contractor or inspector may encounter in the field during construction are summarized in Table 8.3, along with recommended solutions. Table 8.4 summarizes potential performance problems that may be observed shortly after the project is completed and opened to traffic, along with recommended solutions.

Table 8.3. DBR-Related Construction Problems and Associated Solutions (FHWA 2005; ACPA 2006)

Problem	Typical Cause(s)	Typical Solution(s)
Slots are not cut parallel to the roadway centerline.	There is improper alignment of slot cutting machine.	Misaligned dowels can cause joint/crack lock-up 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 been removed, fill the sawcuts with an epoxy resin and recut at different locations). The use of a multiple saw slot-cutting machine can ensure that slots are parallel to each other.
Dowel bar slots are too shallow.	There have been 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 deeper, remove the concrete to the proper depth, and complete as specified.
Dowel bar slots are too deep.	• There have been improper slot-cutting techniques. • There is improper jackhammer weight. • There has been improper jackhammering technique.	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: 1. Use a lightweight jackhammer (14 kg [30 lb]). 2. Do not lean on the jackhammer. 3. 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. 4. Stop chipping within 50 mm (2 in.) of the bottom of the pavement.
Concrete fin is not easily removed.	Concrete could contain 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.	There has been improper jackhammering technique or extremely deteriorated concrete.	Make an FDR across the entire lane width at the joint/crack.
There are areas on the dowel where the factory-applied dowel coating is missing.	There has been nonuniform application of the factory-applied dowel coating or mishandling of dowels in the field.	Areas of exposed steel can become concentrated points for corrosion that can eventually lead to the lock-up of the dowel. If observed, recoat dowel with 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 bottom of the slots may become contaminated.
Dowel cannot be centered over joint/crack because slot does not extend far enough.	There has been improper slot preparation.	Chip out additional slot length with a jackhammer to facilitate proper placement of the dowel in accordance with contract documents. Typically at least 150 mm (6 in.) of each 350 mm (14 in.) dowel extends on each side of the joint/crack. Properly sized chairs will fit snugly into the slot.
Joint/crack caulking filler material in the joint does not extend all the way to the edge of the slot.	There has been improper caulk installation.	Improperly placed caulking in the joint can allow incompressible patching material to enter the joint, therefore 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 joint or crack extrudes into a slot more than 13 mm (0.5 in.).	There has been 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.	• The vibrator contacted the dowel assembly. • There has been overvibration of material. • There is improper width of the slots.	1. Do not allow the vibrator to touch the dowel assembly. 2. Check for overvibration; each slot should require only two to four short, vertical penetrations of a small diameter spud vibrator. 3. Ensure that the slots are sized the exact width of the plastic dowel bar chairs.

Table 8.4. Potential DBR-Related Performance Problems and Prevention Techniques

Problem	Typical Cause(s)	Typical Solution(s)
Cracking of in-place patching material	• Joint is not well isolated. • Dowels are not all properly aligned. • Patching material is too strong. • Patch was opened to traffic too soon. • Material was used that encountered too much shrinkage.	Confirm that proper construction practices are followed and patching material used is resistant to cracking.
Pop out of patching material	• Slot is not properly cleaned or prepared. • There has been improper curing (i.e., unexpected material shrinkage during curing).	Verify that proper construction procedures are followed.
Wearing off of patching material	Some materials are not very durable or don't perform well if not properly mixed and handled.	Check material specifications, material preparation, and placement conditions to be sure that material is being handled properly.

9. 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 grouting tiebars into holes drilled across the joint or crack at angles of 35° to 45° 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 U.S. highway by the Utah DOT in 1985 (ACPA 2001b). Utah DOT engineers used cross stitching to strengthen uncontrolled cracks on a new 229-mm (9-in.) JPCP design on I-70 in central Utah. Considerable reflection cracking from the 102-mm (4-in.) lean concrete base occurred soon after construction. The cracks of major concern were the longitudinal cracks in or near the wheelpaths of the driving lanes. After 15 years of service, a review of this project found the pavement to be in generally good condition, with some faulting across nondoweled transverse contraction joints (IGGA 2010). The performance of cross-stitched cracks was favorable in most areas, except those with the highest degree of deterioration.

Purpose and Application

Cross stitching is applicable for a number of situations where strengthening joints or cracks is required, including the following (ACPA 2001b):

- Strengthening longitudinal cracks in slabs to prevent slab migration and to maintain aggregate interlock.
- Mitigating the issue of tiebars being omitted from longitudinal contraction joints (due to construction error).
- Tying roadway lanes or shoulders that are separating and causing a maintenance problem. (Cross stitching should not be used, however, to tie any new traffic lanes that are added to an existing roadway.)
- Tying centerline longitudinal joints that are starting to fault.

Cross stitching is not recommended for use on transverse cracks, especially those that are working, because cross stitching does not allow movement. If used on working transverse cracks, a new crack will likely develop near the stitched crack, or the concrete will spall over the reinforcing bars (ACPA 1995). Also, experience demonstrates that 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 (ACPA 2006). For these cases, a sand cement grout is a suitable backfill for this purpose (ACPA 2006).

Construction Considerations

Cross stitching generally uses a 19-mm (0.75-in.) diameter deformed tiebar to hold the joint or crack tightly together and enhance aggregate interlock (ACPA 2001b). The bars are typically spaced at intervals of 500–750 mm (20–30 in.) along the joint or crack, and alternated on each side of the joint or crack; see Figure 8.18. The spacing of the bars varies by truck traffic levels, with a 500-mm (20-in.) spacing recommended for heavy truck traffic and a 750-mm (30-in.) spacing recommended for light traffic and interior highway lanes. A properly drilled hole is one that intersects the joint or crack at mid-depth (ACPA 1995). Overall recommendations on cross stitching bar dimensions and angles/locations of holes are presented in Table 8.5.

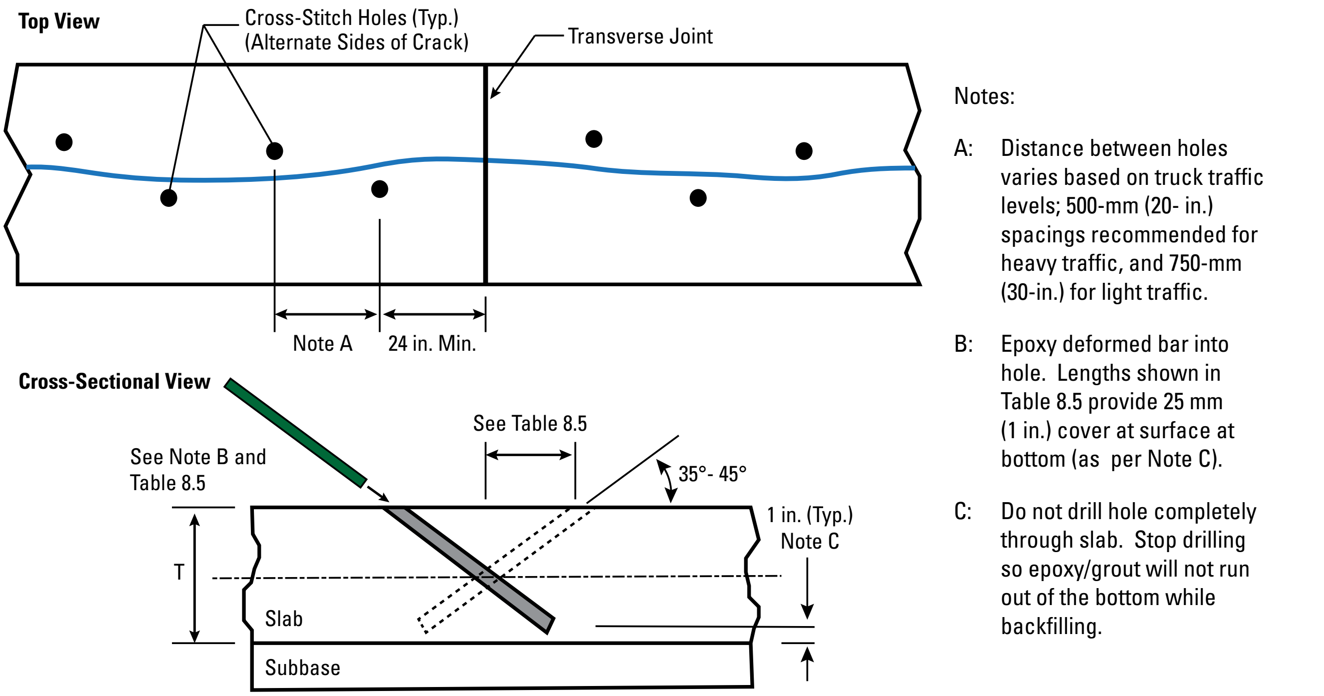


Figure 8.18. Schematic of cross stitching (adapted from IGGA [2010])

Table 8.5. Cross Stitching Bar Dimensions and Angles/Locations or Holes (IGGA 2010)

Angle	Slab Thickness, mm (in.)							
	200 (8)	225 (9)	250 (10)	275 (11)	300 (12)	325 (13)	350 (14)	380 (15)
	Distance from Crack to Hole, mm (in.)							
35°	145 (5.75)	165 (6.50)	180 (7.25)	195 (7.75)	210 (8.50)	—	—	—
40°	—	—	—	165 (6.50)	180 (7.25)	195 (7.75)	205 (8.25)	—
45°	—	—	—	—	150 (6.00)	165 (6.50)	175 (7.00)	190 (7.50)
	Length of Bar, mm (in.)							
35°	240 (9.50)	275 (11.00)	315 (12.50)	365 (14.50)	400 (16.00)	—	—	—
40°	—	—	—	315 (12.50)	350 (14.00)	400 (16.00)	465 (18.50)	—
45°	—	—	—	—	300 (12.00)	350 (14.00)	415 (16.50)	450 (18.00)
	Diameter of Bar, mm (in.)							
	19 (0.75)	19 (0.75)	19 (0.75)	19 (0.75)	19 (0.75)	25 (1.0)	25 (1.0)	25 (1.0)

The cross-stitching process requires the following steps and considerations (IGGA 2010):

- Drill holes at an angle to the pavement so that they intersect the joint or crack at mid-depth; see Figure 8.19. It is important to start drilling the hole at a consistent distance from the joint or crack to consistently cross the joint or crack at mid-depth. Select a drill that minimizes damage to the concrete surface (e.g., hydraulic powered drill), and select a drill diameter no more than 9.5 mm (0.375 in.) larger than the tiebar diameter.
- Blow air into the holes to remove dust and debris after drilling.
- Pour epoxy into the hole, leaving some volume for the bar to occupy the hole.
- Insert the tiebar, remove excess epoxy, and finish flush with the pavement surface. The pavement may be reopened to traffic as soon as the epoxy has fully set. A completed project is shown in Figure 8.20.

Figure 8.19. Drilling holes for cross stitching

Figure 8.20. Completed cross stitching

10. Slot Stitching

Introduction

Slot stitching is a preservation repair technique for longitudinal cracks and joints that grew out of the DBR technique. The process and technique for slot stitching is similar to DBR except for the use of deformed tiebars and its application at longitudinal joints and cracks.

Purpose and Application

The purpose of slot stitching is to hold together adjoining concrete slabs or segments through the use of deformed tiebars, typically 25 mm (1 in.) in diameter or larger, placed in slots cut into the existing concrete pavement (IGGA 2010). The tiebars contribute to maintaining aggregate interlock and adding reinforcement and strength to the crack or joint (IGGA 2010).

Construction Considerations

The slot-stitching process is similar to 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 DBR, precision alignment is not critical since 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 backfill materials from flowing into the area between the slabs.
- Place deformed bars into the slot.
- Place 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 8.21 shows a schematic of the slot-stitching preservation treatment. Note that the slots can be cut approximately perpendicular to the longitudinal joint or crack, and if the joint or crack width is significantly wide, consideration should be given to reforming and caulking prior to the placement of the patching material. Figure 8.22 shows slot stitching that has been performed on a longitudinal crack.

Slot Stitching vs. 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. Slot stitching is generally applied to more severe cracks and cracks with wider widths. As described previously, the cause of the longitudinal cracking should be carefully evaluated in order to be fixed correctly; working cracks that are stitched by either method could lead to the development of additional cracking in other locations.

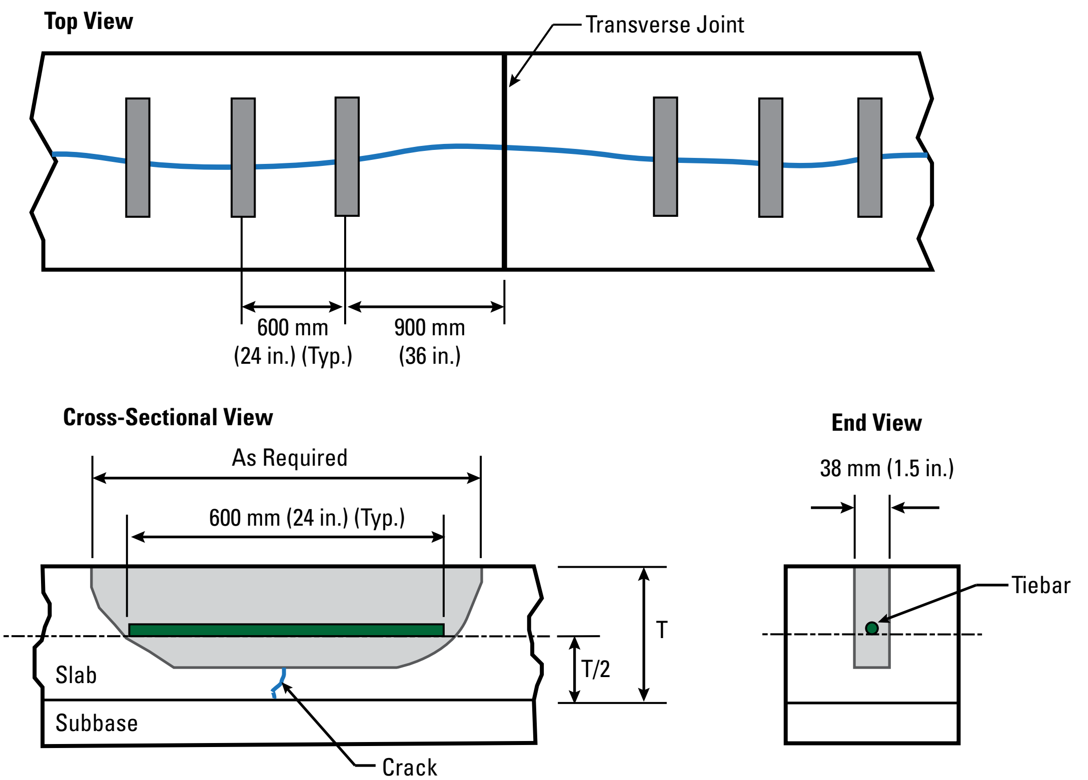


Figure 8.21. Schematic of slot stitching (adapted from IGGA [2010])

Figure 8.22. Completed slot stitching on longitudinal crack (courtesy of John Donahue)

11. Summary

This chapter provides guidance for properly designing and installing retrofitted dowel bars in concrete pavements. Dowel bar retrofit 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 (little or no distress) but are exhibiting poor 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 the chapter primarily focuses on the details of DBR technique, it also contains 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. Learning Outcomes

This chapter describes recommended procedures for surface restoration of concrete pavements. Upon completion of this chapter, the participants will be able to accomplish the following:

- Differentiate between conventional diamond grinding and diamond grooving, and list the purpose and benefits of each.
- List characteristics of other surface texturing techniques, namely optimized texture for city streets (OTCS), next generation concrete surface (NGCS), and cold milling.
- Identify appropriate blade spacing dimensions for conventional diamond grinding, diamond grooving, and NGCS.
- Describe recommended construction procedures for conventional diamond grinding, diamond grooving, and NGCS.
- Identify typical construction problems and remedies.

2. Introduction

Conventional diamond grinding 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., dowel bar retrofit, partial-depth repairs, full-depth repairs) 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 and application of both diamond grinding and diamond grooving and discusses important design considerations and construction procedures for successful projects using each treatment. Although those two treatments are the focus of this chapter, three other surface-texturing processes are also introduced:

- OTCS, a texture similar to conventional diamond grinding but with reduced land area heights and widths that make it more favorable to pedestrians and bicyclists.
- NGCS, a manufactured, low-noise texture developed for both new and existing concrete pavements. Because of its potential for use in rehabilitation of excessively noisy pavements, additional guidance is provided on this texture.
- 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 not as a final riding surface.

3. Purpose and Project Selection

Conventional Diamond Grinding

Diamond grinding is the removal of a thin layer of hardened concrete pavement surface 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 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.

Diamond grinding was first used in California in 1965 on a then-19-year-old section of Interstate 10 to eliminate significant faulting (Neal and Woodstrom 1976). In 1983, concrete pavement restoration (CPR) was conducted on that same pavement section, including the use of additional grinding to restore the rideability and skid resistance of the surface. In 1997, this pavement was reground for a third time, where it was carrying significant truck traffic (nearly 2.25 million ESAL (equivalent single axle load) applications per year in the truck lane).

Figure 9.1 shows a picture of the grinding head that is mounted on the grinding machine, 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.

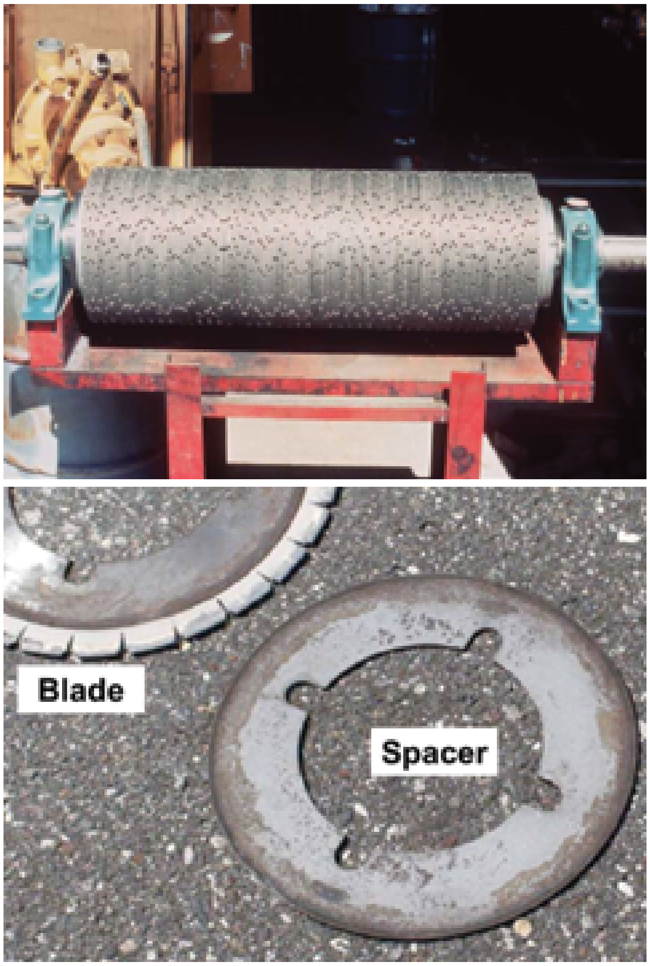

Figure 9.1. Grinding head (top) and saw blades and spacers (bottom) (photos courtesy of John Roberts, IGGA)

Figure 9.2. Surface texture provided by diamond grinding (courtesy of John Roberts, IGGA)

A schematic of the surface texture produced by the diamond grinding operation is shown in Figure 9.3 and is noted to consist of a groove width, a land area, and a 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 range of typical dimensions for diamond grinding operations.

Even though the land area is conceptually easy to visualize, its use in specifications is problematic because its dimensions depend on other factors. 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

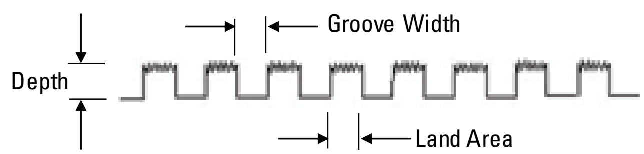

Figure 9.3. Schematic of grinding surface texture

Table 9.1. Range of Typical Dimensions for Diamond Grinding Operations

	Range	Hard Aggregate[1]	Soft Aggregate[2]
Groove width	2.29–3.81 mm (0.090–0.150 in.)	2.29–3.81 mm (0.090–0.150 in.)	2.29–3.81 mm (0.090–0.150 in.)
Land area	1.78–3.25 mm (0.070–0.130 in.)	1.78–2.79 mm (0.070–0.110 in.)	2.29–3.25 mm (0.090–0.130 in.)
Depth	1.00–3.00 mm (0.040–0.12 in.)	1.00–3.00 mm (0.040–0.12 in.)	1.00–3.00 mm (0.040–0.12in.)
No. of Blades	165–200/m (50–60/ft)	175–200/m (53–60/ft)	165–180/m (50–54/ft)

[1] Such as granite, quartz, or some river gravels
[2] Such as limestone or dolomite

blades all affect the land area width. This is illustrated in Figure 9.4 and shows how the saw blade segments extend out beyond the width of the saw blade core, and it also shows, for this example, that a 2.67-mm (0.105-in.) saw blade core and a 2.79-mm (0.110-in.) spacer produce a land area of 2.29 mm (0.090 in.). Moreover, blade irregularities and wear, machine setup and operator variability, and difficulties in obtaining in-field measurements further complicate the use of land area as a specification item.

Because of these issues, it is recommended that the number of blades per unit length be adopted for use in the grinding specification (see bottom row in Table 9.1). In general, more blades will be used per unit length for projects with harder aggregates (in order to produce a thinner land area), and fewer blades will be used per unit length for projects with softer aggregates (in order to produce a wider land area). The contractor will work with the blade manufacturer to select the appropriate number of blades needed for a given project.

Since its first use 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 wheelpath "rutting" caused by studded tire wear.

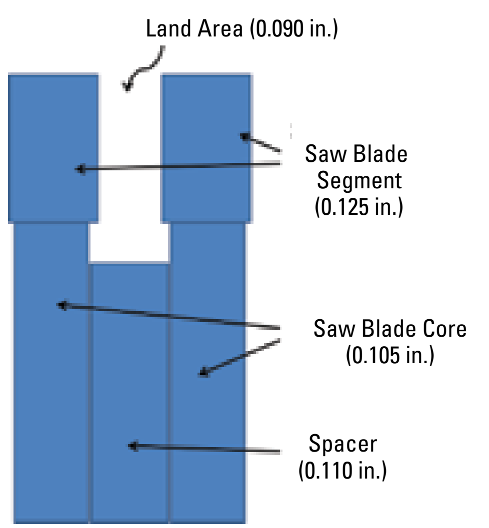

Figure 9.4. Schematic configuration of typical saw blade and spacer pairings

- Removal of "built-in" slab curling or slab 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.

Although most highway agencies have their own criteria for considering diamond grinding as a preservation treatment, some general guidelines for its selection on a specific concrete pavement project include the following:

- Average transverse joint faulting in excess of 2 mm (0.08 in.).
- Roughness in excess of 2.5–3.5 m/km (160–220 in./mi).
- Wheelpath wear greater than 6–10 mm (0.25–0.40 in.).
- Surface friction values below agency standards for the roadway facility and location.
- As required 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, and current distress types, severities, and extents must be taken into account. Some of these additional considerations are described below to help agencies determine the feasibility of diamond grinding for a particular 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. Agencies should consider the structural condition of the pavement and the economics of grinding compared to other preservation/rehabilitation alternatives (e.g., concrete or asphalt overlays) to determine which approach is most cost effective for the given project.
- Faulting of transverse joints suggests load transfer and slab support issues, so agencies should consider the installation of retrofitted dowel bars, slab stabilization, and possibly retrofitted edgedrains prior to the grinding operation in order to address the root cause

of the faulting. If the underlying cause of the faulting is not addressed, it can quickly redevelop under additional traffic loadings (Pierce 1994).

- Structural distresses such as corner breaks, working transverse cracks, and shattered slabs will require repairs before grinding (Correa and Wong 2001). The presence of significant slab cracking in a project (often taken as more than 10 percent of the slabs) suggests a structural problem, and diamond grinding may not be appropriate. 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, and its direct impact on the cost of grinding, can influence whether or not a project is a feasible grinding candidate. Grinding a pavement with extremely hard aggregate (such as granite, 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 cut texture longer and can provide for an extended service life.
- Concrete pavements suffering from durability problems, such as D-cracking or ASR, indicate that diamond grinding is not a suitable preservation technique and that a more substantial rehabilitation strategy may be required (Correa and Wong 2001).
- Jointed reinforced concrete pavements may have wire mesh located near the surface of the pavement, which could create localized surface raveling issues if 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, depth of removal, size of 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, joint deterioration), and those roughness contributions may not be fully addressed through diamond grinding. Thus, each agency is encouraged to develop guidelines based on their local experience, project conditions, aggregate types, and contractor availability to determine the appropriateness of performing diamond grinding.

Diamond Grooving

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 19 mm (0.75 in.). The principal objective of grooving is to provide escape channels for surface water, thereby reducing the incidence of hydroplaning that can otherwise contribute to wet-weather crashes. Diamond grooving should only be used on pavements that are structurally and functionally adequate. Figure 9.5 shows a schematic of the recommended groove dimensions, whereas Figure 9.6 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 advantages of transverse grooving are that it provides the most direct channel for the drainage of water from the pavement and it introduces a surface that provides considerable braking traction. Although common on bridge decks, transverse grooving is not often used on highway pavements due in part to construction difficulties encountered in maintaining traffic on the adjacent lane and in part to higher noise levels that can be generated.

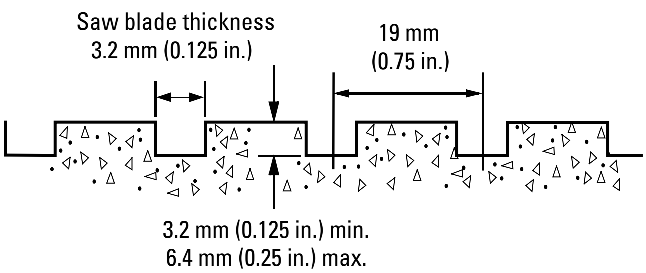

Figure 9.5. Typical dimensions for pavement grooving

Figure 9.6. Longitudinally grooved surface

Longitudinal grooving is more commonly used on highways, and it is often done in localized areas where wet-weather crashes have been a problem, such as curves, exit ramps, bridges, and intersection approaches. Although longitudinal grooving does not improve the drainage characteristics of the pavement surface as well as transverse grooving, it does provide a channel for the water and produces a tracking effect for vehicles, particularly on horizontal curves.

OTCS

The OTCS is a surface texture specifically designed for city streets and urban thoroughfares; by employing a texture with a reduced land height and width, the resulting surface is favorable to bicycles, roller blades, and other urban recreational traffic. The texture is produced using diamond grinding technology with a groove depth between 0.51 and 1.3 mm (0.02 and 0.05 in.) and thin blade spacers on the order of 0.76 mm (0.03 in.) wide. The cost is approximately 25 percent higher than conventional diamond grinding.

NGCS

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 pavement-tire interaction phenomenon and the effects of diamond grinding on noise levels (Scofield 2012). Research determined that the noise levels for diamond ground textures were more a function of fin (land) profile than blade or spacer widths (Dare et al. 2009). From that research, the NGCS was developed as a manufactured surface that controls the resulting fin profile and minimizes positive texture (Scofield 2010). A comparison of conventional diamond grinding and NGCS is shown in Figure 9.7.

The NGCS uses conventional diamond grinding equipment and blades, but in such a way that no "fins" or positive texture are produced (Scofield 2012). It consists of two activities, flush-grinding followed by grooving, and these activities can be performed in either a single-pass operation (flush grinding and grooving done at the same time) or a two-pass opera-

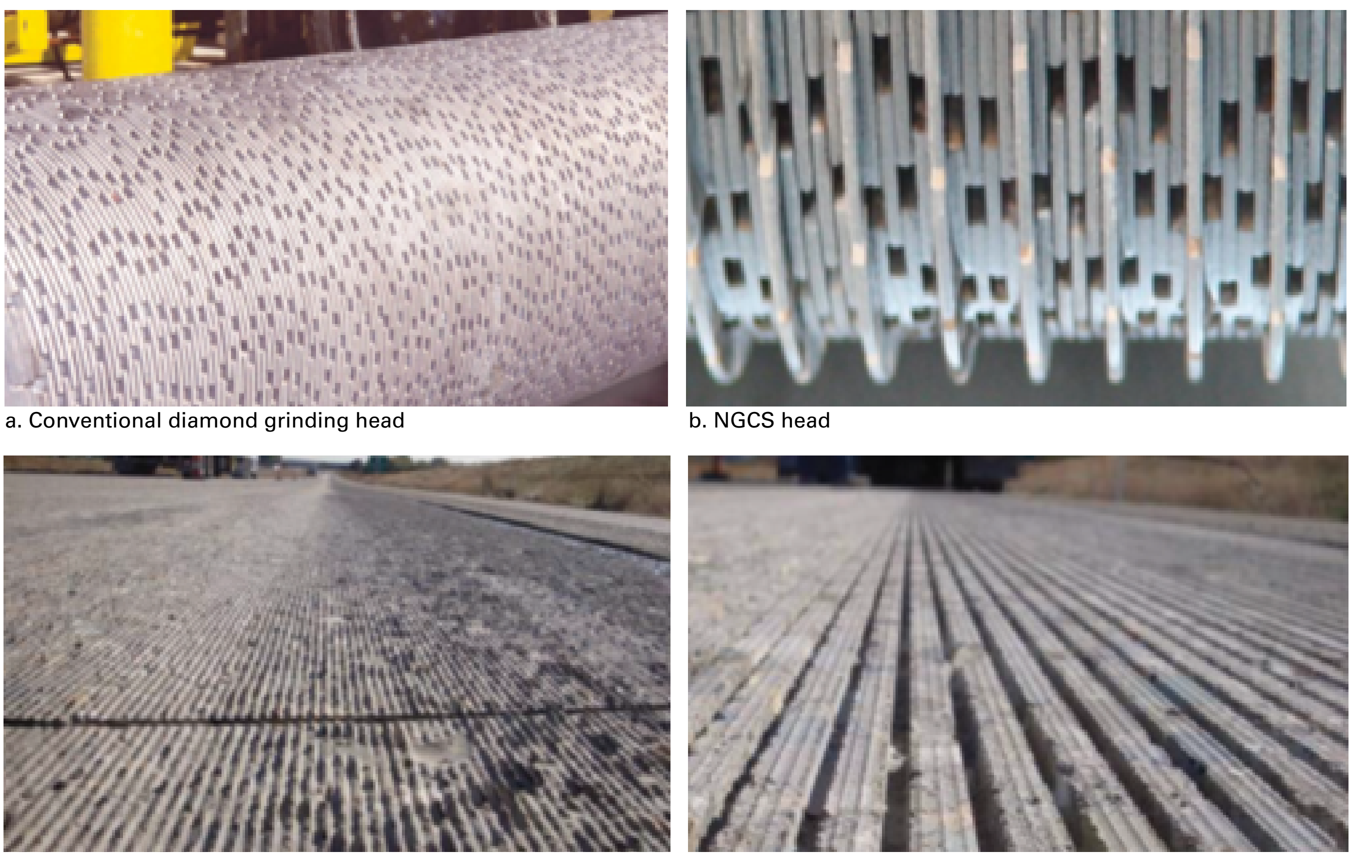
a. Conventional diamond grinding head
b. NGCS head
c. Conventional diamond ground surface
d. NGCS surface

Figure 9.7. Conventional diamond ground surface and NGCS (Anderson et al. 2011)

tion (flush grinding performed first, followed by a separate grooving operation). The NGCS can be used on both new pavement construction and the rehabilitation of existing concrete pavements (Scofield 2012).

The NGCS has been determined to be the quietest nonporous concrete texture developed to date. 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 between 99 dBA and 101 dBA (Scofield 2012). Table 9.2 summarizes the difference in OBSI results in a controlled experiment between NGCS and conventional diamond grinding; each project had NGCS and conventional diamond grinding performed at the same time and allowed for meaningful direct comparisons of results for the given traffic and climate. For all projects, the section with the NGCS was quieter than the counterpart section with conventional diamond ground surface at the time of construction, and the NGCS has maintained a quieter level for all but one project after 1–4 years of service.

Table 9.2. Summary of OBSI Differences between Conventional Diamond Grinding and NGCS (adapted from Scofield [2012])

Project Location	OBSI Difference at Time of Construction (dBA)[1]	OBSI Difference in 2011 (dBA)[1]	Surface Age in 2011(year)
Arizona	-2.9	-1.6	1
Illinois	-0.2	+0.2	4
Iowa	-1.3	-0.5	1
Kansas	-2.3	-1.7	3
Minnesota	-4.2	-2.1	4
Average Difference	-2.1	-1.1	

[1] Negative value indicates NGCS quieter than conventional diamond grinding.

Cold Milling

Cold milling is an operation significantly 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. The primary difference between diamond grinding and cold milling is the way the concrete layer is removed. With diamond grinding, the diamond blades cut into the top of the concrete surface, while the cold milling head chips away leaving a rough surface and fractured joint faces (ACPA 2001).

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 those bits on the drum to produce a smoother surface than that produced by conventional cold milling. The resulting surface is more favorable to carrying traffic and is particularly useful in preparing an existing asphalt pavement for receiving a thin bituminous surface layer.

Cold milling is not recommended for restoring a concrete pavement smoothness because it leaves a rough pavement surface and damages the transverse and longitudinal joints (ACPA 2001). The damage inflicted by cold milling at joints is shown in Figure 9.8. Cold milling has been shown to be an effective and productive method of preparing small surface areas for partial-depth repairs, as described in Chapter 5. Moreover, cold milling is commonly used to prepare an existing bituminous pavement prior to the placement of a concrete overlay, or it may be used in combination with shot blasting to prepare a concrete surface for a bonded concrete overlay.

Figure 9.8. Joint damage (foreground) caused by cold milling

4. Limitations and Effectiveness

Diamond Grinding

A number of studies on the effectiveness of diamond grinding have indicated excellent long-term performance when grinding is conducted in conjunction with other required CPR activities (Rao, Yu, and Darter 1999; Correa and Wong 2001; Stubstad et al. 2005). One possible explanation for this positive impact on pavement life is the long-standing theory that eliminating faulting and restoring smoothness reduces the dynamic effects of loadings on the pavement.

Performance Life

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 second grinding or rehabilitation was needed) was 14 years, whereas the expected longevity at an 80 percent reliability level was 11 years (i.e., 80 percent of the sections lasted at least 11 years) (Rao, Yu, and Darter 1999; Rao et al. 2000). 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 percent level of reliability) and a longevity of 14.5 years at an 80 percent level of reliability (Stubstad et al. 2005); these are shown in Figure 9.9. The rehab trigger for these values is taken as a 78 percent increase in the roughness of the newly ground surface (ratio = 1.78).

In addition to addressing pavement roughness, diamond grinding also produces a pavement surface with ample macrotexture that contributes to surface friction. An Arizona study showed that the increase in friction values associated with different grinding configurations ranged between 15 and 41 percent, with an overall average improvement of 27 percent (Scofield 2003). In Wisconsin, Drakopoulos et al. (1998) found that the overall crash rate for diamond ground surfaces was only 60 percent of the crash rate for the unground surfaces. As discussed in Section 5, however, the hardness of the aggregate will affect the longevity of the surface texture, because softer aggregates will tend to polish and can lose their surface texture more quickly.

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 the 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 highway agencies are also allowing con-

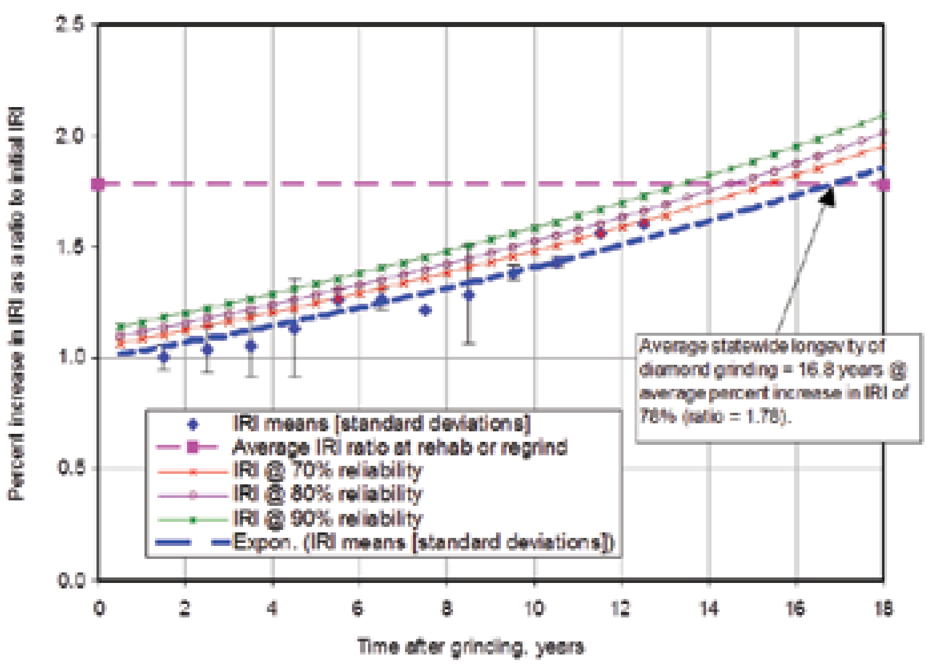

Figure 9.9. Survivability of diamond ground pavements in California (Stubstad et al. 2005)

tractors to use diamond grinding as the final surface texture, since it can produce a consistent, smooth, 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).

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). The observation from one study indicates that following diamond grinding, faulting redevelops at a fast rate initially but stabilizes to the rate comparable to that just prior to grinding (Rao, Yu, and Darter 1999). This is illustrated in Figure 9.10, which shows time-series faulting data from several different diamond grinding projects. 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.

Figure 9.11 illustrates an example of the effects of concurrent work on the projected faulting performance of diamond ground pavements (Snyder et al. 1989). These results again emphasize that diamond grinding by itself does not address the underlying mechanism of joint faulting, and therefore it should be combined with other appropriate preservation techniques in order to prevent or delay the recurrence of faulting.

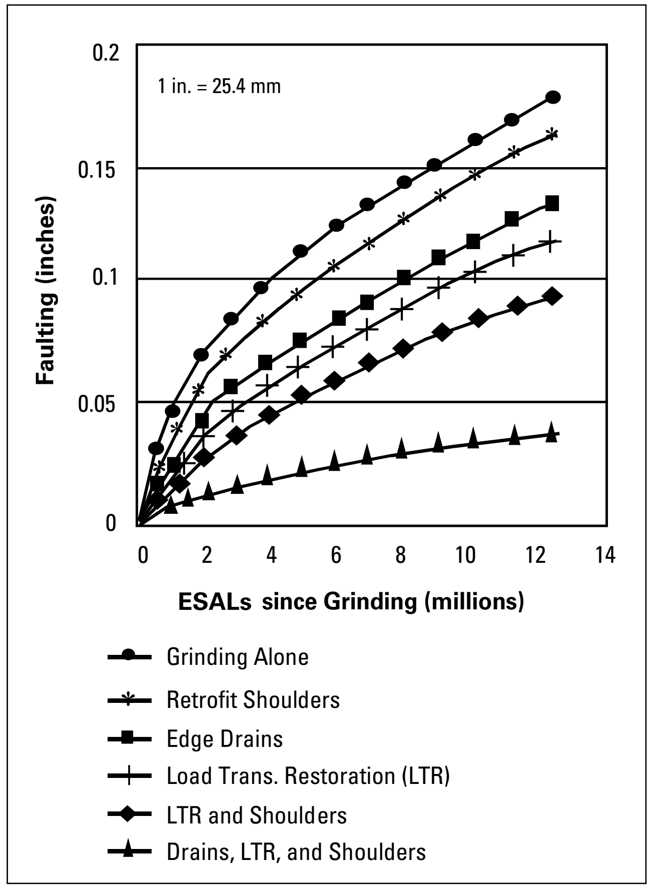

Figure 9.11. Effect of concurrent CPR techniques on faulting (Snyder et al. 1989)

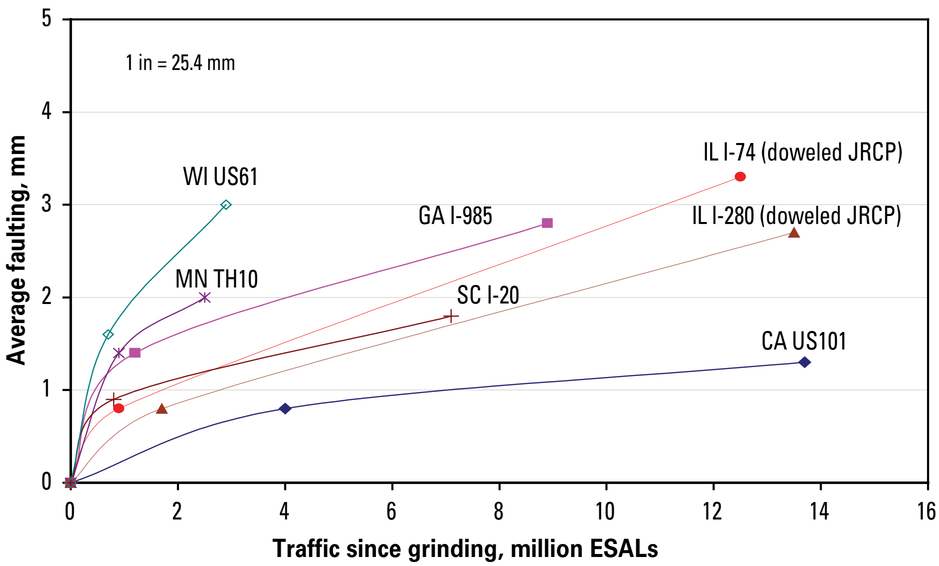

Figure 9.10. Time history faulting data (since diamond grinding) for diamond ground projects (Rao, Yu, and Darter 1999)

Diamond Grooving

As previously described, diamond grooving increases the macrotexture of the pavement and provides channels for the water to escape, thereby decreasing the potential of hydroplaning. Figure 9.12 shows the number of wet-weather crashes over time on an early diamond grooving project in California; the number of crashes increased with time until year 7.5, at which point diamond grooving was performed and the number of crashes was reduced by nearly 80 percent (Ames 1981).

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 vehicle. This subject was studied at length by the California Division of Highways in the 1960s and 1970s (Zube, Skog, and Kemp 1968; Sherman, Skog, and Johnson 1969; Karr 1972). Although some small lateral movement was noted by these vehicles on longitudinally grooved pavements, using 3-mm (0.125-in.) wide grooves and groove spacings of 19 mm (0.75 in.) minimized those effects.

In 2007, several of California's original longitudinally grooved pavements were re-evaluated for noise. It was determined that longitudinal grooving is not an effective treatment for noise mitigation, but it is effective in providing lateral stability and improved friction (ACPA 2007).

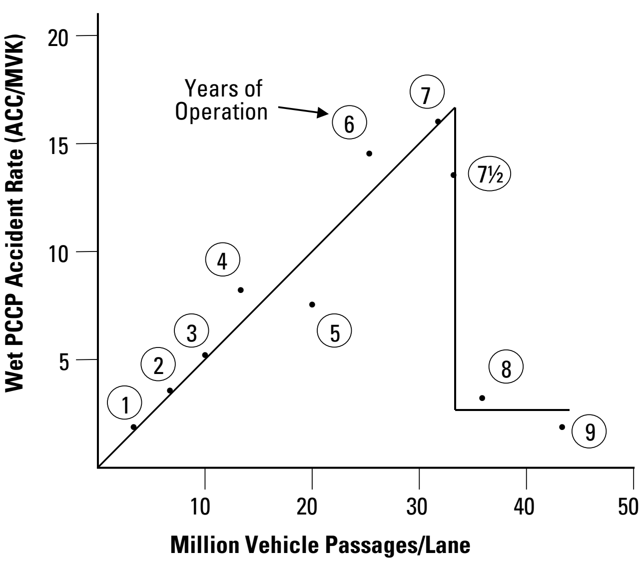

Figure 9.12. Wet-weather crashes (ACC/MVK, or crashes per million vehicle kilometers) for a selected California pavement before and after longitudinal grooving (Ames 1981)

5. Design Considerations

Prior to performing diamond grinding or grooving, pavement information should be obtained and evaluated to determine the feasibility of these rehabilitation techniques on their own or concurrent with other rehabilitation techniques.

Conventional Diamond Grinding

When considering a diamond grinding operation, information on the degree of 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 areas for diamond grinding and which concurrent restoration activities are required.

The surface characteristics of the pavement after grinding are highly dependent on the blade spacing, which in turn is selected based upon the hardness of the aggregate. The frictional resistance of easily polished aggregate (or softer aggregate such as limestone) can be improved by increasing the blade spacing to expand the "land area" between the sawed grooves. Although the friction characteristics for softer aggregates may be improved by increasing the spacing between blades, light vehicles and motorcycles may experience vehicle tracking. Some agencies specify tighter blade spacings (such as 165–180 blades/m [50–54 blades/ft]) to specifically address light vehicle tracking issues (Rao, Yu, and Darter 1999).

Because diamond grinding is removing a portion of the slab thickness, there is a concern about potential reductions in the load-carrying capacity of the pavement, which could potentially result in increased cracking. Studies have indicated, however, that this slight reduction in slab thickness does not significantly compromise the fatigue life of the slab, largely because the long-term strength gain of the concrete offsets any slight reductions in slab thickness (Rao, Yu, and Darter 1999). In fact, it is suggested that a typical concrete pavement may be ground up to three times (13–18 mm [0.5–0.7 in.]) without compromising the fatigue life of the pavement (Rao, Yu, and Darter 1999).

NGCS

As previously mentioned, the NGCS can be constructed on existing concrete pavements as well as on new concrete pavements. Although the reasons for considering the NGCS for an existing pavement may be similar to those for performing diamond grinding (namely, fault removal, surface friction, and noise reductions), the most compelling reason for constructing the NGCS is its impact on reducing noise emissions; therefore, an agency would be most likely to consider it on an existing pavement that exhibited critical noise issues. Additional benefits provided by the NGCS, however, are the reduction in hydroplaning potential and the increased lateral stability.

Whether done in a single-pass or two-pass operation, the blades and spacings for the flush grinding and longitudinal grooving operations are standardized as follows (IGGA 2011):

- **Flush Grinding**—The flush grinding operation will use 3.18-mm (0.125-in.) wide blades separated by 0.89 mm ± 0.13 mm (0.035 in. ± 0.005 in.) spacers. The blades should be flat across their contact surface and should lie in the same plane with other flush grind blades when mounted.
- **Longitudinal Grooving**—The longitudinal grooves will be 3.18 mm (0.125 in.) wide, 3.18–4.76 mm (0.125–0.1875 in.) deep, and spaced on 12.7- to 15.9-mm (0.5- to 0.625-in.) centers. The grooves are constructed parallel to the centerline.

Diamond Grooving

Grooving operations are intended to reduce hydroplaning and accompanying crashes. Information regarding an area with a high number of crashes, as well as surface friction data for the section, should be reviewed prior to considering grooving operations.

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, intersections). The entire lane width should be grooved; however, allowance should be made for small areas that were not grooved because of pavement surface irregularities.

6. Construction Considerations

Diamond Grinding

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 highway grinding operations has a "wheel base" of between 3.0 and 4.3 m (10 and 14 ft, as measured from the leading bogie wheels to the depth control wheels), and a minimum weight of 15,876 kg (35,000 lb), which includes the grinding head (IGGA 2010). The grinding machines are also equipped with a vacuuming system for removing grinding residue from the pavement surface. Figure 9.13 shows a diamond grinding machine in operation.

Figure 9.13. Diamond grinding machine (top) and cutting head (bottom) (courtesy of John Donahue, Missouri DOT)

The cutting head affixed on the equipment typically has a width ranging from 1.22 to 1.27 m (48 to 50 in.), as shown in Figure 9.14. As provided earlier in Figure 9.1, the diamond blades are typically spaced in the range of 164 to 197 blades per meter (50 to 60 blades per ft), depending on the hardness of the aggregate.

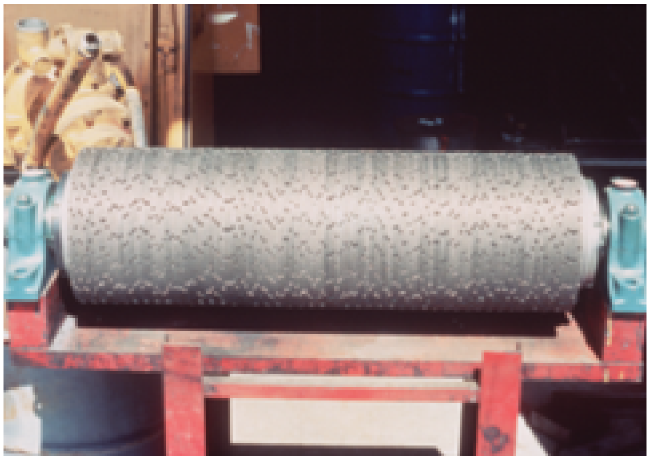

Figure 9.14. Diamond grinding cutting head (courtesy of John Roberts, IGGA)

Procedures

Figure 9.15 shows a schematic 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 percent of the area within any 1 m-by-30 m (3 ft-by-100 ft) test area be textured by the grinding operation. Isolated low spots of less than 0.2 m^2 (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 perpendicular to the pavement centerline and should also be consistently maintained parallel to the centerline. Grinding has typically been conducted on multilane facilities using a mobile single 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.

Because of the relatively narrow width of the cutting head, more than a single pass of the grinding equipment is required. It is recommended that a 25-mm (1-in.) overlap be maintained between adjacent passes of the diamond grinding equipment. Multiple grinding machines working together can be used to help expedite the grinding operation.

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 lasers on a textured surface, the profiler should be equipped with a wide-footprint 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 is rerun and evaluated to determine whether or not the ground surface meets the smoothness requirements. Furthermore, a minimum improvement in the pavement smoothness (before and after grinding) is sometimes required, using the following equation:

$$\%\ \text{Improvement} = [(S_b - S_a)/S_b] \times 100 \qquad (9.1)$$

where:

S_b = Smoothness before grinding (typically expressed in terms of IRI, but could be any smoothness statistic)

S_a = Smoothness after grinding

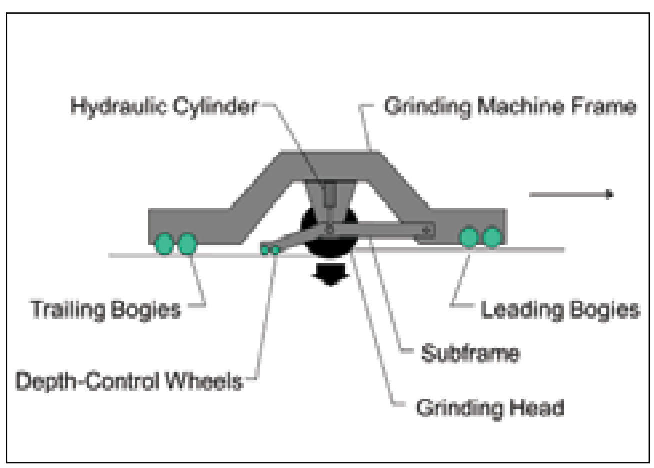

Figure 9.15. Schematic of diamond grinding machine

For example, if an existing pavement has an IRI before grinding of 2.4 m/km (150 in./mi), and the IRI after grinding is 1.6 m/km (100 in./mi), then the improvement in pavement smoothness is 33 percent. Typical improvement requirements may be in the 30–35 percent range, but these will vary by highway agency and by type of facility (e.g., rural interstate, urban arterial).

It is important to note that diamond grinding is most effective at removing short wavelength roughness, such as that caused by faulted joints; it generally cannot remove roughness caused by long wavelengths. If long wavelengths are contributing to high roughness levels, it may be difficult for diamond grinding to meet the prescribed roughness requirements. In that case, FHWA's PROVAL software can be used to help analyze the before-grinding profile to see what level of improvements can be realistically achieved.

"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 the diamond grinding operation, most specifications provide for a small amount of holidays within a project (for example, 95 percent coverage is commonly specified). Figure 9.16 shows several holidays on the ground surface of an inside turning lane.

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 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 9.17 shows a gutter apron before and after a feathering pass.

Figure 9.16. Holidays on diamond ground surface of inside turning lane (courtesy of Dan Frentress, IGGA)

Figure 9.17. Gutter apron before (top) and after (bottom) feathering pass (courtesy of Dan Frentress, IGGA)

NGCS

Equipment

Depending on the type of NGCS operation, either one or two pieces of equipment will be needed for the construction of the NGCS. If a single-pass operation is employed, a single, self-propelled machine designed specifically for diamond grinding and pavement texturing shall be used, with the blade and spacer configurations presented in Section 5; if a two-pass operation is used, then 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 in Section 5.

Procedures

The use of either the single-pass or two-pass operation in the NGCS construction will be determined by the contractor. 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 lines normal to the pavement centerline at the project limits. Passes of the grinding head shall not overlap more than 25 mm (1 in.), and no unground surface area is permitted between passes.

Figure 9.18. Diamond grooving head (top) and grooving operation (bottom)

Diamond Grooving

Equipment

Equipment used to groove pavements is specifically designed for this task. Because fewer diamond blades are required on the cutting head, the head width can be substantially greater than that used in diamond grinding. Some pieces of equipment are available that have a grinding head width of 1.8 m (6 ft) or more.

The diamond blades are typically spaced 19 mm (0.75 in.) apart for longitudinal grooving; see Figure 9.18. The grooves have a width between 2.5 and 3 mm (0.1 and 0.125 in.) and are cut to a depth of 3–6 mm (0.125–0.25 in.).

Procedures

As previously indicated, grooving is most commonly performed longitudinally along the pavement. Typically, only localized areas (such as curves or intersection approaches) are grooved, instead of the entire project length. Surface friction and wet-weather crash data, however, can be used to determine the extent of the grooving that may be needed.

Procedures typically follow those described previously for diamond grinding. 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.

Slurry Handling

All of the various grinding and grooving operations described herein produce a slurry consisting of ground concrete and the water used to cool the blades, which has been shown to be an inert, nonhazardous by-product (IGGA 1990; Holmes and Narver 1997; FHWA 2001; IGGA 2010). This slurry is picked up by on-board wet-vacuums and must be disposed of in accordance with local environmental regulations. The following provides recommendations for the handling and disposal of grinding slurry (IGGA 2010; MnDOT 2012):

- **Slurry Spreading Disposal**—For rural areas with vegetated slopes, the slurry can be deposited on the roadway inslope or backslope as the diamond grinding process progresses along the roadway, as shown in Figure 9.19. Wetlands and other environmentally sensitive areas where slurry discharge is not permitted should be clearly identified in the contract documents and along the shoulder of the roadway. Slurry generated in wetland and environmentally sensitive areas should be picked up and hauled for disposal to nonsensitive areas of the project. The diamond grinding equipment should include a vacuuming system to remove the standing slurry and deposit it along the roadway slopes by dragging a flexible hose or other approved device along the slope. Additional requirements include the following:
    - Slurry material should not be deposited onto the shoulder.
    - Slurry material should be spread at least 0.3 m (1 ft) from the shoulder edge and distributed down the roadway slope with each pass of the diamond grinding equipment.
    - Slurry material should not be spread within 30 m (100 ft) of a natural stream or lake or within 1 m (3 ft) of a water-filled ditch, and it should not be allowed to enter into a closed drainage system (such as city storm sewers).
- **Slurry Collection and Pond Decanting**—In urban and other areas where the slurry cannot be deposited along the slope, it should be collected and transported in water-tight tanker units to settlement ponds constructed by the contractor; see Figure 9.20. Ponds should be constructed to allow for settlement of the solids and decanting of the water. At the end of the grinding operation, the excess water should be allowed to evaporate or be used in commercially

Figure 9.19. Depositing slurry on vegetated slopes in rural areas (IGGA 2010)

Figure 9.20. Vacuum system (top) and slurry deposit in settlement pond (bottom) (IGGA 2010)

useful applications. The remaining dried solids can be used as a fill material, a component in recycled aggregate, or other commercially useful applications.

- **Slurry Collection and Plant Processing**—Slurry material is collected and hauled to a processing plant (e.g., centrifuge, belt plant) according to the slurry collect and pond decanting method. Additional requirements include following state storm water runoff regulations, restoring the site to its original condition, and processing the water and solids according to the settlement ponds.
- **pH Control Plan**—During diamond grinding and slurry processing, the contractor should monitor and control the pH of the slurry. The slurry pH should be maintained within agency-specified values (e.g., between 6 and 12).

7. Quality Assurance

As with any pavement project, the performance of diamond grinding and grooving projects is greatly dependent on the quality of the construction procedures. Paying close attention to the procedures during construction greatly increases the chances of obtaining a surface with desired characteristics at the end of the project. The remainder of this section summarizes the recommended quality control activities for diamond grinding as presented in the FHWA's *Diamond Grinding of Portland Cement Concrete Pavements* (FHWA 2005). Although the list of activities in this checklist is specific to the diamond grinding process, many of the same activities can easily be applied to the diamond grooving process.

Preliminary Responsibilities

Agency and contractor personnel should collectively conduct a review of the project documentation, project scope, intended construction procedures, material usage, and associated specifications. Such a collective review is intended to minimize any misunderstandings in the field between agency designers, inspectors, and construction personnel. 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 or diamond grooving. 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.
- Assess the overall condition of the joints and cracks. Joints and transverse cracks exhibiting severe faulting (equal to or greater than 12 mm [0.5 in.]) or displaying evidence of pumping (e.g., surface staining or isolated wetness) are potential candidates for load transfer restoration with dowels prior to diamond grinding.
- Verify that structural repairs are completed in the proper sequence (i.e., FDRs, partial-depth repairs, DBR, diamond grinding, and joint resealing).

Document Review

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.
- Agency application requirements.
- Traffic control plan.
- Equipment specifications.
- Manufacturer's instructions.
- MSDS (if required for concrete slurry).

Equipment Inspections

Prior to beginning construction, all construction equipment must be examined. The following are equipment-related items that should be checked:

- Verify that the diamond grinding machine meets requirements of the contract documents for weight, horsepower, and blade configuration.
- Verify that the blade spacing on the diamond grinding cutting head meets the requirements of the contract documents.
- Verify that the vacuum assembly is in good working order and capable of removing concrete slurry from the pavement surface.
- Verify that the profilograph or pavement profiler meets requirements of the contract documents.

- Verify that the unit has been calibrated in accordance with manufacturer's recommendations and contract documents.
- Verify that the operator of the profilograph or pavement profiler meets the requirements of the contract documents for training/certification.

Project Inspection Responsibilities

During the construction process, an inspector should verify the following:

- Diamond grinding proceeds in a direction parallel with the pavement centerline, with beginning and ending lines normal to the pavement centerline.
- Diamond grinding results in a corduroy texture extending across the full lane width and complying with contract documents.
- Coverage requirements (such as 95 percent of the area must be ground) are common in most specifications; these are enforced by a visual inspection and the measurement of any holidays over a 30-m (100-ft) length of the project.
- Texturing cut into the existing pavement surface is in accordance with texturing requirements presented in the contract documents. Although typical values were presented in Table 9.1, specific dimensions and tolerances contained in the project documents take precedence.
- Each application of the diamond ground texture overlaps the previous application by no more than the amount designated in the contract documents, typically 25 mm (1 in.).
- Each application of the diamond ground texture does not exceed the depth of the previous application by more than the specified amount (typically 6 mm [0.25 in.]).
- The transverse slope of the ground surface is uniform to the extent that no misalignments or depressions that are capable of ponding water exist. Project documents typically have specific measurable criteria for transverse slope that must be met. Some "feathering passes" may be required to maintain a uniform cross slope with adjacent shoulders or traffic lanes.
- The diamond ground texture meets smoothness specifications (check on a daily basis).
- The concrete slurry is adequately vacuumed from the pavement surface and is not allowed to flow into adjacent traffic lanes.
- The grinding residue is not discharged into a waterway, a roadway slope within 30 m (100 ft) of a waterway, or any area prohibited by the contract documents or engineer. Concrete slurry should be disposed of by spreading along roadway slope in rural areas or collected and discharged at a disposal area in urban and environmentally sensitive areas. Slurry handling shall be in accordance with the contract documents.

Weather Requirements

The following weather-related items should be checked immediately prior to construction:

- Air and/or surface temperature should meet minimum agency requirements (typically 2°C [35°F] and above) for diamond grinding operations in accordance with contract documents.
- Diamond grinding shall 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 checked or verified:

- Verify that the signs and devices used match the traffic control plan presented in the contract documents.
- Verify that the setup complies with the Federal or local agency MUTCD or local agency procedures.
- 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 unsafe conditions, if any, are reported to a supervisor (contractor or agency).

8. Troubleshooting

Potential construction problems associated with diamond grinding and diamond grooving that may be encountered are presented in Tables 9.3 and 9.4, respectively. Typical causes and recommended solutions are also provided in these tables.

Table 9.3. Potential Diamond Grinding Construction/Performance Problems and Associated Solutions (ACPA 2000; ACPA 2006; FHWA 2005)

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 (typically 25 mm [1 in.]) between passes and steady steering by the 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 percent coverage for grinding texture and allow for 5 percent unground isolated areas.
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. A less than 3 mm per 3 m (0.12 in. per 10 ft) vertical overlap requirement is often required.
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 the opening. • 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 slabs deflect from the weight of the grinding equipment, lowering the grinding head may help, but stabilizing the slab or retrofitting dowel bars may be a better alternative.
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 cutting blades are not worn, the blade spacing should be reduced.
Large amounts of slurry on the pavement during 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 slurry flows into adjacent traffic lanes or drainage structures, the surface grinding operations should be stopped. Inspect the equipment and make necessary repairs.
Vehicle tracking experienced by light vehicles and motorcycles	This indicates a problem with the spacing between the blades.	Reduce the spacing between the blades.

Table 9.4. Potential Diamond Grooving Construction Problems and Associated Solutions (ACPA 2000)

Problem	Typical Cause(s)	Typical Solution(s)
Lack of horizontal overlap	As with grinding operations, 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 operator will avoid 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 (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	As with diamond grinding: • Expansion joints or other wide gaps in the pavement can cause the cutting head to dip if the leading wheels drop into the opening. • 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 slabs deflect from the weight of the grooving equipment, lowering the grooving head may help, but stabilizing the slab or retrofitting dowel bars may be a better alternative.
Large amounts of slurry on the pavement during grooving	As with grinding, 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 slurry flows into adjacent traffic lanes or drainage structures, the surface grooving operations should be stopped. Inspect the equipment and make necessary repairs.

9. 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 done 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 it 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 (increasing macrotexture) and reducing tire-pavement interaction noise.

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 wet-weather crashes associated with hydroplaning as well as splash and spray. Longitudinal grooving is commonly employed along 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.

Several other surface texturing processes were also introduced in this chapter. The 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. The OTCS was described as a texture with a reduced land height and width and one that is intended to be more favorable to bicycles, roller blades, and other urban recreational traffic. Finally, cold milling was acknowledged as a method of pavement removal for such applications as partial-depth repairs or in preparing a bituminous pavement for a concrete overlay, but it is not advocated as a corrective texturing method for faulted concrete pavements.

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Chapter 10

Joint Resealing and Crack Sealing

1. Learning Outcomes

This chapter describes recommended procedures for both joint resealing and crack sealing operations on concrete pavements. Upon successful completion of this chapter, the participants will be able to accomplish the following:

- List the benefits of joint resealing and crack sealing.
- Identify the types of projects that are suitable for joint resealing and crack sealing.
- Describe the types of sealant materials available for use.
- List the desirable sealant properties and characteristics.
- Describe the key considerations in designing how joints are resealed and cracks are sealed.
- Describe recommended installation procedures.
- Identify typical construction problems and appropriate remedies.

2. Introduction

Joint resealing and crack sealing are pavement preventive maintenance activities that serve two primary purposes. One purpose is to reduce the amount of moisture that can infiltrate a pavement structure, thereby reducing moisture-related distresses such as pumping, loss of support, joint faulting, corner breaks, and concrete deterioration at or beneath the joint. The other is to prevent the intrusion of incompressible materials (sand, pebbles, and other solid debris) so that pressure-related distresses such as spalling and blowups are prevented. Keeping joints and cracks sealed also has the beneficial effect of reducing noise emissions caused by "tire slap" or "joint slap" (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 routinely performed by many highway agencies. A recent survey of the preservation practices at 50 highway agencies in the United States and Canada revealed that 55 percent of agencies perform joint resealing on their medium-traffic rural roads and 78 percent perform it on their high-traffic urban roads (Peshkin et al. 2011). Similarly, 56 percent of agencies perform crack sealing on their medium-traffic rural roads and 73 percent perform it on their high-traffic urban roads.

This chapter presents detailed discussions 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. For pavements initially sealed at the time of construction, the general recommendation is that the joints should continue to be regularly resealed over the life of the pavement in order to minimize the infiltration of water and incompressible materials.

It is recognized that some agencies differentiate between joint/crack "sealing" and joint/crack "filling." Sealing employs additional preparation of the joint/crack channel, including the provision of a designed reservoir, along with the use of generally higher-quality materials. Filling includes very little preparation and generally uses lower-quality materials. The focus of this chapter is on sealing activities in either a joint or crack application.

3. Purpose and Project Selection

As previously described, free water entering joints or cracks can accumulate beneath the slab, contributing to conditions such as pumping, loss of support, faulting, corner breaks, and concrete deterioration. This is most often the case for concrete pavements constructed on erodible bases and exposed to high truck traffic levels. In addition, incompressibles that infiltrate poorly sealed joints or cracks can interfere with normal opening and closing movements, causing compressive stresses in the slab and increasing the potential for spalling. If the compressive stresses exceed the compressive strength of the deteriorated pavement, blowups or buckling may occur. Even if blowups do not occur, continual intrusion of incompressibles may cause the pavement to "grow." This growth can force movement of nearby bridge abutments or other pavement structures that may, over time, cause serious damage and necessitate major rehabilitation.

The performance of the joint and crack sealing treatments (i.e., how long they effectively perform their primary functions) varies considerably with the type of material, the reservoir design, prevailing climatic conditions, and the quality of the installation process. Based on a review of a number of available studies, the performance of concrete joint resealing installations was noted to range from 2 to 8 years, while the performance of concrete crack sealing was noted to range from 4 to 7 years (Peshkin et al. 2011); these are based on a failure definition of 25 percent of the sealant installation being no longer functional. That is not to say, however, that longer performance lives are not possible. For instance, using nearly 7 years of performance data, the 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, a recent pavement evaluation documented performance lives of more than 20 years for silicone sealants on two separate paving projects, one located in Arizona and one located in Washington State (Scofield 2013).

Sealing operations in concrete pavements may be performed at both joints and cracks to minimize water ingress and to prevent the infiltration of incompressibles. Most joint sealing and resealing operations focus on the transverse joints, because achieving an effective long-term seal at these joints is more challenging than at longitudinal joints because of the greater ranges of movement typically experienced. Because a substantial portion of the water that enters a pavement from the surface does so through the longitudinal joints—one study indicated as much as 80 percent entering through the lane-shoulder joint (Barksdale and Hicks 1979)—those joints are often sealed at the same time. The importance of sealing the longitudinal lane-shoulder joint was demonstrated in a preventive maintenance study conducted at the Minnesota road research test facility, Mn/ROAD (Olson and Roberson 2003). That study showed that the amount of water entering the pavement system can be reduced by as much as 85 percent by sealing the joint between the concrete mainline pavement and the asphalt shoulder.

As noted previously, the general recommendation is that transverse joints should be resealed if they were sealed at the time of original construction. Some agencies may choose not to reseal transverse joints, but only when design factors (e.g., narrow joints, drainable bases), climatic conditions (e.g., low annual rainfall), or their local experience favor such a decision.

Application of Joint Resealing

Joint resealing should be performed when the existing sealant material is no longer performing its intended functions. This is indicated by missing sealant, sealant that is in place but not bonded to the joint faces, or sealed joints that contain incompressibles; see Figure 10.1. Some agencies specify that joints be resealed when a certain amount of sealant material (typically 25 to 50 percent 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, Smith, and Romine 1999). Furthermore, the pavement should still be performing well and be in relatively good condition.

The optimum time of the year to perform joint resealing is generally during the spring and 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 are expected when the pavement is not severely deteriorated and when joint resealing is performed in conjunction with other pavement restoration activities, such as FDR, partial-depth repair, DBR, and diamond grinding.

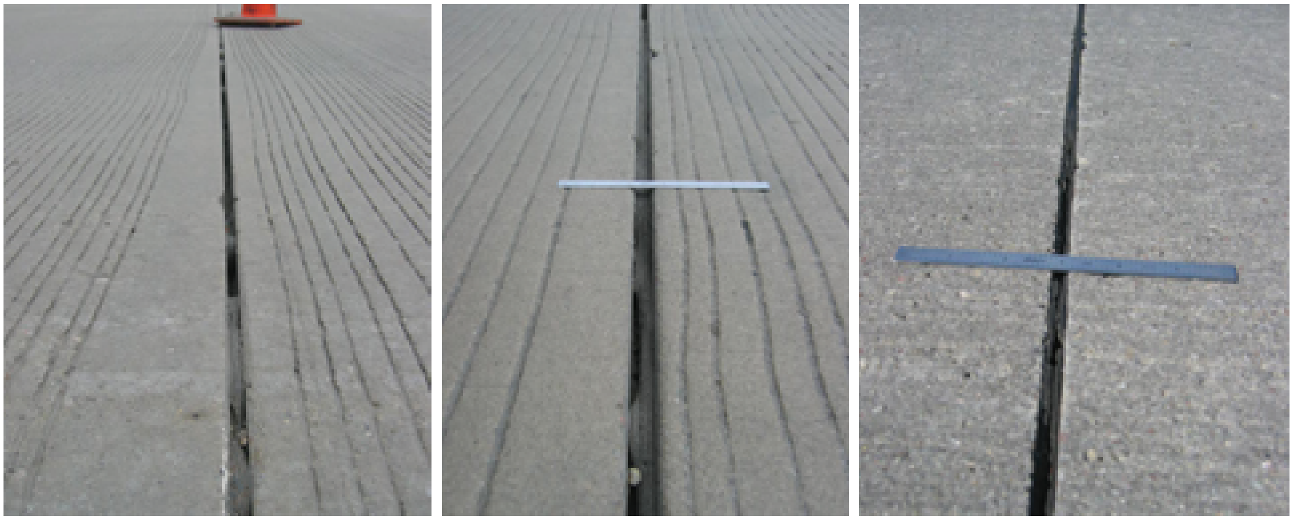


Figure 10.1. Example of joint sealant failures

Application of Crack Sealing

Crack sealing is a comprehensive operation involving thorough crack preparation and placement of high-quality materials into or over candidate cracks to significantly reduce moisture infiltration and to retard the rate of crack deterioration. Crack sealing is most effective when performed on concrete pavements that exhibit minimal structural deterioration and when the cracks are relatively narrow with minimal spalling and faulting. Crack sealing may, however, be used on random transverse and longitudinal cracks of low or medium severity where the crack width is 13 mm (0.5 in.) or less (ACPA 1995). Full-depth working transverse cracks can experience about the same range of movement as transverse joints; therefore, it is recommended that these cracks be sealed to reduce the potential of incompressible and water infiltration. 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 sealing them (ACPA 1995).

4. Material Selection

When planning a joint resealing project, one of the primary design activities is the selection of an appropriate sealant material. Material selection is dependent on a number of factors, including the following:

- Climate conditions (at the time of installation and during the life of the sealant).
- Joint/crack characteristics and spacing/density.
- Traffic level and percent trucks.
- Material availability and cost.

The first two factors govern the range of movement that the joints/cracks—and the sealant installed in them—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 movement and sealant elongation is available that can be used to assist in the material selection process (ACPA 2013a).

The remainder of this section discusses material selection considerations for a given sealing project. Specifically, this section introduces the different types of sealant materials that are typically used on concrete pavement joint resealing and crack sealing projects, introduces some of the more critical performance-related material properties, and discusses cost considerations that may impact the selection of the sealant material.

Available Material Types

Joint resealing and crack sealing operations generally employ either hot-applied thermoplastic materials or cold-applied thermosetting sealant materials. Table 10.1 lists some of the hot- and cold-applied materials available for sealing joints and cracks in concrete pavements (modified from ACPA 2006). Details of the different material type categories typically used for joint resealing or crack sealing projects are described below.

Note that even though preformed sealant types are included in Table 10.1, those materials see more widespread application in new pavement construction, particularly where long-term performance is sought (ACPA 2006). Their use in a resealing operation may be precluded by various challenges, including uneven joint widths along a joint, the presence of minor spalling along the joint, and nonuniform joint widths throughout a project.

Hot-Applied Thermoplastic Sealant Materials

Thermoplastic 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. Thermoplastic sealants are typically applied in a heated form (i.e., hot applied) on concrete pavements, although some are diluted such that they can be installed without heat (i.e., cold applied).

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. In recent years, softer grades of asphalt cement have been used in polymerized/rubberized asphalts 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 polymerized/rubberized asphalt materials are governed by ASTM D 6690, which includes four classes of sealants to better match low-temperature performance with climate. The left photo of Figure 10.2 shows a transverse joint sealed with a hot-applied thermoplastic material.

Table 10.1. Common Material Types and Related Specifications for Concrete Pavement Joint Sealing and Resealing (adapted from ACPA [2006])

Material Type	Specification(s)	Description
Liquid, Hot-Applied Sealants		***Thermoplastic***
Polymerized/Rubberized Asphalts	ASTM D 6690, Type I (AASHTO M 324)	Moderate climates, 50% extension at 0°F (-18°C)
	ASTM D 6690, Type II (AASHTO M 324)	Most climates, 50% extension at -20°F (-29°C)
	ASTM D 6690, Type III (AASHTO M 324)	Most climates, 50% extension at -20°F (-29°C) with other special tests
	ASTM D 6690, Type IV (AASHTO M 324)	Very cold climates, 200% extension at -20°F (-29°C)
Liquid, Cold/Ambient-Applied Sealants		***Thermosetting***
Single-Component Silicone	ASTM D 5893, Type NS	Non-sag, toolable, low modulus
	ASTM D 5893, Type SL	Self-leveling, no tooling, low modulus
Two-Component Elastomeric Polymer (polysulfides, polyurethanes)	Fed Spec SS-S-200E, Type M	Jet-fuel resistant, jet-blast resistant, machine-applied fast-cure
	Fed Spec SS-S-200E, Type H	Jet-fuel resistant, jet-blast resistant, hand-mixed retarded-cure
Solid, Cold/Ambient-Applied Sealants		
Preformed Compression Seals –Polychloroprene Elastomeric (Neoprene) –Lubricant	ASTM D 2628 ASTM D 2835	Jet-fuel resistant preformed seal Used in installation of preformed seal
Expansion Joint Filler		
Preformed Filler Material	ASTM D 1751 (AASHTO M 213)	Bituminous, nonextruding, resilient
	ASTM D 1752, Types I–IV (AASHTO M 153)	Sponge rubber, cork, and recycled PVC
	ASTM D 994 (AASHTO M 33)	Bituminous
Backer Rod (if used)	ASTM D 5249	For hot- or cold-applied sealants

Note 1: ASTM D 1190 was withdrawn in 2002 and replaced with ASTM D 6690 (Type I).
Note 2: ASTM D 3405 was withdrawn in 2002 and replaced with ASTM D 6690 (Type II).
Note 3: The use of preformed compression seals in resealing operations will depend on the condition of the joints.
Note 4: A few agencies no long use backer rods because of concerns that they trap moisture in the joint.

Figure 10.2. Hot-applied thermoplastic (left) and silicone (right) sealants

Ch 10. Joint and Crack Sealing

Cold-Applied Thermosetting Sealant Materials

Thermosetting sealants are typically one- or two-component materials that either set by the release of solvents or cure through a chemical reaction. Some of these sealants have shown potential for good performance, but the material costs are typically higher than standard polymerized/rubberized asphalt. Thermosetting sealants, however, are often placed thinner and may have slightly lower labor and equipment costs because of less time required for daily preparation and cleanup (e.g., no initial material heating, no purging of lines and pump).

A variety of thermosetting sealant materials is available, including polysulfides, polyurethanes, and silicones. Of these, silicones have been most widely used in pavement applications and have demonstrated long-term performance capabilities. Polysulfide and polyurethane sealants are not widely used in highway sealing/resealing operations.

Silicone sealants are one-part cold-applied materials that exhibit good extensibility and strong resistance to weathering. These sealants have good bonding strength in combination with a low modulus, which allows them to be placed thinner than the thermoplastic sealants. The right photograph in Figure 10.2 shows a project with both the transverse and longitudinal joints sealed with a silicone material.

Silicone sealants are governed by ASTM D 5893, which includes two classes of material—nonsag and self-leveling. The nonsag or nonself-leveling 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, some agencies have noted problems with the adhesion of silicone to concrete containing certain dolomitic aggregates, even when a primer was used (McGhee 1995). Under such conditions, the use of the silicone sealant should be carefully considered.

Sealant Properties

Critical sealant properties that significantly affect the performance of the sealant material include the following:

- **Durability**—Durability refers to the ability of the sealant to withstand the effects of traffic, moisture, sunshine, and climatic variation. A sealant that is not durable will blister, harden, and crack in a relatively short time. And, if exposed to traffic because of sealant placement configuration and/or extrusion from a closed joint, a nondurable sealant may soften under higher temperatures and may wear away under traffic.
- **Extensibility**—The extensibility of a sealant controls the ability of the sealant to 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 thermoplastic sealants, however, resilience and resistance to stone intrusion are often sacrificed in order to obtain extensibility. Hence, a compromise is generally warranted, taking into consideration the expected joint or crack movement and the presence of incompressible materials for 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 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 determines the sealant's bonding ability. Cohesive failures are most common in sealants that have been placed too thin in depth and/or that have hardened significantly over time, losing their elasticity.

Cost Considerations

Thermoplastic materials are generally less expensive than the thermosetting materials, with 2013 installed costs ranging roughly from about $3.28 per linear meter ($1.00 per linear foot) for thermoplastic materials to about $6.56 per linear meter ($2.00 per linear foot) for thermosetting materials. Because these costs can vary considerably geographically and by the size of the project and the design of the joint reservoir, however, local contractors and suppliers should be consulted for more accurate values. Furthermore, when making any cost comparisons, the total installation cost and the anticipated life of the sealant material must be considered. Some of the better-performing materials may have a higher unit cost, but they may last sufficiently longer or require less material so that the overall (life-cycle) cost of the materials may actually be lower.

5. Design Considerations

After the selection of a suitable sealant material has been made, the design of a joint resealing or crack sealing project requires decisions to be made regarding the selection of the sealant reservoir dimensions and an appropriate sealant configuration. 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 pavement-tire noise. A tool is available that can be used to evaluate the impact of joint geometry (sealed or unsealed) on existing pavement-tire noise levels (ACPA 2013b).

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 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 (ACPA 2013a).

Joint Shape Factor

Sealant Stresses

The performance of thermoplastic and thermosetting sealants (such as polymerized/rubberized asphalt and silicone) 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 10.3 illustrates the stresses produced in sealants placed to different depths. 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 shallower sealant. 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. With the deeper sealant, the necking down effect and the resultant stresses are greater.

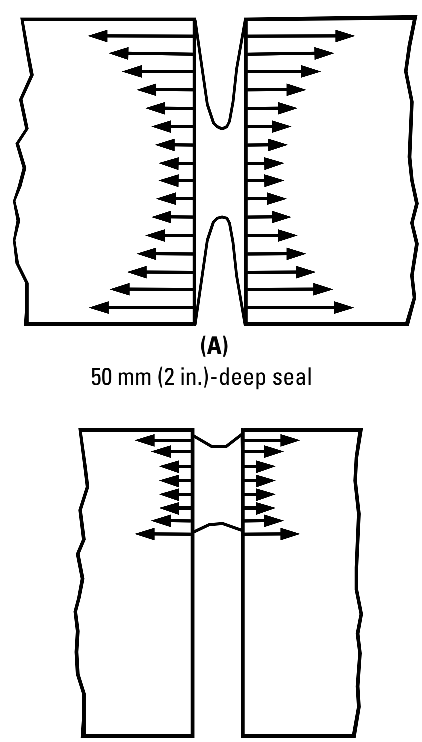


Figure 10.3. Relative effect of shape factor on sealant stresses

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 10.4. A proper shape factor minimizes the stresses that develop within the sealant and along the sealant/pavement interface as the joint opens.

For good performance, the sealant must also be kept from bonding to the bottom of the reservoir. A backer rod, also shown in Figure 10.4, may be installed prior to sealing to not only prevent the sealant from bonding to the bottom of the reservoir, but also to help achieve the desired sealant shape factor and to prevent the uncured sealant from running down into the crack beneath the reservoir.

The backer rod must be flexible and compressible, and it should be nonabsorbent. These materials should be selected such that their uncompressed width is about 25 percent larger than the width of the joint or crack in which they will be placed. Both closed-cell and open-cell backer rods—see Figure 10.5—are commonly available, but closed-cell products are recommended because they are more resistant to moisture absorption than open-cell materials (SNS 2011b). It is equally important that the selected backer rod be compatible with the selected sealant material and as specified by the sealant manufacturer. Backer rods are commonly manufactured from polyethylene, polyurethane, polychloroprene, or polystyrene; materials such as paper, rope, or cord should not be used (ACPA 2006).

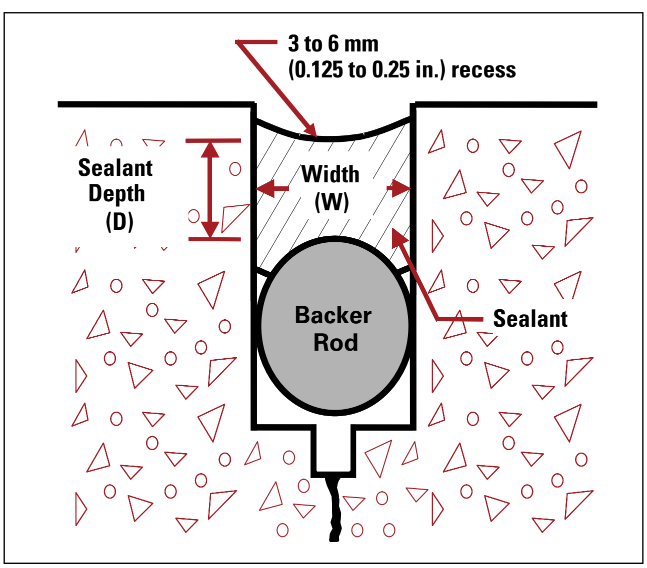
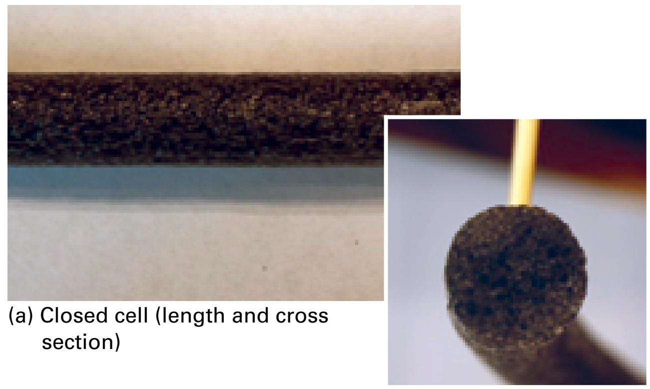
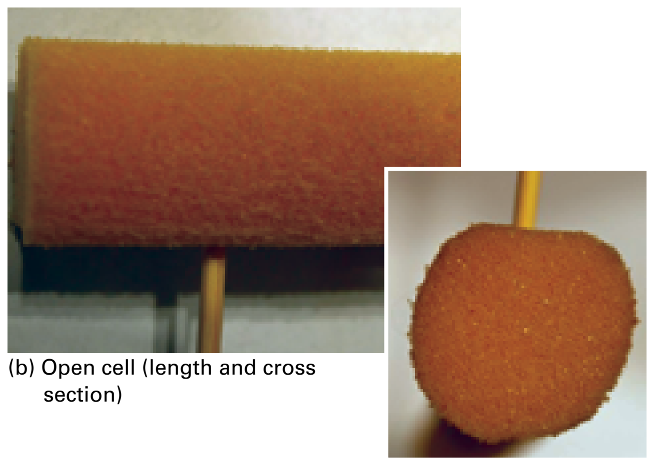


Figure 10.4. Illustration of sealant shape factor

The use of a backer rod in joint sealing operations should be considered with caution in some cases. For example, a recent study suggests that a backer rod may trap water beneath the sealant, thereby contributing to the deterioration of the concrete at or below the joint, as shown in Figure 10.6 (Taylor et al. 2012). This may



(a) Closed cell (length and cross section)



(b) Open cell (length and cross section)

Figure 10.5. Various types of backer rods (courtesy of Larry Scofield, IGGA)

Figure 10.6. Illustration of joint deterioration below the joint sealant (Taylor et al. 2012)

be particularly problematic when the concrete is of marginal durability. Because of this potential, a few agencies no longer use a backer rod but instead fill the entire reservoir with sealant to prevent water from ponding.

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 thermoplastic sealants on the market today are designed to withstand strains of roughly 25 to 35 percent of their original width, whereas silicone sealants are designed to tolerate strains from 50 to 100 percent. As an example, a thermoplastic material placed in a 13-mm (0.5-in.) wide joint can withstand an opening of 3 mm (0.125 in.) (13 mm x 25 percent) before exceeding a strain of 25 percent. A silicone material placed in a 13-mm (0.5-in.) wide joint can withstand an opening of 6.5 mm (0.25 in.) (13 mm x 50 percent) before exceeding a strain of 50 percent.

Shape factors recommended for different sealant types are summarized in Table 10.2 (Evans, Smith, and Romine 1999). In addition, it is also recommended that cold-applied silicone and polysulfide/polyurethane sealants be recessed below the surface; typical recess values range from about 3 to 6 mm (0.125 to 0.25 in.), although some agencies specify larger recess values (up to 12 mm [0.50 in.]). These recommendations assume that the joints are opened to a uniform width.

Table 10.2. Typical Recommended Shape Factors (Evans, Smith, and Romine 1999)

Sealant Material Type	Typical Shape Factor (W:D)
Polymerized/Rubberized Asphalt	1:1
Silicone	2:1
Polysulfide and Polyurethane	1:1

Longitudinal Joints

Because of the limited amount of movement, concrete-to-concrete longitudinal joints rarely have a designed reservoir. These joints are typically very narrow (around 6 mm [0.25 in.] wide) and are commonly sealed or filled with a thermoplastic material. A backer rod is often not used.

For longitudinal joints between a mainline concrete pavement and a hot-mix asphalt (HMA) shoulder, vertical movements are the primary concern. This joint, which as noted earlier can be a primary entry point for infiltration of surface water, is particularly difficult to seal because of the differential vertical movement that occurs between the two materials (Barksdale and Hicks 1979). The differential vertical movements are due to the structural differences of their cross sections and to the differences in the thermal properties of the materials. Settlements or heaving of the shoulder are quite common along these joints, and they often will require a wider reservoir to withstand that vertical movement. A reservoir configuration of either 19 mm by 19 mm (0.75 in. by 0.75 in.) or 25 mm by 25 mm (1 in. by 1 in.) for the lane-shoulder joint is suggested in order to accommodate the anticipated movements.

Cracks

Crack seal design should largely follow the precepts of transverse joint reseal design, particularly if the cracks are full-depth transverse working cracks. A crack reservoir will be created using a diamond-bladed saw. The width of the reservoir will generally be governed by the upper end of the range of crack widths that exist throughout the project, such that one standard saw-cut width can be used. Typically, the reservoir width will range between 6 and 13 mm (0.25 and 0.5 in.), covering cracks that are wider than 3 mm (0.125 in.) yet no more than about 13 mm (0.5 in.). Cracks wider than 13 mm (0.5 in.) should be addressed through more appropriate means such as FDR or load transfer retrofit, particularly if they have an appreciable amount of spalling or faulting.

Sealant Configurations

Joints in concrete pavements are typically sealed in the recessed configuration shown in Figure 10.7. Some manufacturers of hot-applied thermoplastic materials, however, recommend that the recess be eliminated and that the sealant be installed flush with the pavement surface. The benefits of this modification are the tendency for these sealants to remain more ductile when subjected to the kneading action of passing tires and the elimination of the reservoir area where sand, pebbles, and other debris can collect and potentially cause joint damage. Also, with reduced exposure to standing water, sealants placed in the flush-fill configuration experience less age-hardening damage. This was demonstrated on a long-term evaluation of sealants placed on an airfield, where the flush-fill configuration increased the life of the hot-applied sealants by more than 50 percent when compared to those placed in the standard recessed configuration (Lynch et al. 2013).

The overband configuration shown in Figure 10.7 is perceived to perform better because of the additional bonding area, but its use is not universally appropriate. For example, three disadvantages of the practice of overbanding are as follows (Evans, Smith, and Romine 1999):

- On high-trafficked pavements, overbanded sealant material is typically worn away by traffic within 1–3 years. After the overband is worn, traffic tires can pull the sealant from the joint edge (particularly if the seal is already partly debonded), leading to adhesion failure.
- Snowplow blades used on highways in cold regions tend to damage overbanded sealants by pulling them up from the pavement surface.
- The overband can negatively impact ride quality and create an aesthetically unpleasant surface (note that this can also occur with flush-filled seals, depending on the time of the year the resealing is performed).

Silicone sealants should never be placed in overband configuration or placed flush with the pavement surface. Manufacturers of silicone sealants recommend a minimum of 6–9 mm (0.25–0.38 in.) recess below the surface to prevent abrasion and wear of the sealant (Evans, Smith, and Romine 1999).

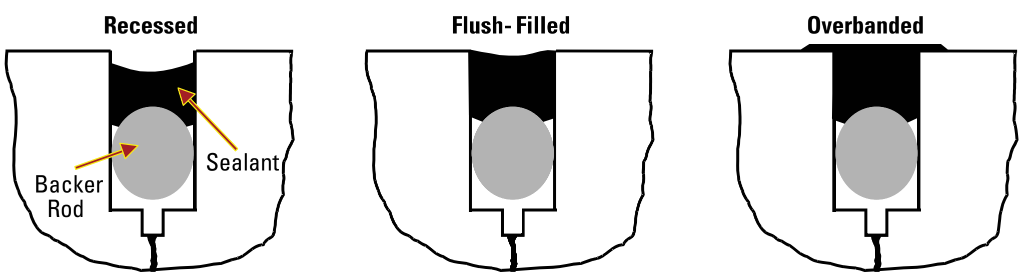


Figure 10.7. Schematic illustrations of various joint sealant configurations

Ch 10. Joint and Crack Sealing

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 performance of the sealant. Many projects have performed poorly because of improper or inadequate construction practices, and this section presents the recommended procedures for an effective sealant installation. It is noted that joint sealing is often required in conjunction with other preservation activities (such as full- or partial-depth repairs), and the same general steps are followed in those applications. In all cases, successful sealing projects require close attention to detail.

Transverse Joint Resealing

The resealing of transverse joints in concrete pavements consists of the following steps, each of which is described in detail in 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 incompressibles. Initial removal can be done by any procedure that does not damage the joint itself, such as using a rectangular joint plow or removal with a diamond-bladed saw. Diamond-bladed sawing as a means of sealant removal has gained widespread acceptance because it combines the sealant removal and joint refacing steps in a single process. It is most effective at removing existing silicone sealants and existing thermoplastic sealants when they have hardened and will not melt and "gum up" the saw blade or joint face. If a joint plow is used, it should be rectangular shaped 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 for bonding with the new sealant and to establish a reservoir with the desired shape factor. If a diamond-bladed saw has been 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 the old sealant material, then a separate joint refacing operation must be performed.

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; see Figure 10.8. The use of ganged blades may be quicker, but it could lead to excessive widening depending on how uniform the joint widths are in a project; in those cases, two passes of a single sawblade to "skim" the edges of the joint may be more suitable.

Typically, a joint should be widened by no more than 2 mm (0.08 in.) total during the refacing operation. This will limit the amount of concrete that is removed, increase production, and limit the width of the joint through successive joint resealing operations (which could lead to tire slap noise issues).

The use of routers is not recommended for joint refacing operations. Although they have been used in the past, their production rate is much slower than diamond-bladed saws. In addition, they can leave irregular or spalled joint walls and may smear the existing sealant on the sidewalls.

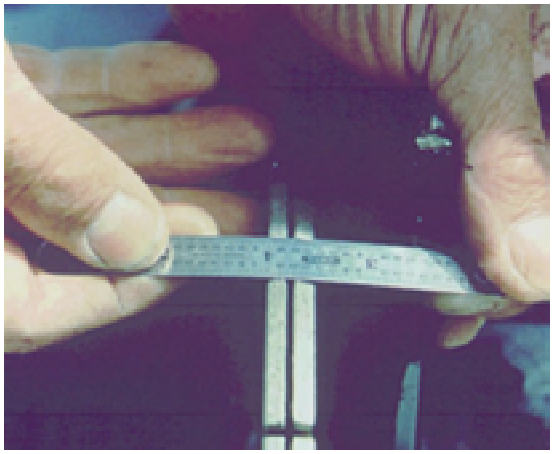


Figure 10.8. Ganged sawblades to provide desired cutting width

Step 3: Joint Reservoir Cleaning

The importance of effective cleaning of the joint sidewalls cannot be overemphasized. 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 the 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

Immediately after joint refacing, the joint should be cleaned with high-pressure air followed by light sandblasting; see Figure 10.9. Sandblasting effectively removes laitance (wet-sawing dust) and any other residue on the joint faces, and it should be conducted in two passes so that each joint face is cleaned. Air compressors used with the sandblasters must be equipped with working water and oil traps to prevent contamination of the joint bonding faces. Compressors should be tested prior to sandblasting operations using a clean white cloth to ensure oil/water-free operations. Water blasting may occasionally be used for cleaning in applications where sandblasting 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).

Figure 10.9. Sandblasting along a transverse joint

The sandblasting operation should proceed along each side of the joint and should result in joint sidewalls that are clean and dry and exhibit newly exposed concrete. For optimum cleaning, the nozzle on the sandblast wand should be held no more than 50 mm (2 in.) from the pavement surface. The rate of progression along the joint should be slow enough such that the joint sidewalls are effectively cleaned, yet fast enough such that spalling of the joint edge or other joint damage does not occur.

For worker protection, the sandblasting equipment should include a remote shut-off valve and protective clothing for the operator (Evans et al. 1999). In addition, the operator must be equipped with an air-fed protective helmet and an air supply purifier to avoid the risk of silicosis, an occupational lung disease caused by the inhalation of crystalline silica dust. For traffic protection, portable protective barriers should be installed, as appropriate, between the sandblaster and adjacent traffic.

Following sandblasting and immediately prior to backer rod and sealant installation, the joints should be blown again with high pressure (> 621 kPa [90 lbf/in.^{2}]) clean, dry air to remove sand, dust, and other incompressibles that remain in the joint. A backpack blower typically cannot generate sufficient pressure to clean joints thoroughly and should not be used for final cleaning. Joints and surrounding surfaces should be airblown in one direction away from prevailing winds, taking care not to contaminate previously cleaned joints. Care must also be taken not 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 essentially ineffective, and it can smear the old sealant across the concrete sidewall and/or leave a metal sheen, creating a surface to which the new sealant cannot bond.

Step 4: Backer Rod Installation

As previously described, the backer rod must be a flexible, nonabsorptive material that is compatible with the sealant material in use. Closed-cell products are recommended because of their nonabsorptive nature. The melting temperature of the backer material should be at least 14°C (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 as soon as possible after the joints are air blasted, and it should be about 25 percent 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. Wide joints or segments of joints in which the backer rod does not provide a tight seal should be filled with a larger-diameter backer rod. Figure 10.10 shows the backer rod being placed to the proper depth with a special roller.

Figure 10.10. Installation of the backer rod

Step 5: New Sealant Installation

As soon as possible after backer rod placement, the sealant material should be installed. This helps to avoid 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.

Hot-Applied Thermoplastic Sealant Materials

Hot-applied thermoplastic sealant materials should be placed only when the air temperature is at least 4°C (40°F) and rising (FHWA 2002). 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 3–6 mm (0.125–0.25 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 and overband 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 10.11 shows the installation of a hot-applied sealant material in a joint resealing project.

Figure 10.11. Installation of hot-applied joint sealant

It is also important to follow the manufacturer's recommendations with regard to the maximum sealant temperature, the recommended placement temperature, and any prolonged heating limitations. Many of the polymerized/rubberized sealants break down or degrade when subjected to temperatures above the recommended safe heating temperature. Prolonged heating can cause some sealant materials to gel in the heating tank, while others experience significant changes in their elastic properties. 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 the new sealant, can reduce sealant performance. Using an additional thermometer to monitor sealant temperatures can help eliminate damage due to sealant overheating.

Cold-Applied Thermosetting Sealant Materials

Silicone sealants should not be placed at temperatures below 4°C (40°F). As with the thermoplastic materials, 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 not recommended that they be placed any thinner than half the width of the joint, with a minimum thickness of 6 mm (0.25 in.). For narrow joints (say, 6 mm [0.25 in.] wide), a 1:1 shape factor will be required, in accordance with the manufacturer's recommendations. Traffic should not be allowed on the pavement for about 1 hour after sealant placement. Again, the sealant manufacturer should be consulted for recommendations for exposing the sealant to traffic.

As mentioned previously, silicone materials come in two varieties: nonself-leveling and self-leveling. The nonself-leveling 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 6 mm (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. Figure 10.12 shows the tooling operation as it follows the installation of a silicone sealant on a new concrete pavement construction project.

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 the sealant can easily flow around a loose backer rod prior to curing and may flow out at the joint ends if not properly blocked. In addition, because the joints in resealing operations are rarely uniform in width, there may be a tendency for the sealant to not be placed to consistent depth and to lack sufficient contact with the concrete for bonding. As a result, some agencies have mandated tooling in order to achieve the required depth and to enhance the bond between the pavement and the sealant.

When installing both silicone and thermoplastic sealants, such as in a project with silicone sealant in the transverse joints and hot-applied thermoplastic materials 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.

Although not commonly used in highway applications, polysulfide and polyurethane sealants require a curing period to gain their strength and resiliency, similar to silicone. In addition, these sealants require a special application nozzle and careful control of the application equipment. Traffic should not be allowed on these sealants until the surface has skinned over and the possibility for stone intrusion is minimized.

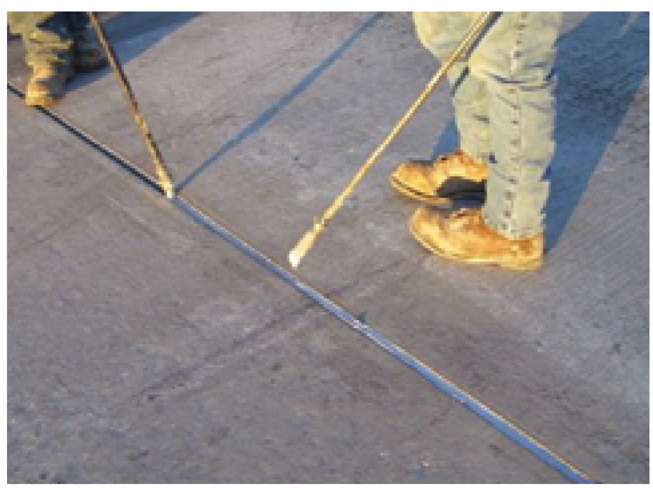


Figure 10.12. Sealant placement at left followed by tooling operation at right (courtesy of John Roberts, IGGA)

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 the 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-to-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. This joint is generally tied together with deformed tiebars 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-poured thermoplastic material. A few agencies may use silicone for the longitudinal joints, particularly if they are already using silicone in the transverse joints. If the transverse joints are to be sealed with silicone and the longitudinal joints with hot-applied sealant, however, it is important that the longitudinal joints be sealed last to prevent contamination of the transverse joints with the hot-applied material.

Concrete Mainline/HMA 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/warping of the concrete, the lack of a tie between the concrete mainline and the HMA shoulder, and frost heave/swelling in the subgrade beneath the shoulder.

Again, the steps required for the sealing of lane-shoulder joints are the same as 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 19 mm by 19 mm (0.75 in. by 0.75 in.) to 25 mm by 25 mm (1 in. by 1 in.). The reservoir can be created using either a router or a diamond-bladed saw. Figure 10.13 shows pictures of the prepared and sealed joint between a concrete mainline and asphalt shoulder.

The joint reservoir between the concrete mainline and asphalt shoulder should be cleaned prior to the placement of the 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 thermoplastic materials to seal this joint, although there are some silicone materials that have been specifically developed for this type of application.

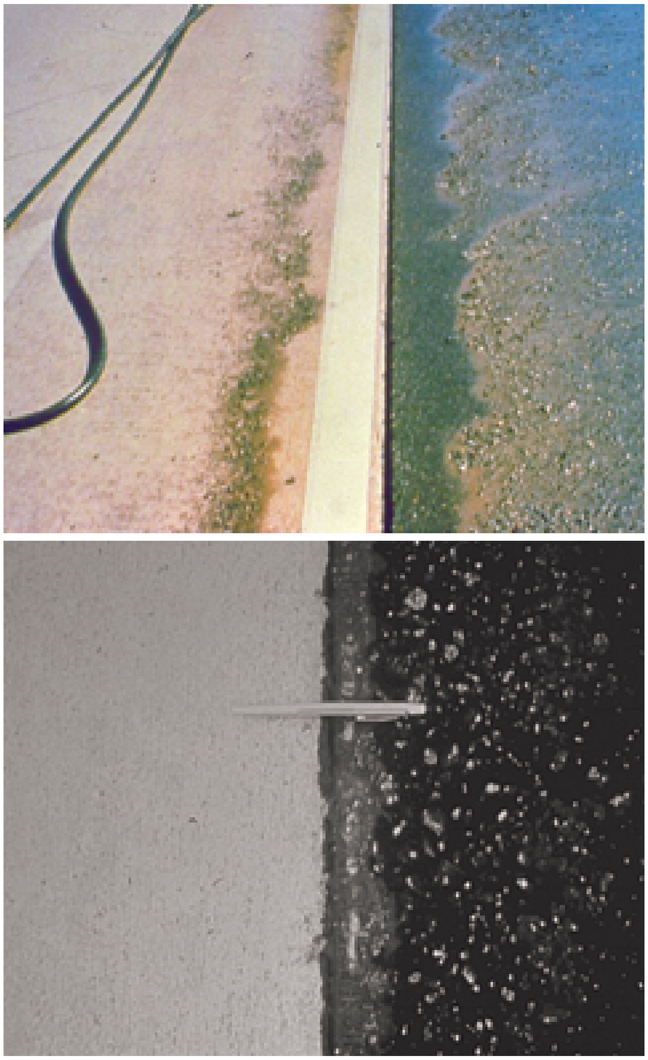


Figure 10.13. Prepared (top) and sealed (bottom) joint between concrete mainline and asphalt shoulder

Crack Sealing

With the exception of a 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. The random orientation of most concrete pavement cracks, however, makes it difficult to create a uniform sealant reservoir directly over the crack. The formation of a reservoir should be accomplished with a small-diameter diamond-bladed saw (ACPA 2006). Note that while crack routers have been used in the past to form sealant reservoirs, their use is not recommended because of the chipping and microcracking damage this equipment causes to the concrete (ACPA 2004).

The cutting blades for the crack saws are typically 175–200 mm (7–8 in.) in diameter and 6–13 mm (0.25–0.5 in.) wide. The width of the sawcut generally provides an appropriate shape factor to accommodate the expected crack movement. Smaller blade diameters, in addition to lightweight two- or three-wheel unit designs, allow crack saws to pivot and follow irregular crack profiles.

Once the reservoir is created, the crack should be cleaned following those steps prescribed for joint resealing. Sandblasting is particularly recommended to remove laitance from the sawing operation. After cleaning, the crack is blown with high-pressure compressed air and the backer rod (if specified) and sealant material are installed. The same precautions that apply to the installation of sealant materials into joints also apply here (ACPA 1995).

Construction Equipment

Equipment for Sealant Removal and Joint/Crack Refacing

Joint Plow

A joint plow is a rectangular blade mounted on the hydraulic mount of a tractor or the bucket of a skid loader. The plow blade is inserted into the joint and pulled along each joint edge, scraping the sealant from the sidewalls. The blade must be rectangular and fit freely into the joint. A V-shaped blade should never be used because these blades can spall the joint. The rectangular tool must be mounted such that it is free to move vertically and horizontally in the joint without binding. Blades of several widths should be on hand, because joint widths are seldom uniform over an entire project. Because of the difficulty in consistently removing the old sealant from the joint sidewalls and the potential for damaging the joint, great care and attention to details must be exercised when using a joint plow.

Diamond-Bladed Saw

Diamond-bladed saws are typically 26- to 46-kW (35- to 65-hp), water-cooled devices equipped with diamond-edged blades. A single, full-width blade is useful for maintaining joint width; however, the edges wear quickly, reducing the effectiveness of the sawing. Two blades separated by a spacer to the desired width can be used on the same arbor.

Equipment for Joint/Crack Cleaning

Sandblasting Equipment

Sandblasting equipment consists of a compressed air unit, a sandblasting machine, hoses, and a wand with a venturi-type nozzle. The compressed air supply is the most critical part of the sandblasting operation. In general, it is recommended that at least 620 kPa (90 lbf/in.^{2}) of pressure and 4.3 m^3/min (150 ft^3/min) of oil- and moisture-free air be provided to the joint/crack. These rates may need to be adjusted somewhat, however, along with the rate of progression and the nozzle proximity and angle, to deliver the most effective cleaning without damaging the joint/crack. The use of a jig is also strongly recommended to reduce operator fatigue and to ensure that the sandblast nozzle is properly positioned to direct sand against the sidewalls to provide more efficient cleaning (Evans, Smith, and Romine 1999).

Air Blasting Equipment

Air blasting equipment consists of high-pressure air compressors with hoses and wands. High-pressure air compressors are effective at removing dust and debris from a joint/crack, but they are not as effective as sandblasting at removing laitance. As a minimum, compressed air units should have a blast pressure of 690 kPa (100 lbf/in.^{2}) and a blast volume of 4.3 m^3/min (150 ft^3/min). As discussed previously, air compressors should be equipped with working moisture and oil traps to prevent contamination of the joint/crack faces.

Equipment for Joint/Crack Sealant Placement

Melters

Hot-applied thermoplastic materials are heated and mixed in an indirect-heat, agitator-type melter. These machines burn either propane or diesel fuel, and the resulting heat is applied to a transfer oil that surrounds a double-jacketed melting vat containing the sealant material. This indirect method of heating is safer and provides a more controlled and uniform heat.

Silicone Pumps

Single-component silicone materials are typically pumped from storage containers using compressed air-powered pumping equipment. A feed rate of at least 1.5 L/min (0.4 gal/min) is recommended, and the wand should be equipped with a nozzle that allows filling from the bottom up.

Applicators

Most sealant applicators are pressure-wand systems, normally equipped on sealant melters. The applicator consists of a pump, hoses, and an applicator wand. Sealant material is pumped directly from the melter-vat through the system and into the joint/crack.

7. Quality Assurance

Proper sealant application is a process that relies heavily upon the care and conscientiousness of the contractor. Paying close attention to this 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 by the FHWA (2002).

Preliminary Responsibilities

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

Project Review

An updated review of the project is warranted to ensure that it is still a viable candidate for joint resealing or crack sealing. Specifically, it should be verified that conditions have not significantly changed since the project was designed and that the prevailing distresses are still in the acceptable ranges used for project selection. Also, the selected methods for sealant removal, refacing, and cleaning should be reviewed. Finally, for joint resealing projects, the selected joint design and sealant type should be reviewed to make sure they are still appropriate for the expected project climate and conditions.

Document Review

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.
- MSDS.
- Agency application requirements.

Review of Materials

In preparation for the construction project, the following list summarizes many of the material-related items that should be checked or reviewed prior to construction (FHWA 2002):

- Sealant meets specification requirements.
- Sealant material is from an approved source or listed on agency QPL (if required).
- Sealant material has been sampled and tested prior to installation (if required).
- Sealant material packaging is not damaged (i.e., leaking, torn, or pierced).
- Backer rod is of the proper size and type for the selected sealant material.
- Chemically curing sealants (if used) are within shelf life.
- Sufficient quantities of all materials are available for completion of the project.

Inspection of Equipment

Prior to beginning 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 (FHWA 2002).

Hot-Applied Sealant Melters

For hot-applied sealant melters, an indirectly heated double boiler-type melter with effective agitation is typically used. Prior to construction, these melters should be inspected to ensure that they are in good working order and that all internal mechanisms (such as heating, agitation, pumping systems, valves, and thermostats) are functioning properly. Also, the contractor should verify that the proper size wand tips are available.

Cold-Applied Sealant Pumps (Single- and Two-Component Materials)

For cold-applied sealant materials, the contractor should make sure that the pump is in working order, the follower plates are in good shape and lubricated, and the hoses are not plugged. For two-component pumps, the contractor should verify that the pump contains a mixing head that meets manufacturer's requirements and that the pump is delivering material at the correct ratio.

Joint/Crack Cleaning Equipment

For the joint/crack cleaning equipment, the following items should be verified (FHWA 2002):

- Abrasive cleaning unit is adjusted for correct abrasive feed rate.
- Abrasive cleaning uses environmentally acceptable abrasive media.
- Abrasive cleaning operators use air purification systems as required.
- Air compressors have sufficient pressure and volume to adequately clean joints/cracks and meet agency requirements.
- Air compressors are equipped with oil and moisture filters/traps that are properly functioning.
- Joint plows (if used) are of correct size and configuration to remove required amount of old sealant without spalling joint edges.
- Concrete saws/blades are of sufficient size to adequately cut the required joint/crack width and depth, and the saw is in good working order.

Weather Requirements

The weather conditions at time of construction can have a large impact on the performance of the installed sealant. Specifically, the following weather-related items should be checked prior to construction (FHWA 2002):

- Review manufacturer installation instructions for requirements specific to the sealant material.
- Air and/or surface temperature meets manufacturer and all agency requirements (typically 4°C [40°F] and rising) for sawing and sealing.
- Sealing should not proceed if rain is imminent.
- Application does not begin if there is any sign of moisture on the nearby surface or in the joint/crack.

Traffic Control

Immediately prior to construction, it should be verified that the on-site traffic control signs and devices match those defined in the traffic control plan. Also, it should be verified that the setup complies with the Federal or local agency MUTCD.

After the sealing activities have been completed, traffic should not be allowed back on the pavement until the sealant has adequately cooled or cured so that it is not tracked by vehicle tires.

Construction Inspection Responsibilities

Joint/Crack Preparation

During the joint preparation steps, the inspector should ensure the following (FHWA 2002):

- All safety mechanisms and guards on equipment are in place and functioning properly, and operators are using required personal protective equipment.
- Old joint sealant is effectively removed from the joint.
- Joint/crack is refaced to produce a reservoir that meets the requirements of the selected sealant material.
- Joint/crack surfaces are cleaned using abrasive cleaning.
- Abrasive cleaning is accomplished with the nozzle 25–50 mm (1–2 in.) above the joint/crack using two passes (each directed at one of the joint/crack faces).
- Joint/crack is blown clean with clean, dry air. A propane torch or hot-air lance should not be used for drying.
- Joint/crack is inspected prior to sealing by rubbing finger along the sidewalls to insure that no contaminants (dust, dried saw residue, dirt, moisture, or oil) exist. If

dust or other contaminants are present, the joint/crack should be recleaned to a satisfactory condition.

Backer Rod Installation

During the backer rod installation process, the inspector should check that the backer rod is being installed uniformly to the required depth. Also, the inspector should check that the backer rod fits snugly in the joint/crack (no gaps along the side) and is not being stretched or damaged during installation.

Sealant Installation

Hot-Applied Sealant Materials

As previously discussed, many of the newer sealant materials are sensitive to heating and application temperatures. The use of supplementary temperature monitoring devices is recommended so that the sealant temperature can be closely observed. Underheating the material results in poor bonding, whereas overheating the material destroys its ductile properties and increases its aging.

More specifically, a project inspector should check or verify the following:

- Melter heat transfer medium is heated to the correct temperature range.
- Sealant is being heated into the manufacturer's recommended pouring or application temperature range. In addition, the inspector should check that the heating temperature does not exceed the material's safe heating temperature. The use of a supplementary temperature measuring device not part of the equipment is recommended.
- Sealant is continuously agitated to assure uniformity, except when adding additional material.
- Operator wears required personal protective equipment (e.g., air-fed protective helmet and air supply purifier for sandblasting operator).
- If melter is equipped with a heated hose system, the hose is heated prior to beginning sealant application.
- If melter does not have a heated hose, verify that the hose is clear and unclogged prior to beginning sealant application.
- If melter does not have a heated hose, the sealant should be recirculated through the hose to warm the hose prior to application. During idle periods, or if it is noted that sealant is cooling in the hose, sealant shall be recirculated through the hose back to the material vat to keep the hose at an acceptable temperature.
- Melting vat should be kept at least one-third full to help maintain temperature uniformity.
- Joint/crack is filled from the bottom up to the specified level to produce a uniform surface with no voids in the sealant.
- Detackifier or other blotter is applied to reduce tack prior to opening to traffic (if needed).
- Traffic is not allowed on the project until sealant is tack-free or cooled.
- Verify adequate adhesion at several random sections of cooled sealant. A simple knife test can be used to determine how well the sealant has adhered to the joint/crack sidewalls (ACPA 1995). Such a test consists of using a dull knife blade or thin metal strip to probe between the sealant and the sidewall. A loose, effortless penetration indicates adhesion loss, while good adhesion provides resistance (ACPA 1995).

Cold-Applied Sealant Materials (Single- and Two-Component)

During the installation of a single- or two-component sealant, as a minimum, the project inspector should check the following:

- Joint/crack is filled from the bottom up to the specified level to produce a uniform surface with no voids in the sealant.
- Nonself-leveling sealants (and self-leveling sealants, as appropriate) are properly tooled to force the material against the sidewalls and to form a smooth surface at the specified recess from the surface.
- Sealant is permitted to cure to a tack-free condition prior to opening to traffic.
- Verify adequate adhesion at several random sections of cured sealant. As with the hot-applied sealants, a simple knife test can be used to test for adhesion.

Cleanup

After the joint resealing or crack sealing construction process is complete, any excess sealant application or spills must be removed from the surface. Melters and other application equipment should be properly cleaned in preparation for their next use.

8. 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 (FHWA 2002; ACPA 2006)

Problem	Typical Solutions
Punctured or stretched backer rod	A punctured or stretched backer rod can result in an improper shape factor or adherence of sealant to bottom of 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 sharp edges (ACPA 1995). Note: The joint or crack will 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 sealant removal or joint cleaning steps. Note: A V-shaped joint plow blade can spall joint sidewalls. Irregularities on joint walls can reduce the sealant's lateral pressure, therefore allowing the sealant to extrude or pop from the joint (ACPA 1995). If irregularities are observed, the agency and contractor should agree on an appropriate method for repairing potential problem areas.
Sealant not adhering to joint/crack	• Reclean joint or crack. • Allow sidewalls to dry before sealing. • Heat to correct temperature or verify temperature gauges. • Wait for higher ambient temperature before sealing. • Use correct recess for joint/crack width (especially important for cold-applied sealants).
Sealant gelling in melting chamber (melter)	• Check melter temperature gauges. • Use fresh sealant. • Use sealant with longer pot life, or conform to manufacturer's recommended pot life.
Bumps or irregularities in surface of tooled sealant application	• Check 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 correct mix ratios and mixing systems.
Sealant picks up or pulls out when opened to traffic	• Close to traffic and delay opening. • Seal during cooler temperatures. • Apply sealant flush with surface or with specified recess. • Use stiffer sealant if too soft for climate. • Use a detackifier or blotter to reduce initial tack. • Install at correct temperature and continuously verify the temperature gauges on the melter. • Repeat preparation routine and then reseal joints/cracks that were contaminated with solvent or heat transfer oil. • Reclean joint/crack sidewalls to remove offending material and then reseal.
Voids or bubbles in cured sealant	• Seal during cooler periods and then allow concrete to further dry or use nonsag type sealant to resist void formation. • Backer may be melting with hot-applied sealants; use heat-resistant backer material and check for proper sealant temperature. • Install backer rod carefully to avoid damage (i.e., puncturing). • Apply sealant from the bottom up. • Tighten all connections and bleed off entrapped air. • Replace backer material if moisture is present. • Cure primer according to manufacturer's recommendations.
Sink holes in sealant	• Use larger backer material, reapply (top off) sealant to correct level, or use nonsag sealant. • Use heat-resistant backer material.

9. Summary

This chapter presents information on joint and crack sealing in concrete pavements. The need for sealing operations is discussed, including guidelines for identifying candidate projects. Various available sealant materials are presented, along with their properties, applicable specifications, and design considerations.

Procedures for the sealing of transverse joints, longitudinal joints, and cracks in concrete pavements are described. In almost every project, a successful sealing operation includes the following steps: removing the old material (joint resealing only), refacing the existing joint/crack reservoir, cleaning the reservoir, installing backer rod, and installing the new sealant material. Because the quality of the construction practices is extremely important to the long-term performance of the sealant installation, recommended quality control and troubleshooting procedures are also presented. These procedures also cover the safety of the workers and the traveling public.

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Chapter 11

Concrete Overlays

1. Learning Outcomes

This chapter describes concrete overlays as an important preservation and rehabilitation treatment under the category of resurfacing. The participants will be able to accomplish the following upon successful completion of this chapter:

- List benefits and appropriateness of using concrete overlays.
- List advantages and disadvantages of different overlays.
- Describe recommended materials.
- Identify recommended construction activities for each type of overlay.

2. Introduction

This chapter is a summary of the *Guide to Concrete Overlays, Sustainable Solutions for Resurfacing and Rehabilitating Existing Pavements*, 3rd Edition (Harrington and Fick 2014). To ensure the reader understands the entire breadth of concrete overlay solutions, all six types of concrete overlays are described (including overlays over asphalt and composite systems) and the full range of overlay strategies presented (from preservation to rehabilitation). The guide is available from the National Concrete Pavement Technology Center at www.cptechcenter.org/technical-library/library-search. The general term "concrete resurfacing" is used in this chapter when collectively discussing both bonded and unbonded concrete overlay solutions.

The use of concrete to resurface existing pavements can be traced to as early as 1901. An NCHRP (National Cooperative Highway Research Program) synthesis document (McGhee 1994) showed that a service life of 20 years with little maintenance can be expected and that many resurfacings have provided as much as 30–40 years of service. It was evident by the mid-1980s that many new concrete overlays were being constructed and that the technology was rapidly maturing into a standard practice in some agencies.

Concrete overlay solutions exist for all pavement types (concrete, asphalt, and composite [concrete with an asphalt surfacing]) and their conditions. The thickness of concrete overlays can vary from 51 to 254 mm (2 to 10 in.) or more, depending on the existing pavement condition, the anticipated traffic levels, and the desired design life. Generally speaking, concrete overlays in the thickness range of 51–102 mm (2–4 in.) fall within the pavement preservation window (preventive maintenance and minor rehabilitation), with thicker overlays considered more for major rehabilitation.

Concrete overlays share at least two design requirements with on-grade concrete pavement structures if satisfactory performance is to be realized: they require uniform support conditions and effective management of movement in the design process. Nearly all the documented cases of premature overlay failure can be traced to some violation of these simple requirements, often a result of "picking the wrong project" to resurface. For this reason, the evaluation of the existing pavement is paramount in determining if uniform support and movement control exists or if the underlying pavement and interface layer can be cost-effectively repaired or milled to remove surface deterioration.

Concrete resurfacing can be designed to cost-effectively accommodate all combinations of traffic loading and design life. Despite a demonstrated history of hundreds of successful concrete overlay projects, some agencies and contractors are hesitant to design and construct concrete overlays. This may be based on a combination of misconceptions that concrete overlays are expensive, difficult to build, or niche solutions that have limited applicability. Agencies that are new to adopting concrete overlays as a part of their asset management strategy often struggle with aligning expectations with desired performance and budgets. In these cases there is often an organizational perception that concrete pavements are limited to projects that require a long-term solution (20–50 years), while other alternatives are used for short-term solutions (5–15 years).

There are two options for concrete resurfacing: bonded overlays and unbonded overlays. Bonded overlay projects require that the existing pavement be in good to fair structural condition. The overlay help eliminates surface distresses, with the new overlay and exiting pavement acting as a monolithic pavement. Unbonded overlays add structural capacity to the existing pavement and do not require bonding to the existing pavement. Unbonded overlays are essentially a new pavement on a stable base (existing pavement). Figure 11.1 summarizes some of the characteristics and applications for both bonded and unbonded concrete overlay systems.

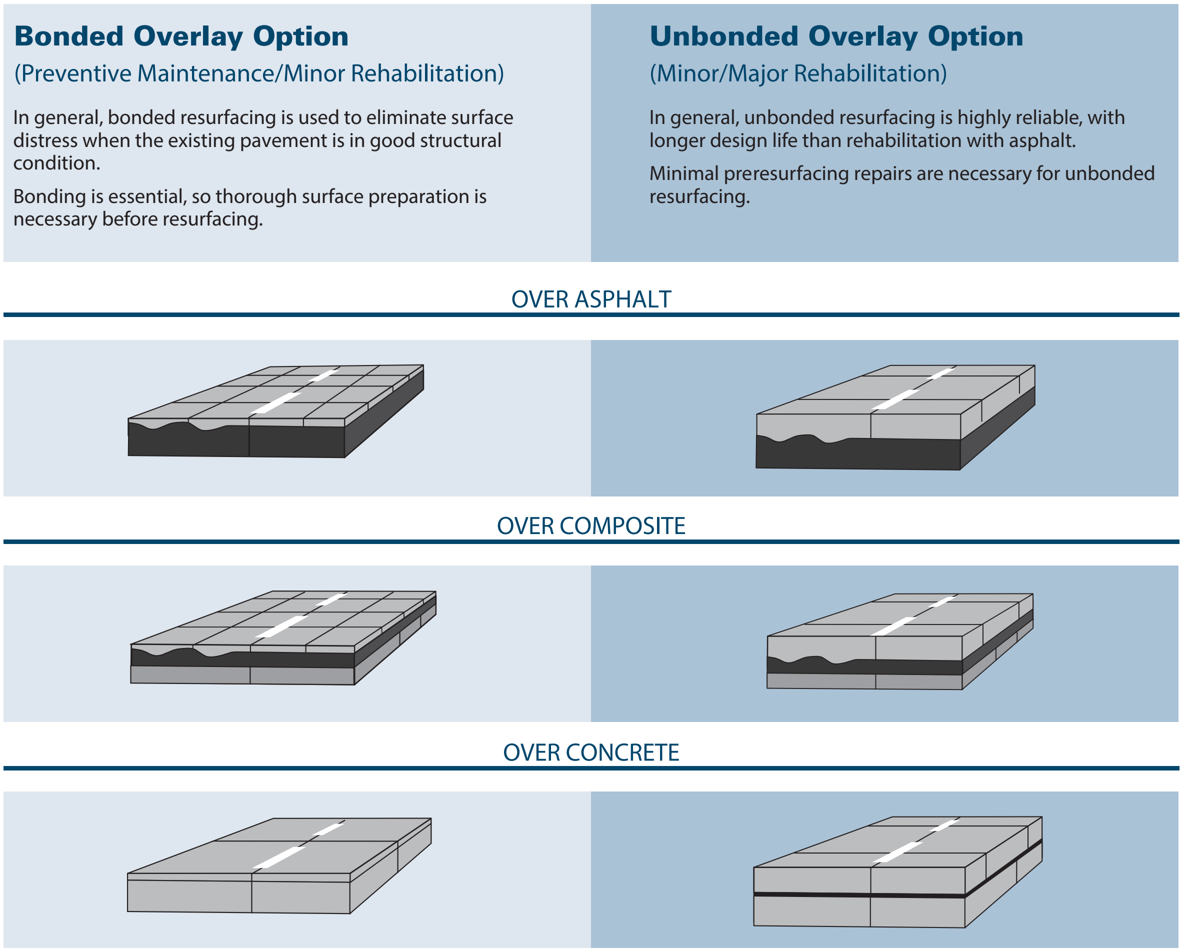


Figure 11.1. Types of concrete overlays (Harrington and Fick 2014)

Benefits of Concrete Overlays

Concrete overlays offer a number of different benefits, as summarized below:

- Concrete overlays consistently provide cost-effective solutions.
  - Dollar for dollar, they are one of the most effective, long-term pavement preservation or major rehabilitation options for existing pavements.
  - Because of the wide range of overlay thicknesses that can be used, combined with the minimal preoverlay work required, concrete overlays provide cost-effective solutions for almost any pavement type and condition, desired service life, and anticipated traffic loading.
- Concrete overlays can be constructed quickly and conveniently.
  - The existing pavement does not need to be removed. In fact, it is factored into the overlay design as contributing to some of the load-carrying capacity.
  - In most cases, minimal preoverlay repairs are necessary.
  - Concrete overlays are placed using normal concrete pavement construction practices.
  - Many concrete overlays can be opened to traffic within a day of placement. Nondestructive strength indicators, like maturity testing, enable engineers to take advantage of this benefit.
  - Accelerated construction practices can be used. This chapter provides recommendations for coupling concrete overlays with accelerated construction techniques.
- Concrete overlays are easy to maintain.
  - Repairing concrete overlays, especially thin overlays, is usually much easier than repairing a section of conventional pavement.

- Thin overlays constructed without reinforcement can be easily and economically milled out and replaced with a new concrete surface.
- Utility repair locations can be restored to original surface elevation and ride quality with ease.

- Concrete overlays can serve, in and of themselves, as complete preventive maintenance, preservation, or rehabilitation solutions.
- Concrete overlays are an effective means to enhance pavement sustainability by improving surface reflectance (albedo), increasing structural longevity, enhancing surface profile stability, and maintaining ride quality.

Resurfacing

Resurfacing is a generic term for providing a new or fresh surface on the existing pavement and is considered a concrete (minor and major) rehabilitation practice. Concrete resurfacing consists of both bonded and unbonded concrete overlays. It is an integral component of a comprehensive asset management approach because it cost-effectively extends pavement life and improves both functional and structural characteristics.

Figure 11.2 represents a typical pavement condition curve over the life of a pavement. The preservation and rehabilitation zones are noted where bonded, and unbonded overlays could be used to restore pavement to its original (or better) condition. Chapter 2 of this guide discusses each category for preservation and rehabilitation, and the following outlines how concrete overlays fit into each category.

Preventive Maintenance

Preventive maintenance is a major component of pavement preservation. It consists of extending the service life of structurally sound pavements by applying cost-effective treatments to the surface or near the surface. Bonded concrete overlays of approximately 51–102 mm (2–4 in.) thickness provide preventive maintenance strategies for all types of pavements.

Minor Rehabilitation

Minor rehabilitation is used when structural capacity needs to be restored to a pavement but major rehabilitation is not required. One of the major advantages of concrete overlays as a preservation solution is that they increase the pavement's structural capacity, even if that is not the primary objective of the preservation activity. Bonded and unbonded concrete overlays of 102 mm (4 in.) provide excellent minor rehabilitation solutions.

Major Rehabilitation

For pavements needing structural improvement, rehabilitation is the approach typically used. Major rehabilitation calls for structural enhancements that extend the service life of an existing pavement and/or improve its load-carrying capability. Bonded concrete overlays from 127 to 178 mm (5 to 7 in.) are common, and unbonded overlays greater than 127 mm (5 in.) have been the norm.

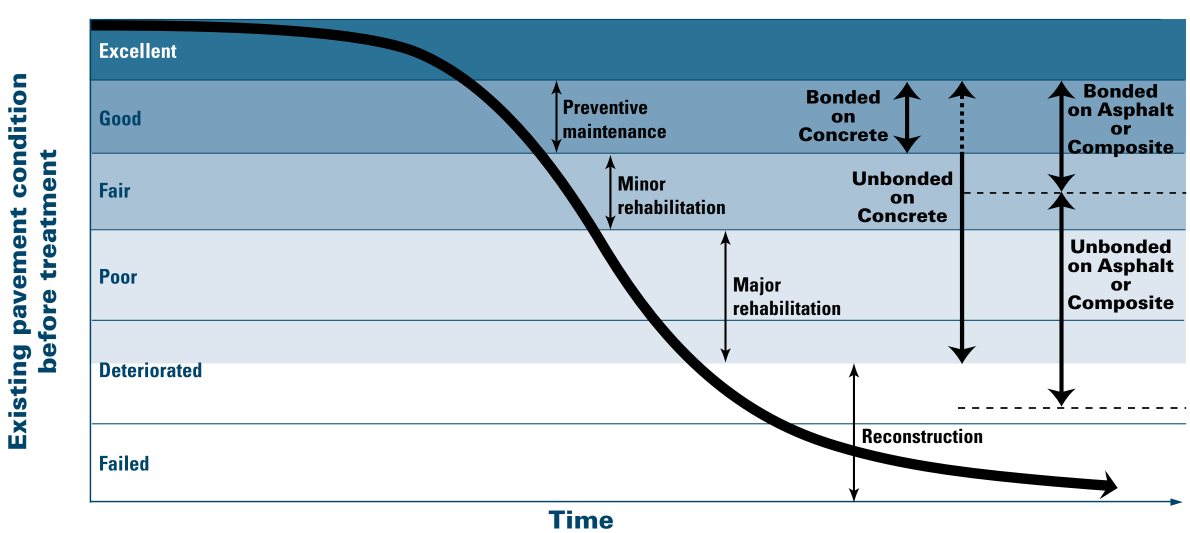


Figure 11.2. Timing application of bonded and unbonded concrete solutions (Harrington and Fick 2014)

3. Purpose and Project Selection

As previously mentioned, concrete resurfacings require uniform support conditions if satisfactory performance is to be realized. Nearly all the documented cases of premature overlay failure can be traced to some violation of this single requirement, often a result of "picking the wrong project" to resurface. For this reason, the evaluation of the existing pavement is paramount in determining if uniform support and movement control exists, or if it can be cost-effectively achieved. If so, the question then arises whether a bonded concrete overlay will act as a monolithic unit with the underlying pavement and provide the structural capacity, load transfer, and drainage system required to meet the design life, or if an unbonded overlay will be necessary to meet the same criteria but with the added burden of meeting critical elevation constraints.

The preservation fix represents the lowest possible cost (with possible small amounts of localized failures) within the design life. The preservation fix is typically in the bonded overlay category but can include thin unbonded overlays. For unbonded overlays, concrete resurfacing represents the placement of a new surface over an existing pavement that, in most cases, has significant deficiencies. Because of the existing pavement distresses, the unbonded overlay is placed over essentially nonengineered materials. To have a successful overlay, regardless if it is bonded or unbonded, the good and poor characteristics of the existing pavement must be understood along with the level of expected success for dollars expended.

Purpose of Bonded Overlay Systems

The purpose of bonded concrete overlays is to add structural capacity and eliminate surface distresses on existing pavements that are in good to fair structural condition. Bonded overlays generally provide resurfacing solutions for routine or preventive pavement maintenance and for minor rehabilitation, as shown in Figure 11.2.

Bonded concrete overlays are relatively thin structures (51–152 mm [2–6 in.]). Bonded together, the overlay and the existing pavement perform as one monolithic pavement. Bonding between the overlay and the existing pavement is essential. The bond ensures that the overlay and existing pavement perform as one structure, with the original pavement continuing to carry a significant portion of the load. All bonded overlay projects, therefore, are carefully designed and constructed to achieve and maintain a bond between the overlay and the existing pavement. Factors that affect the performance of the resurfaced pavement include the structural integrity of the underlying pavement, the effectiveness of the bond, the ability of the two layers to move monolithically to maintain the bond, and overlay jointing and curing techniques.

The key to achieving desired performance is to ensure the two structures—the existing pavement and the overlay—behave as one structure. Therefore, it is important to understand movement-related properties, such as expansion and contraction properties, of both the existing pavement and the overlay. For example, for a bonded concrete overlay of an existing concrete pavement, the CTE of the overlay concrete mixture should be similar to or less than that of the existing concrete pavement. Bonded overlay projects are more challenging to construct than unbonded overlay projects.

Purpose of Unbonded Overlay Systems

The purpose of an unbonded overlay is to restore structural capacity to an existing pavement that is moderately to significantly deteriorated. Unbonded overlays are minor to major rehabilitation strategies (as shown previously in Figure 11.2).

The term "unbonded" simply means that bonding between the overlay and the underlying pavement is not needed to achieve the desired performance (i.e., the thickness design procedure does not consider the existing pavement as a structural component of the surfacing layer). Thus the overlay performs as a new pavement, and the existing pavement provides a stable base. When the underlying pavement is asphalt or composite, partial or full bonding between the concrete overlay and the underlying asphalt layer should not cause a problem. In fact, such bonding generally adds some load-carrying capacity to the system. Therefore, unbonded concrete overlays on existing asphalt or composite pavements are not designed and constructed to prevent bonding between the layers. When the underlying pavement is concrete, however, unbonded concrete overlays are carefully designed and constructed to prevent bonding between the two concrete layers. That is because any bonding between the two concrete layers may stress the overlay and result in undesired reflective cracking.

Project Selection: Pavement Evaluation Process

The purpose of evaluating the existing pavement's condition is to collect details about any distresses and performance problems that currently exist and their causes. This information helps the owner-agency determine if a pavement is a good candidate for a concrete overlay and, if so, the extent of spot repairs required before an overlay is constructed. The extent of repairs needed is an important factor in determining whether a bonded or unbonded overlay will be a cost-effective solution.

Evaluating the existing pavement's condition involves at least four steps, as depicted in Figure 11.3 and described below:

1. The first step is to review the pavement's historical design and performance record, gathering information on layer thicknesses and other design attributes, mixture materials and design, construction date and method, traffic loadings, design life, maintenance activities to date, and so on. Along with looking at the historical records, this step should include recording future performance requirements, such as expected traffic loadings and desired overlay design life. This step should include a determination of any elevation limits and/or grade restrictions that signal potential clearance issues for overlay construction.

2. The second step is a visual examination of the pavement's condition, noting visible surface and structural distresses.

3. The third step is a more thorough examination of the pavement structure through a core analysis. This step will identify distresses or performance problems that cannot be determined by a visual examination alone. Core analyses verify the pavement thickness, the subgrade/base material and thickness, and the depth and perhaps type or cause of distresses.

4. The final step is based on information learned in Steps 1 through 3 to identify the need for any additional testing or evaluation. For example, tests related to materials or durability distresses, possible support problems, or surface conditions may be necessary. The following questions can help the pavement owner determine if additional tests are advisable:

   - What is the extent of pavement distresses, based on the visual evaluation and core analysis?
   - What is the pavement's expected service level and life? Major highways with significantly high truck volumes and/or long service life require more extensive and comprehensive evaluations than lower-volume roadways.

Results from all initial evaluation steps should be recorded in a pavement evaluation report that documents the critical information pertaining to the existing pavement structure.

Initial Evaluation (Steps 1–4)

1 Pavement History and Performance Goals

- Pavement material (including aggregate CTE), design, age, thickness, layers.
- Existing traffic and performance level.
- Design life.
- Remaining life.
- Desired traffic and performance level.
- Desired design life.
- Elevations and grade restrictions.
- Other historical information.

2 Visual Examination

Concrete | Asphalt / Composite

3 Core Analysis

- Type of distress.
- Depth of distress.
- Verification of thickness for pavement base/subbase.

4 Optional Analyses

(depending on extent of problems)

4-a. Material-related Tests

(indicated by core analysis)

Conduct if (a) material or durability issues are indicated, or (b) roadway provides service for high levels of traffic, especially if a bonded overlay is being considered.

- Petrography analysis.
  - ConcreteMRD.
  - Poor air-void system.
- Asphalt stripping.
- CTE.

Conduct if (a) pavement or subgrade support issues are indicated, or (b) roadway provides service for high levels of traffic, especially if a bonded overlay is being considered.

- FWD tests.
  - Subgrade/subbase support (*k* value).
  - Subgrade/subbase variability.
  - Pavement properties.
  - Load transfer efficiency.
  - Presence of voids.
  - Asphalt stiffness.
  - Concrete flexural strength.
- Subgrade tests.
  - Freeze-thaw characteristics.
  - Shrink-swell characteristics.
  - Soil strength (dynamic cone penetration or standard penetration test).

4-c. Surface Texture Tests

Conduct if (a) materials or durability issues are indicated, or (b) roadway provides service for high levels of traffic, especially if a bonded overlay is being considered.

Figure 11.3. Pavement evaluation process (Harrington and Fick 2014)

4. Limitations and Effectiveness

The decision to select a bonded or unbonded concrete overlay depends on several factors, as listed below and expanded upon in Table 11.1:

- The owner's purpose in treating the pavement.
- Restrictions of the project, i.e. vertical restrictions, shoulders, curb and gutter, bridge, clearances, fill slopes, guardrail, and ADA sidewalk criteria.
- The condition of the existing pavement.
- The kind of improvements desired.

Bonded concrete overlays are generally not good solutions in any of the following situations:

- The underlying concrete pavement has widespread materials-related issues such as ASR or D-cracking, subgrade support is inadequate or nonuniform, or drainage is poor.
- The underlying asphalt pavement has significant structural deterioration, inadequate base or subgrade support, or poor drainage conditions. In these cases, however, unbonded overlays may be considered.

When an unbonded overlay is being considered, the condition of the entire depth of the existing pavement should be evaluated. For example, on an existing asphalt pavement, is the deterioration such that the upper portion needs removal and replacement (partial mill and fill) or does the full depth require replacement (full mill and fill)? These questions are important to consider not only to ensure that the design life is achieved but also that grade and elevations are met.

Figures 11.4 and 11.5 provide a summary of existing pavement conditions, applications, keys to success, and other issues for bonded and unbonded overlays, respectively.

Table 11.1. Bonded vs. Unbonded Overlay Considerations (based on Harrington and Fick 2014).

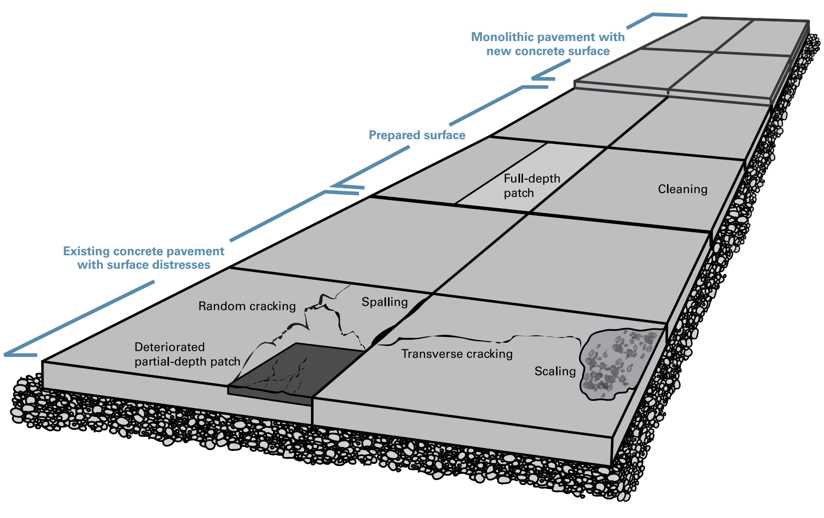
	Concrete Overlay	
	Bonded	**Unbonded**
Purpose	Primarily a preventive maintenance or minor rehabilitation strategy to improve surface characteristics and/or load-carrying capacity	Primarily a major rehabilitation strategy
Condition of Existing Pavement	Pavements in good to fair structural condition or made into that condition	The underlying pavement can be poor to deteriorated but must be, along with the base and/or subgrade, stable and uniform
Resulting Improvements to the Pavement	• Long-term wearing surface added. • Surface defects eliminated. • Surface characteristics like smoothness, friction, and/or noise improved. • Load-carrying capacity added. • Pavement life extended.	• Load-carrying capacity restored and increased. • Pavement life extended. • Surface defects eliminated. • Surface characteristics like smoothness, friction, and/or noise improved.

Bonded Family

Thickness: 2 in.–5 in. depending on desired life (15–25 years), anticipated traffic loading, and condition of underlying pavement

Bonded Concrete Overlays of Concrete Pavements

(Overlay and existing concrete pavement act as one monolithic pavement)



Existing pavement condition

Good structural condition; limited surface distress

Applications

- To eliminate surface distresses.
- To improve friction, noise, and rideability.
- Where increase in traffic loads requires more structural capacity.
- Where vertical clearances must be met.

Keys to success

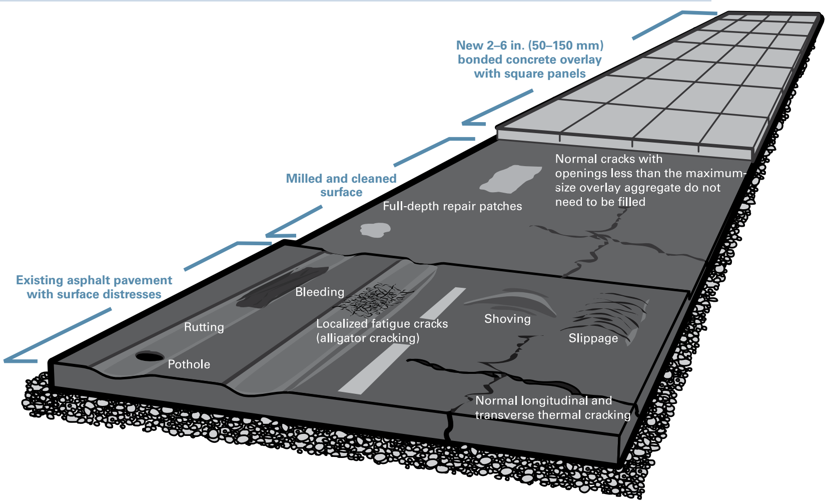
- Overlay aggregate's thermal properties must be similar to existing pavement's.
- Existing joints must be in fair condition or repaired.
- Overlay must establish good bond with existing pavement.
- Thinner pavements may accelerate sawing window.
- Curing must be timely and thorough, especially at edges.
- Joints must align with those of existing pavement.
- Transverse joints: full depth plus 0.5 in. (1.3 cm).
- Longitudinal joints: at least T/2.

Other issues

Working cracks will reflect through unless repaired or the overlay is sawed over the crack.

Bonded Concrete Overlays of Asphalt Pavements

(Overlay and existing asphalt pavement act as one monolithic pavement)



Existing pavement condition

Fair or better structural condition with surface distress

Applications

- To eliminate surface distresses such as rutting and shoving.
- To improve friction, noise, rideability, and surface albedo.
- Where increase in traffic loads requires more structural capacity.
- Where vertical clearances must be met.

Keys to success

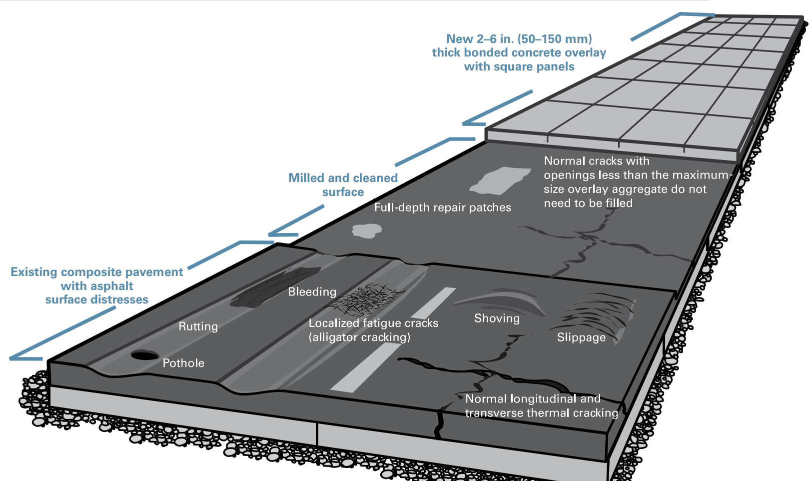
- Thin milling may be required to eliminate surface distortions of 2 in. (5.1 cm) or more and help provide good bond.
- Keep joints out of wheel paths.
- Thinner pavements may accelerate sawing window.
- Saw joints in small, square panels.
- Have enough saws on site to keep up with curing.
- Curing must be timely and thorough.

Other issues

Maintain surface temperature of asphalt below 120°F (48.9°C).

Bonded Concrete Overlays of Composite Pavements

(Overlay and existing pavement act as one monolithic pavement)



Existing pavement condition

Fair or better structural condition with severe surface distress

Applications

- To eliminate surface distresses such as rutting and shoving.
- To improve surface friction, noise, rideability, and surface albedo.
- Where increase in traffic loads requires more structural capacity.
- Where vertical clearances must be met.

Keys to success

- Thin milling may be required to eliminate surface distortions of 2 in. (5.1 cm) or more and help provide a good bond.
- Keep joints out of wheel paths.
- Thinner pavements may accelerate sawing window.
- Saw joints in small, square panels.
- Curing must be timely and thorough.
- Have enough saws on site to keep up with curing.

Other issues

- Maintain surface temperature of asphalt below 120°F (48.9°C).
- Examine profile for vertical distortion at joints that could signal movement in the bottom layer from drainage or material-related distress of underlying pavement.

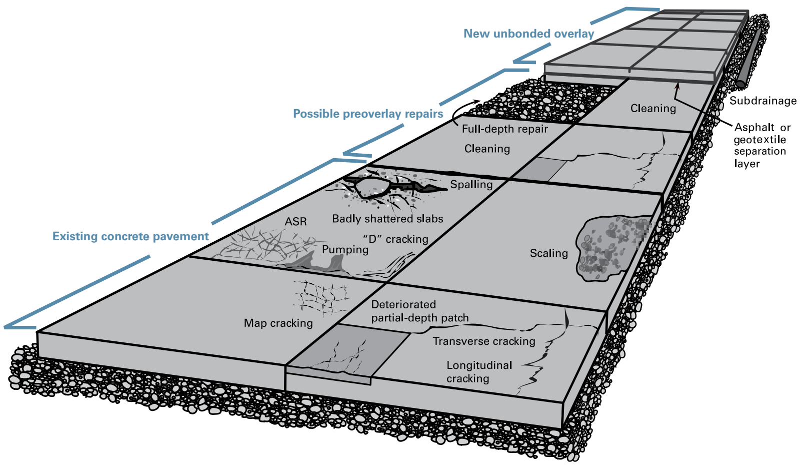
Figure 11.4. Summary of bonded concrete overlays (adapted from Harrington and Fick 2014)

Unbonded Family

Thickness: 4 in.–11 in. depending on desired life (20–30 years), anticipated traffic loading, and condition of underlying pavement

Unbonded Concrete Overlays of Concrete Pavements

(Results in a new pavement on a stable base)



Existing pavement condition

Poor condition but stable and uniform

Applications

- To restore or enhance pavement's structural capacity.
- To increase pavement life equivalent to new full-depth pavement.
- To improve surface friction, noise, and rideability.
- To reduce underlying pavement temperatures, decreasing the potential for existing pavement expansion and buckling.

Keys to success

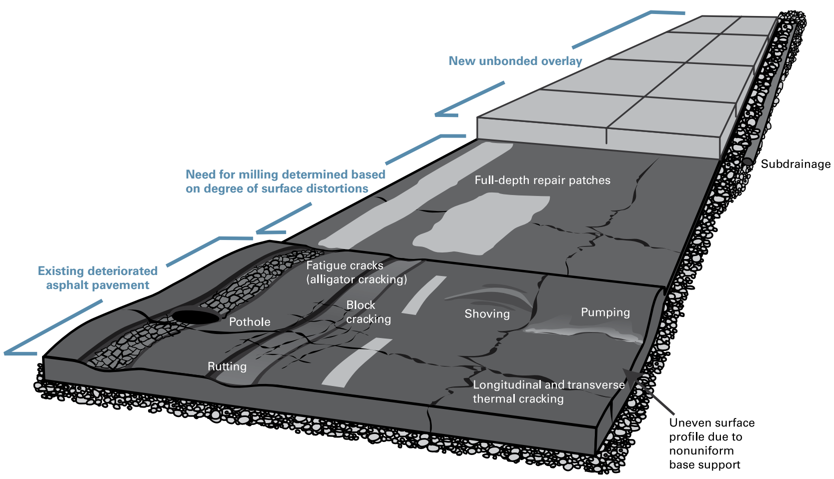
- Full-depth repairs to restore structural integrity in isolated spots may be necessary.
- Use of asphalt or fabric separation layer to minimize reflective cracking.
- Faulting less than or equal to .375 in. (0.9 cm) is generally not a concern when separation layer is 1 in. (2.5 cm) or more.
- Saw joints as soon as possible because the sawing window can be short.
- Use shorter joint spacing than normal full depth pavements to help reduce curling and warping stress.

Other issues

High truck traffic on the asphalt separation layer, in the presence of water, can strip the asphalt; provide adequate drainage or use a more porous asphalt mix to reduce pore pressure.

Unbonded Concrete Overlays of Asphalt Pavements

(Results in a new pavement on a stable base)



Existing pavement condition

May be deteriorated but stable and uniform

Applications

- To restore or enhance pavement's structural capacity.
- To increase pavement life equivalent to a new full-depth pavement.
- To eliminate rutting and shoving problems.
- To improve surface friction, noise, rideability, and surface albedo.

Keys to success

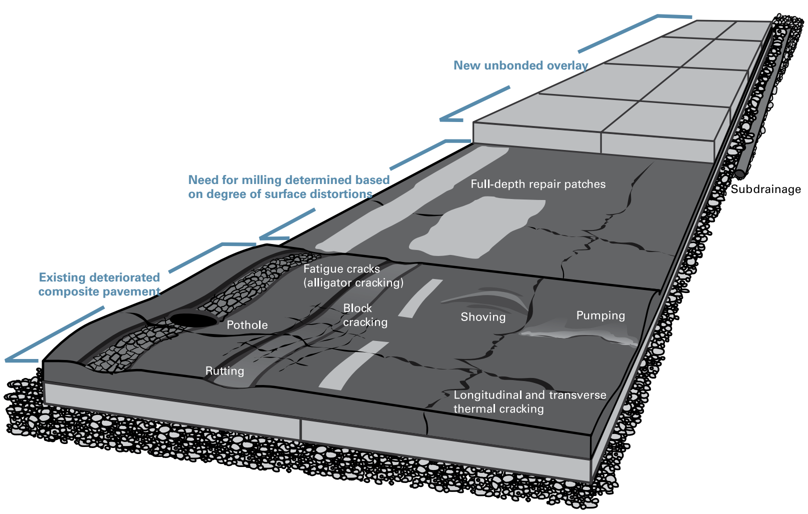
- Consider milling when surface distortions are 2 in. (5.1 cm) or greater.
- Repair isolated areas where structural integrity is lost.
- Timing of joint sawing.

Other issues

Maintain surface temperature of asphalt below 120°F (48.9°C).

Unbonded Concrete Overlays of Composite Pavements

(Results in a new pavement on a stable base)



Existing pavement condition

May be deteriorated but stable and uniform

Applications

- To restore or enhance pavement's structural capacity.
- To increase pavement life equivalent to new full-depth pavement.
- To eliminate rutting and shoving problems.
- To improve surface friction, noise, rideability, and surface albedo.

Keys to success

- Timing of joint sawing.
- Consider milling when surface distortions are 2 in. (5.1 cm) or greater.
- Repair isolated areas where structural integrity is lost.
- Other issues.
- Vertical distortion at joints of composite pavement must be repaired before overlay.
- Maintain surface temperature of asphalt below 120°F (48.9°C).

Figure 11.5. Summary of unbonded concrete overlays (adapted from Harrington and Fick 2014)

5. Concrete Overlay Design

Regardless of the overlay system and design procedure used, the analysis begins with recognition of a number of common inputs to the design process. It is important to first define the scope of the intended project and its intended structural performance requirements. Expected design life will affect both the extent of repairs required on the existing pavement and the design inputs. This in turn influences the thickness, the amount of repair, and thus the cost of the overlay; see Figure 11.6. The engineer is also required to characterize and understand the existing pavement structure, the anticipated traffic loading, and the materials expected to be used. In most cases, climatic influences play a role, particularly with a bonded concrete overlay system.

Figure 11.6. Interrelated overlay design factors (Harrington and Fick 2014)

Design Selection

In most cases, the designer will have an idea of the likely feasible alternatives based on the initial survey of the project. In selecting the final design, however, it is important for the engineer to anticipate the condition of the existing section at the time of actual construction of the new concrete surface. In many cases, construction will not begin for at least 2–3 years. Some degradation of the existing structure should be anticipated and considered in the analysis.

Thickness Design for Concrete Overlays

There are several design procedures available for determining the appropriate thickness of bonded and unbonded concrete overlays. Designers should consult the *Guide to the Design of Concrete Overlays Using Existing Methodologies* (Torres et al. 2012) for comprehensive guidance regarding overlay thickness design. That document provides information and guidance on the use of the following design procedures:

- ACPA procedure for bonded concrete overlays on asphalt pavements (ACPA 2014a).
- AASHTO procedures for bonded overlays of concrete pavements and unbonded overlays of both existing concrete and asphalt pavements (AASHTO 1993).
- AASHTOWare Pavement ME Design procedures for bonded and unbonded overlays of both existing concrete and asphalt pavements (AASHTO 2008).

Table 11.2 provides a summary of the current design procedures, typical input values, and other pertinent information. Two of the most important aspects in concrete overlay design are (1) how each method handles the bond between the existing pavement and the concrete overlay, and (2) whether the method assumes the existing pavement will provide significant structural capacity or, alternatively, contribute to the quality of the pavement foundation. With this type of information, pavement designers are able to make an informed decision about which method to apply when designing a certain type of concrete overlay.

Table 11.2. Summary of Available Concrete Overlay Design Procedures (Torres et al. 2012)

	Overlay Type	Typical Design and Software Parameters							
		Traffic (Millions of ESALs)	Typical Concrete Slab Thickness	Maximum Joint Spacing (ft)	Range of Condition of Existing Pavement	Macro-fibers Option (in software)	Transverse Joint Dowel Bars	*Mainline Longitudinal Tiebars	Recommended Design Procedure
Bonded	**Bonded Concrete Overlay of Asphalt Pavement**	Up to 15	3–6 in.	1.5 times thickness (inches)	Fair to Good	Yes	No	No	ACPA 2014a; Vandenbossche 2014
Bonded	**Bonded Concrete Overlay of Concrete Pavement**	Up to 15	3–6 in.	Match existing cracks and joints and cut intermediate joints	Fair to Good	Yes	No	No	AASHTO 1993; AASHTO 2008; ACPA 2014b
Bonded	**Bonded Concrete Overlay of Composite Pavement**	Up to 15	3–6 in.	1.5 times thickness (inches)	Fair to Good	Yes	No	No	ACPA 2014a; Vandenbossche 2014
Bonded	**Thin Fibrous Overlays of Asphalt Pavements**	Up to 15	2–3 in.	4–6 ft	Fair to Good	Yes	No	No	Bordelon 2011
Unbonded	**Unbonded Concrete Overlay of Asphalt Pavement**	Up to 100	4–11 in.	Slab < 6 in.: use 1.5 times thickness (inches) Slab ≥ 6 in.: use 2.0 times thickness (inches) Slab > 7 in.: use15 ft	Deteriorated to Fair	Yes	For slabs > 7 in.	T ≥ 6 in.: use Agency standards	AASHTO 1993; AASHTO 2008; ACPA 2014b
Unbonded	**Unbonded Concrete Overlay of Concrete Pavement**	Up to 100	4–11 in.	Slab < 5 in.: use 6 ft x 6 ft panels Slab 5–7 in.: use 2.0 times thickness (inches) Slab > 7 in.: use15 ft	Deteriorated to Fair	Yes	For slabs > 7 in.	T ≥ 6 in.: use agency standards	AASHTO 1993; AASHTO 2008; ACPA 2014b
Unbonded	**Unbonded Concrete Overlay of Composite Pavement**	Up to 100	4–11 in.	Slab < 6 in.: use 1.5 times thickness (inches) Slab ≥ 6 in.: use 2.0 times thickness (inches) Slab > 7 in.: use 15 ft	Deteriorated to Fair	Yes	For slabs > 7 in.	T ≥ 6 in.: use agency standards	AASHTO 1993; AASHTO 2008; ACPA 2014b
Unbonded	**Unbonded Short-jointed Concrete Slabs**	Up to 100	> 3 in.	4–8 ft	Poor to Fair	Yes	For slabs > 7 in.	For ≥ 3.5 in.: slabs at tied concrete shoulders or for T ≥ 6 in.: use agency standards	TCPavements 2014

Note: See 2014 edition of the Guide for Concrete Overlays (Harrington and Fick 2014) for the links to the above software programs.

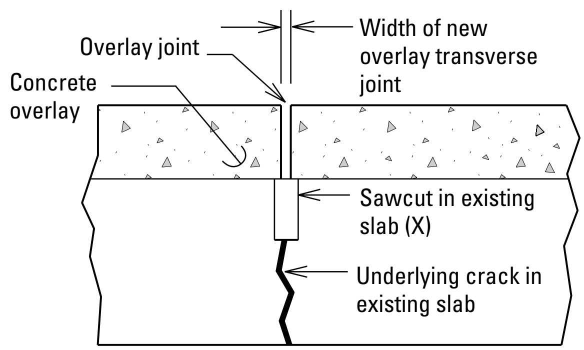
Bonded Overlay Designs

Joints in bonded concrete overlays must match those in the existing pavement. Transverse joints should be cut full depth of the overlay plus 13 mm (0.50 in.) and must be as wide as or greater than the crack below the joint in the underlying pavement; see Figure 11.7. Longitudinal joints should be cut at least T/2. Overlay joint sawcut width shall be greater than the actual crack width in the existing pavement (note this is different from the existing joint reservoir width).

Bonded overlays of existing CRCP are also possible. In this case, however, there is no need to match the transverse joints because none exist (with the exception of terminal joints and FDRs).

The use of steel reinforcement or dowels is not usually a consideration for bonded overlays on concrete pavements unless the overlay is thicker than usual, new shoulders are being tied, or there is also a desire to retrofit load transfer.

Properly built, bonded overlays can reasonably be expected to provide a minimum service life of 15 years before maintenance is required. The first indication of early failure problems on these overlay projects is usually delaminating at the bond plane, quickly followed by classic fatigue failure at isolated joint locations. These can be repaired using repair techniques if the underlying slab remains sound. If the overlay is truly bonded, then over the long term other distresses typical of monolithic slabs, such as transverse slab cracking and corner breaks, will predominate.



Note: Overlay joint width shall be equal to or greater than crack in the existing slab.

If "X" is 0.50 in. (13 mm) or greater, the underlying crack width in the existing slab should be measured. If crack is 0.25 in. (6.4 mm) or greater, and existing pavement does not have dowel bars, the joints should be evaluated to determine if load transfer rehabilitation is required to eliminate faulting. If there are numerous joints of this type, the existing pavement may not be a good candidate for a bonded overlay.

Figure 11.7. Joint sawcut details for bonded concrete overlays (Harrington and Fick 2014)

Unbonded Overlay Designs

Unbonded overlay designs usually do not consider bond, although in fact some bonding with existing asphalt pavement usually occurs and is beneficial. They are essentially designed as new concrete pavements, with the pavement being overlaid acting as a base. Adaptation of existing design procedures is relatively straightforward and construction relatively easy. Unbonded overlays are usually designed to serve 20–30 years.

The selection of the load transfer coefficient in the AASHTO procedure should be made with recognition of the character of the underlying layer in addition to the intended load transfer system for the overlay. Consideration should be given to the underlying structure providing additional load transfer, which is not necessarily true of new concrete pavements. The designer should not arbitrarily pick a "conservative" value, as this is not the intent of the design procedure. The ACPA's WinPAS software program (based on AASHTO 1993) includes an entire section for use in designing these types of systems.

Interlayer

All unbonded concrete overlays on concrete must be separated from the existing concrete pavement by a stress-relief layer, or interlayer, to prevent reflective cracking from movement of the existing pavement. Interlayers serve several different purposes:

- Provide a shear plane that relieves stress and helps prevent cracks from reflecting up from the existing pavement into the new overlay.
- Prevent bonding of the new pavement with the existing pavement, so both are free to move independently.

- Provide a channel for the removal of infiltrating water along the cross slope to the pavement edge.
- Provide a cushion for the overlay to prevent keying from existing faulting.

The design should consider the relative importance of each purpose based on project-specific conditions and the condition of the existing pavement.

Asphalt Interlayer

The most common stress relief interlayer is a thin layer of HMA material typically open graded with an adequate drainage outlet. Thickness is not critical, but 25 mm (1 in.) is usually adequate to eliminate potential problems with "keying" caused by faulted slabs or localized repairs; see Figure 11.8. When constructing CRCP unbonded overlays over both CRCP and plain jointed pavements, Texas has sometimes increased the asphalt interlayer thickness to greater than 25 mm (1 in.). It is important not to use the asphalt interlayer as a leveling course. All grade corrections, including leveling, should be accomplished with the concrete overlay itself.

Occasional problems have been noted with asphalt stripping within the interlayer under repetitive loading, causing a loss of support for the unbonded overlay; see Figure 11.9. This can occur occasionally with high truck traffic volumes in the presence of water in the interlayer. Usually, the stripping takes several years to develop. The best preventive solutions are the following:

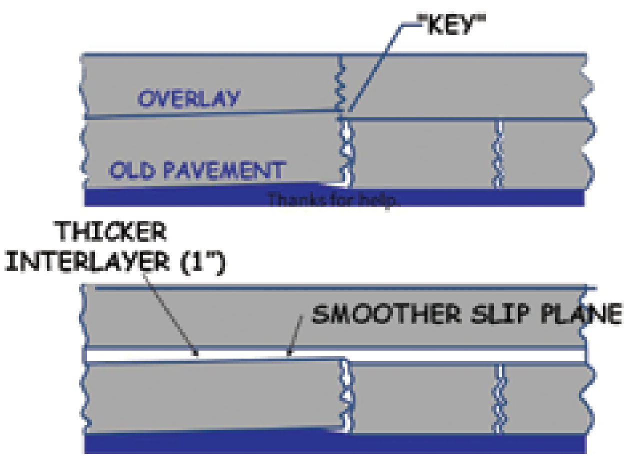


Figure 11.8. Prevention of keying of overlay (upper figure) through the placement of an effective interlayer (lower figure) (Harrington and Fick 2014)

Figure 11.9. Stripping of asphalt interlayer (photo courtesy of Dan DeGraaf, Michigan Concrete Association)

- Provide positive drainage for the asphalt layer. Some highway agencies use a normal asphalt mix with varied results. For example, the Michigan DOT has success draining their asphalt interlayer by using a more porous mixed base of certain aggregate gradation (Harrington and Fick 2014). The asphalt interlayer should be daylighted to the edge of the shoulders or into a subdrainage system. Note that if subdrains are to be installed to drain the interlayer, the soil needs to meet retrofitted subdrainage criteria, as described in Chapter 7.
- Incorporate antistrip lime additives in the asphalt. Liquid antistripping additives were found to be not as effective.
- Seal joints in the concrete overlay and at the shoulders.
- Utilize a geotextile interlayer with positive drainage.

Nonwoven Geotextile Interlayer

For an unbonded overlay, a nonwoven geotextile that meets certain transmissivity requirements has been successfully used for the last 5 years in the United States. Germany has more than 30 years of experience with geotextile interlayers. Nonwoven geotextile interlayers promote drainage (wicks water) but must also have a proper drainage outlet.

Transitions

A concrete overlay design often requires transition details that link the concrete overlay with the existing pavement structure adjacent to the overlay project. Since these locations are often subject to additional stress, a variety of alternatives has been used, including thicker concrete sections, conventional reinforcement or wire mesh, and structural macrofibers. Transitions must be designed and constructed to connect the new overlay pavement with existing pavement, existing structures, and driveways. Figures 11.10 through 11.14 provide details for various transitions used for overlay construction.

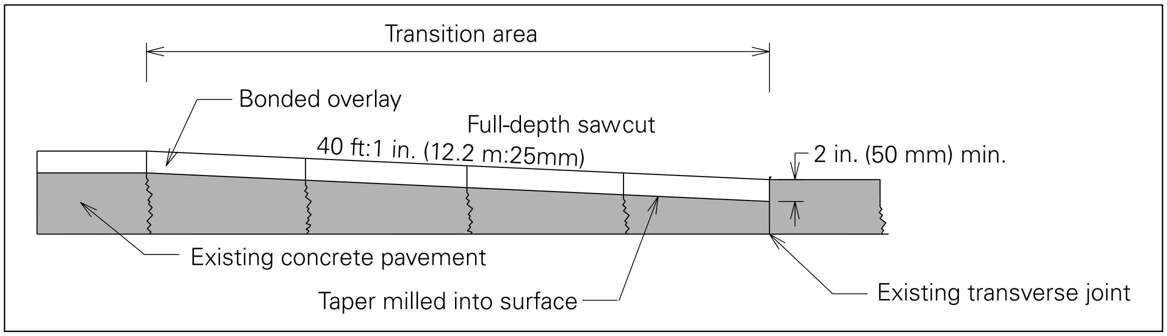


Figure 11.10. Mill and fill transition for bonded concrete overlay of concrete pavement (adapted from ACPA [1998])

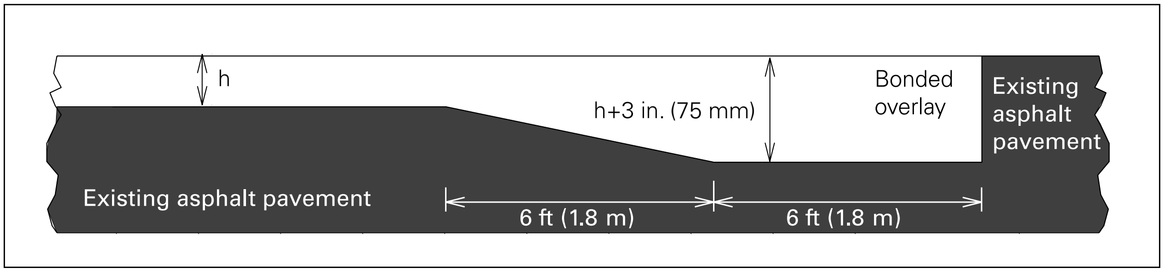


Figure 11.11. Mill and fill transition for bonded concrete overlay of asphalt or composite pavement (adapted from ACPA [1998])

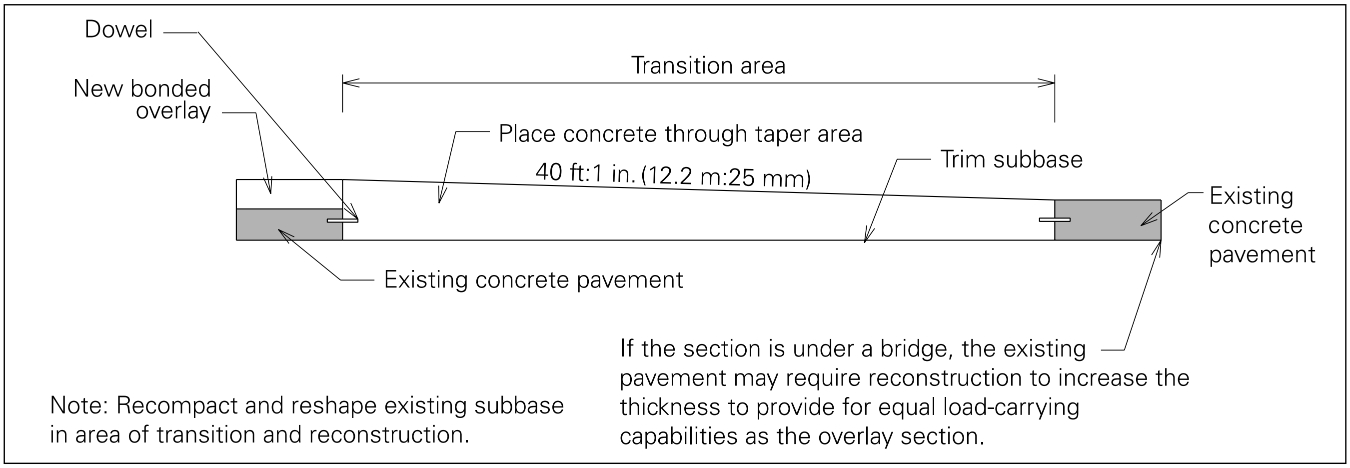


Figure 11.12. New transition tapers used to meet bridge approach slabs or maintain clearance under bridges with bonded overlay of concrete pavement (adapted from ACPA [1990])

Ch 11. Concrete Overlays

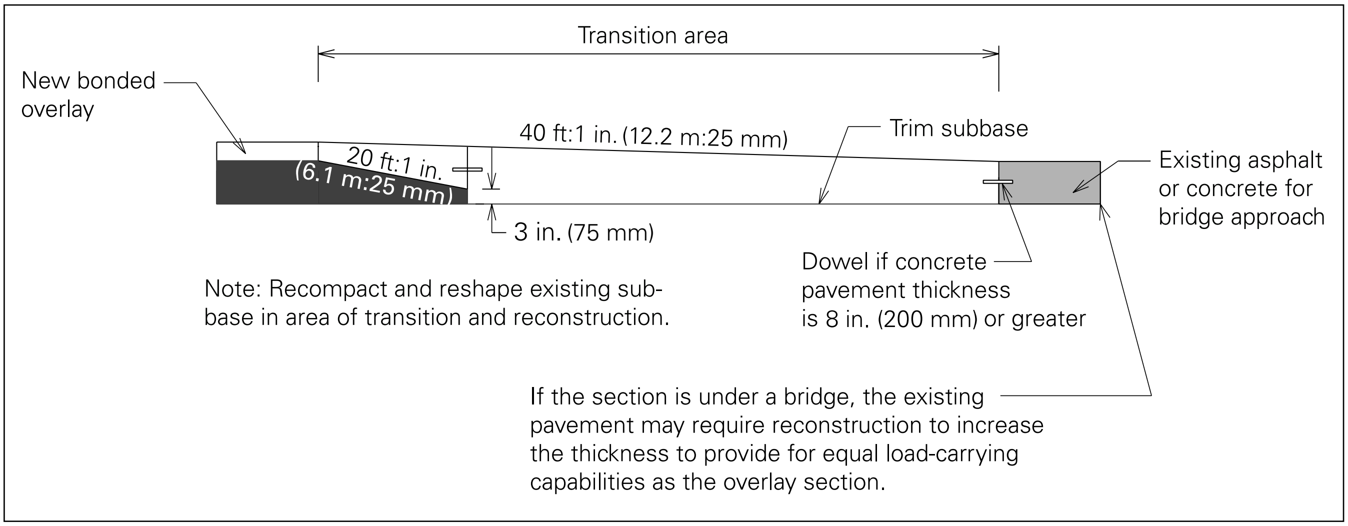


Figure 11.13. New transition tapers used to meet bridge approach slabs or maintain clearance under bridges with bonded overlay of asphalt pavement (adapted from ACPA [1991])

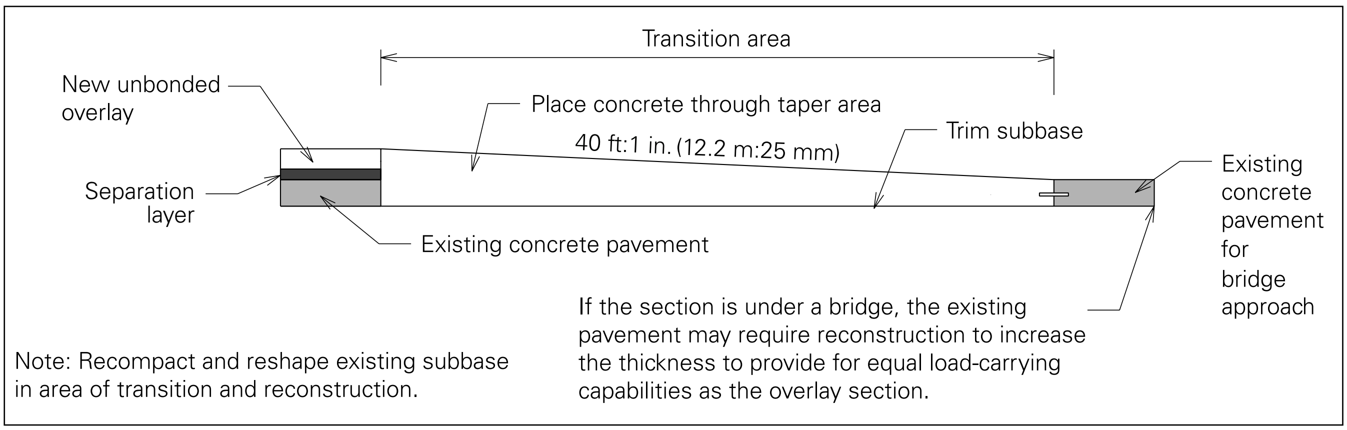


Figure 11.14. New transition tapers used to meet bridge approach slabs or maintain clearance under bridges with unbonded overlay of concrete pavement (adapted from ACPA [1990])

6. Materials

The majority of concrete overlays is designed and constructed using standard materials, but some may use accelerated materials. Accelerated mixtures are designed for a faster rate of strength gain, thereby allowing earlier opening of the concrete overlay to construction and/or public traffic. Fibers may be incorporated into the concrete materials in some cases to provide additional strength or to address potential plastic shrinkage issues. Other materials used in concrete overlays include dowel bars, tiebars, curing compound, joint sealants, and interlayers. This section describes important concrete material properties, introduces the use of fibers in concrete overlays, and presents key interlayer characteristics.

Mixtures for Concrete Overlays

Each of the components used in a concrete mixture should be carefully selected so that the resulting mixture is dense, relatively impermeable, and resistant to both environmental effects and deleterious chemical reactions over the length of its service life. Type I and Type II cements are commonly used in concrete mixtures for concrete overlays. When high early strength is desired, higher amounts of Type I cements can be used to promote the development of high early strength. Since conventional Types I and II cements are normally adequate, the use of Type III cements with overlays is not recommended because of increased thermal shrinkage and potential for thermal shock. As with conventional paving, supplementary cementitious materials (SCMs) normally improve durability and can facilitate construction.

Reduced Concrete Permeability

The permeability of a concrete mixture determines how easily moisture can infiltrate the paste structure of the concrete. A lower permeability is desirable to slow the rate at which concrete will become saturated. Recent work led by the South Dakota DOT includes recommendations to achieve durable, dense, and impermeable concretes that withstand the deleterious effects of deicing chemicals and to prevent or reduce joint deterioration caused by water saturation at the joints (Sutter et al. 2008). Recommendations include the following:

- Low water-cementitious materials (w/cm) ratio.
- Appropriate use of SCMs.
- Well-graded aggregates.
- Adequate air void system.

The target permeability at 56 days should be less than 1,500 coulombs when tested in accordance with the rapid chloride permeability test (ASTM C 1202) or 25 kΩ-cm when tested using surface resistivity measured in accordance with AASHTO TP 95.

The permeability of a concrete mixture is primarily governed by the amount of water in the concrete at the time of mixing. Permeability will decrease as less water is used. The w/cm ratio should not exceed 0.45; ideally, the w/cm ratio should be between 0.38 and 0.42 (especially for wet freeze-thaw environments).

There are a number of ways to achieve uniformly lower w/cm ratios while retaining satisfactory workability, including combinations of the following:

- Using SCMs in appropriate dosages.
- Using water-reducing admixtures.
- Using aggregate systems with combined gradation, which promotes reduced paste volume and improved workability.
- Controlling concrete temperature.
- Not adding water to a ready-mix truck at the point of delivery.

Appropriate Use of SCMs

Replacement of some concrete with SCMs in well-cured concrete has multiple benefits ranging from improved workability to reduced permeability of the hardened concrete. Typical replacement rates with SCMs are 15 percent to 35 percent depending on the chemistry of the system. Supplementary cementitious materials commonly used in concrete mixtures for pavements include Class C fly ash, Class F fly ash, and slag cement.

Well-Graded Aggregates

As summarized in Table 11.3, the use of well-graded aggregates helps to improve permeability in several ways. First, mixtures made with well-graded systems tend to be more workable, which in turn means that less water is required to achieve the same workability, allowing the use of a lower w/cm ratio. Second, well-graded systems allow the use of greater aggregate quantities and lower paste contents. Because paste is more permeable than aggregate, reducing the paste content while maintaining workability will lead to reduced permeability. Third, better workability will lead to better consolidation of the mixture, also improving (reducing) permeability and reducing the risk of overvibration and the attendant problems (Taylor et al. 2011). Finally, the improved workability of well-graded concrete mixtures allows for more efficient placement, especially in handwork, which means that the pavement can be finished earlier while the concrete mixture is still fresh.

The maximum coarse aggregate size used in concrete mixtures for overlays is a function of the overlay thickness. Some thinner concrete overlays may require a reduction in size of the standard aggregate used in concrete paving. It is recommended that the largest practical maximum coarse aggregate size be used in order to minimize paste requirements, reduce shrinkage, minimize costs, and improve mechanical interlock properties at joints and cracks.

Table 11.3. Summary of Effects of Combined Gradations on Concrete Mixtures

Well-Graded Aggregates
Concrete mixtures produced with well-graded, dense aggregate matrix tend to • Reduce the water demand. • Reduce the cementitious material demand. • Reduce the shrinkage potential. • Improve workability. • Require minimal finishing. • Consolidate without segregation. • Enhance strength and long-term performance.
Gap-Graded Aggregates
Concrete mixtures produced with a gap-graded aggregate combination may • Segregate easily. • Contain higher amounts of fines. • Require more water. • Require more cementitious material to meet strength requirements. • Increase susceptibility to shrinkage. • Limit long-term performance.

Although maximum coarse aggregate sizes of 19–25 mm (0.75–1 in.) have been common in the last two decades, smaller maximum coarse aggregate sizes may be required for some concrete resurfacing projects. Smaller size aggregate can also help mitigate D-cracking, if the rock is known to be susceptible to that distress. For nonreinforced pavement structures, a maximum aggregate size of one-third of the slab thickness is recommended.

When selecting aggregate for a bonded concrete overlay on an existing concrete pavement, the CTE becomes a particularly important parameter. Because aggregate composes a majority of the concrete's mass, its CTE is a good indicator of concrete movement due to thermal expansion and contraction. Using an aggregate in the overlay mixture with a CTE similar to that of the existing pavement helps ensure that the two layers move together, thus minimizing stress at the bond line due to differential movement and helping to maintain the bond between the layers. The CTE can be determined using AASHTO provisional test TP-60 (*Coefficient of Thermal Expansion of Hydraulic Cement Concrete)*.

If not similar to the CTE of the underlying concrete pavement, the CTE of the overlay should be less than that of the underlying pavement. This is because the overlay surface is exposed to greater temperature swings than the underlying pavement. Therefore, the lower the overlay's CTE, the less the differential movement between the overlay and underlying pavement.

Adequate Air-Void System

Freeze-thaw durability is primarily affected by the environment (wet freezing conditions) and the air void system of the concrete. An air void system consisting of many small, closely spaced voids is a common means of providing protection against freeze-thaw damage. An adequate air void system in the as-placed concrete is vital. Air void systems can be affected by varying the composition of concrete constituents, placing techniques, and finishing activities. For concrete that is exposed to deicing chemicals or high water saturation (which is considered "severe exposure"), a spacing factor equal to or below 0.2 mm (0.008 in.) is recommended, along with a specific surface area of air voids equal to or greater than 24 mm²/mm (600 in.²/in.). The PCA recommends a minimum of 5 percent to 8 percent air content in the in-place concrete to prevent damage (Kosmatka and Wilson 2011). Recent laboratory work (Peterson and Sutter 2011; Ley, Felice, and Freeman 2012) indicates that these values are still appropriate.

Accelerated Mixtures

Although the use of accelerated mixtures and expedited paving practices has become more common in concrete overlay projects, there has been some concern regarding potential detrimental effects of faster-setting concrete mixtures and reduced construction times on the long-term durability of concrete due to excessive shrinkage, heat generation, and poor microstructure. Some agencies use rapid-strength concrete mixtures with a higher cementitious material content (Type I/II), low w/cm ratio, and smaller top size aggregate (typically 19 mm [0.75 in.]).

These mixtures can be used with accelerating admixtures to provide the early strength required to place traffic on the overlay within a short time period. A water-reducing admixture is used to reduce the w/cm ratio and provide the desired workability properties. Accelerated mixtures typically have a higher potential for shrinkage and warping compared to a conventional mixture due to their higher paste content. Increased shrinkage and warping stresses at early ages can cause early cracking and be detrimental to bond development that will result in premature overlay failures.

Fiber-Reinforced Concrete

In general, the use of fiber reinforcement is normally not needed for most concrete overlays. In certain situations, however—for example, where vertical restrictions limit the overlay thickness, where heavier-weight traffic loads are expected, where increased joint spacing is desirable, or where conventional dowels cannot be used—the use of fibers may be warranted.

During the last two decades, there has been resurgence in the use of fiber reinforcement in concrete. The reason for this is when properly used, new fiber reinforcement technology can contribute to the performance of thin concrete overlays. Whether or not the use of macrofiber in concrete overlays is warranted should be determined based on the existing pavement base thickness and condition, the owner's desired finish, the engineer's expected design life, overlay thickness, and cost. At the appropriate dosages, fibers can perform the following functions in a concrete mixture:

- Help increase concrete toughness (allowing thinner concrete slabs and/or longer joint spacings).
- Help control differential slab movement caused by curling/warping, heavy loads, temperatures, etc. (allowing longer joint spacing).
- Increase concrete's resistance to plastic shrinkage cracking (enhancing aesthetics and concrete performance).
- Hold cracks tightly together (enhancing aesthetics and concrete performance).

Although steel fibers have a long history of use in paving applications, the last two decades have seen synthetic fibers—see Figure 11.15—become more predominant because of their ease of handling, better dispersion characteristics (i.e., less "balling"), and resistance to rust damage. Types of synthetic fiber materials include the following:

- Polypropylene.
    - Monofilament (cylindrical)—fibers of same length.
    - Multifilament—monofilament fibers of different lengths.
    - Fibrillated (rectangular)—net-shaped fiber collated in interconnected clips.
- Polyester.
- Nylon.

Figure 11.15. Synthetic fibers (Harrington and Fick 2014)

For current design technology, the dosage of fiber—whether synthetic, steel, or some blend—is specified to produce certain behavior characteristics in the hardened concrete. These characteristics correlate with forecasts of increased performance such as flexural strength, and hence fatigue capacity is enhanced. It should be noted that the actual strength of the concrete given the current technology increases only slightly, if at all. Concrete will still crack if the load exceeds that which can be borne mechanically at its upper strength limit given the geometric properties of the section, but it will carry a much greater number of lesser loads up to that point and will continue to carry loads beyond that point. A simple analogy is to think of the concrete as being effectively stronger than that measured in a beam test; this effect varies as a function of dosage (not on weight but by volume) of fibers used in the concrete mixture.

Table 11.4 provides a summary of current categories of fibers, with general descriptions and application rates. For a more detailed discussion of fibers, see the *Guide to Concrete Overlays, Sustainable Solutions for Resurfacing and Rehabilitating Existing Pavements* (Harrington and Fick 2014).

Interlayers

When an unbonded concrete overlay pavement is poorly drained and experiences heavy truck traffic, scouring (stripping) of the asphalt interlayer may occur. In an effort to reduce the scour pore pressure and increase stability, some highway agencies modify the asphalt mixture to make it more porous. In particular, the sand content is reduced and the volume of 10 mm (0.38 in.) chip aggregate is increased. This modified mixture has a lower unit weight and lower asphalt content, and it is comparable in cost to typical surface mixtures. The Michigan DOT has designed an asphalt mixture with modified aggregate gradations to address stripping of the interlayer; see Table 11.5. It should be noted that many highway agencies have successfully used conventional asphalt mixes for interlayers as long as an adequate drainage system is employed.

Table 11.5. Michigan DOT Asphalt Interlayer Gradation

Sieve Size	Percent Passing
1/2 inch	100
3/8 inch	85–100
No. 4	22–38
No. 8	19–32
No. 16	15–24
No. 30	11–18
No. 50	8–14
No. 100	5–10
No. 200	4–7

Table 11.4. Summary of Fiber Types (adapted from Harrington and Fick 2014)

Fiber Type	Size (D = dia.) (L = length)	Years Used in U.S.	Typical Fiber Volume (lb/yd³)	Comments
Micro Synthetic Fibers	D < 0.012 in. (0.3 mm) L 0.50 to 2.25 in.	35	1.0 to 3.0	To reduce plastic shrinkage cracking and settlement cracking; limited effect on concrete overlay overall performance; more workability issues when using higher rates
Macro Synthetic Fibers	D > 0.012 in. (0.3 mm) L 1.50 to 2.25 in.	15	3.0 to 7.5	Increases postcrack flexural performance, fatigue-impact endurance; thinner concrete thickness; longer joint spacing; tighter joints, cracks; better handling properties, dispersion characteristics than steel fibers; not subject to corrosion
Macro Steel Fibers (carbon)	L 0.75 to 2.50 in.	40	33 to 100	Increases strain strength, impact resistance, postcrack flexural performance, fatigue endurance, crack width control per ACI 544.4R
Blended		15	Varies	Blend of small dosage of micro synthetic fibers and larger dosage of either macro synthetic fibers or macro steel fibers

As described previously, an alternative to an asphalt interlayer is the placement of a nonwoven geotextile interlayer; see Figure 11.16. The structural condition of the existing concrete pavement must be carefully assessed before selecting a geotextile instead of an asphalt interlayer. Material specifications for a geotextile used as an interlayer for unbonded overlays are shown in Table 11.6. The weight per square yard and thickness should be given when specifying a geotextile interlayer. Examples are the following:

- ≤ 102 mm (4 in.) overlay—308 g per m^2 (13 oz per yd^2) @ 3.3 mm (130 mils).
- ≥ 127 mm (5 in.) overlay—356 g per m^2 (15 oz per yd^2) @ 4.3 mm (170 mils).

Each highway agency is encouraged to develop their own weight and thickness criteria for geotextile interlayer based on their experiences and environmental conditions.

Figure 11.16. Nonwoven geotextile interlayer (The Transtec Group [no date])

Table 11.6. Geotextile Interlayer Material Properties (adapted from [The Transtec Group 2013])

Property	Requirements	Test Procedure
Geotextile Type	Nonwoven, needle-punched, no thermal treatment to include calendaring	EN 13249, Annex F (Certification)
Color	Uniform/nominally same color fibers	(Visual Inspection)
Mass per unit area	*≥ 450 g/m^2 (13.3 oz/sq. yd) ≥ 500 g/m^2 (14.7 oz/sq. yd) ≤ 550 g/m^2 (16.2 oz/sq. yd)	ISO 9864 (ASTM D 5261)
Thickness under load (pressure)	[a] At 2 kPa (0.29 psi): ≥ 3.0 mm (0.12 in.) [b] At 20 kPa (2.9 psi): ≥ 2.5 mm (0.10 in.) [c] At 200 kPa (29 psi): ≥ 0.10 mm (0.04 in.)	ISO 9863-1 (ASTM D 5199)
Wide-width tensile strength	≥ 10 kN/m (685 lb/ft)	ISO 10319 (ASTM D 4595)
Wide-width maximum elongation	≤ 130%	ISO 10319 (ASTM D 4595)
Water permeability in normal direction under load (pressure)	≥ 1 x 10^{-4} m/s (3.3 x 10^{-4} ft/s) at 20 kPa (2.9 psi)	DIN 60500-4 (modified ASTM D 5493)
In-plane water permeability (transmissivity) under load (pressure)	[a] ≥ 5 x 10^{-4} m/s (1.6 x 10^{-3} ft/s) at 20 kPa (2.9 psi) [b] ≥ 2 x 10^{-4} m/s (6.6 x 10^{-4} ft/s) at 200 kPa (2.9 psi)	*ISO 12958 (ASTM D 6574) or ISO 12958 (modified ASTM D 4716)
Weather resistance	Retained strength ≥ 60%	EN 12224 (ASTM D 4355 @ 500 hrs. exposure for grey, white, or black material only)
Alkali resistance	≥ 96% polypropylene/polyethylene	EN 13249, Annex B (Certification)

*Added to (The Transtec Group 2013) specification for overlays.

Ch 11. Concrete Overlays

7. Construction

Concrete overlays are constructed using conventional equipment and procedures. Total construction time for concrete overlays is significantly shorter than reconstruction of a roadway because the existing pavement is left in place and earthwork is limited to minor quantities. And, with adequate planning, expedited staging, and efficient paving operations, resurfaced streets and highways can be opened to traffic within short periods of time. Moreover, the HIPERPAV software tool can be used to assess early-age stresses in concrete that could otherwise lead to cracking, and it is especially useful when considering paving in less-than-desirable conditions (such as inclement weather conditions), when an overlay is particularly thin, or when a project has limited flexibility in scheduling (Xu et al. 2009).

There are some considerations for placement during cooler periods, such as in the spring or autumn. Under these conditions, the existing base and pavement will expand and contract with the daily change in ambient temperature. Cracking may occur if the concrete mixture has not gained enough strength to withstand the stresses caused by differential movement between the underlying pavement structure and the new concrete overlay. Accelerating the rate of strength gain in the concrete mixture is the recommended way to mitigate the effects of differential movement due to changes in ambient temperature. There are a number of methods that can be used to accomplish this; they may be used alone or in many cases in combination:

- Heat the concrete to maintain a fresh concrete temperature of at least 24°C (75°F).
- Use a nonchloride accelerating admixture.
- Cover the new overlay pavement with insulating blankets, burlap, and/or polyethylene sheeting.
- When possible, reduce the quantity of supplementary cementitious materials in the mixture.

Payment for concrete overlays is typically based on two items: square yards and cubic yards. The surface is measured to account for the square-yard surface area, and batch tickets are collected to account for the cubic-yard concrete volume, including variable depths.

Construction of Bonded Concrete Overlays

Bonding is an essential feature of these overlays. A combination of good design and construction practices helps ensure that a bond is achieved and contributes to bonded overlays' overall performance.

- **Repair of Existing Pavement**—Isolated areas of deterioration in the existing pavement should be repaired to promote long-term durability of the resurfaced pavement.
  - *Bonded on concrete pavement*: Unrepaired cracks, especially working cracks, will usually reflect through the bonded overlay unless joints are sawed directly over them. Wide random cracks may require FDR. Asphalt patches should be replaced with concrete or simply filled with concrete when the overlay is placed.
  - *Bonded on asphalt and composite pavements*: Potholes, areas of moderate-to-severe alligator cracking, and areas lacking base/subgrade support may require partial or full-depth spot repairs. Milling may be required to remove surface distortions of 51 mm (2 in.) or more or to reduce high spots and ensure a consistent minimum overlay depth. Care should be taken not to mill off too much asphalt and thereby reduce the existing pavement's load-carrying capacity. The condition of the asphalt at the desired milled depth should be predetermined by coring or other means to ensure that it is not stripped or raveled, which could prevent proper bonding with the concrete overlay. A minimum asphalt structure of 76 mm (3 in.) is recommended after milling. Transverse thermal cracks wider than the largest aggregate in the overlay mixture should be cleaned and filled because the concrete overlay can span smaller cracks.
  - *Bonded on composite pavement*: If there is vertical movement of the underlying concrete adjacent to a crack, the movement can be corrected by replacing or retrofitting the joint. Or the crack can be controlled without repairing the underlying pavement by adding fibers to the mixture or, in some cases, placing reinforcing steel (rebar) over the joint in the overlay. The use of rebar should be avoided when possible because it makes the overlay harder to recycle later. Again, a 76-mm (3-in.) minimum layer of asphalt is recommended after milling.
- **Preparation of Existing Pavement Surface, Following Repairs and Before Cleaning**—A bonding grout or epoxy is not required or recommended.

- *Bonded on concrete pavement*: Concrete pavement surfaces can be roughened to promote bonding with the overlay. The most common and effective roughening procedure on concrete surfaces is shotblasting. On concrete pavements, milling alone often causes microcracking that can weaken the pavement surface and compromise the overlay's performance. Therefore, after milling, shotblasting should be used to remove microcracking.
- *Bonded on asphalt and composite pavements*: Milling may be considered to roughen the surface, which will likely enhance the bond.

- **Surface Cleaning**—To promote bonding, the existing pavement surface should be thoroughly clean and dry. Pressure washing should be considered only when dust control is an issue or when mud is on the surface. No water should be standing on the pavement when the overlay is placed because it could prevent bonding.
  - *Bonded on concrete or asphalt pavement*: Immediately before placing the overlay, the pavement should be swept, followed by cleaning in front of the paver with compressed air.
- Placement Temperature.
  - *Bonded on concrete pavement*: The best time to place a bonded overlay on a concrete pavement is when the difference in temperature between the existing slab and the new overlay is minimal.
  - *Bonded on asphalt and composite pavements*: If the existing pavement surface is very hot (49°C [120°F] or higher), it may pull water from the overlay, making it more susceptible to shrinkage cracking. The surface of the existing pavement can be cooled by water mist, as long as no standing water remains when the overlay is placed.
- **Joint Pattern and Sawing Window**—Thinner overlays have greater potential for rapid shrinkage, contraction and expansion, and curling and warping, all of which can cause stresses to develop at the bond interface. The joint pattern and timeliness of joint construction are important for relieving the stresses. The joint pattern varies based on the existing pavement type. Although not always necessary, the HIPERPAV software can help users establish ideal joint-sawing windows hours or days before constructing overlays on higher-level projects like interstate highways.
  - *Bonded on concrete pavement*: Joints in the overlay should be constructed directly over joints in the existing pavement to help prevent reflective cracking. Transverse joints should be cut full depth plus 13 mm (0.5 in.) and longitudinal joints at least 13 mm (0.5 in.) or greater.
  - *Bonded on asphalt and composite pavements*: The recommended joint pattern is small, roughly square panels, typically 0.9–2.4 m (3–8 ft) or 1.5 times the slab thickness in inches. If possible, longitudinal joints should be constructed outside the wheel path.
- **Curing**—Maintaining the bond is especially critical during the first few days when the overlay is susceptible to curling and warping stresses, especially at the pavement edges. Therefore, the bond must be protected through thorough curing practices, particularly at the pavement edge. This can be accomplished by minimizing relative humidity and temperature differentials between the two layers and by keeping early traffic away from the pavement edges until adequate bond strength has been achieved (usually when opening strength has been achieved). Within 30 minutes of overlay placement, white-pigmented curing compound should be applied liberally. Some highway agencies apply compound at as much as 1.5 to 2 times the manufacturer's recommended rate for typical pavements. After application, the finished surface, including the vertical faces of the pavement edges, should appear as a uniformly painted white surface.

Construction of Unbonded Concrete Overlays

A variety of good design and construction practices can contribute to the successful performance of unbonded concrete overlays:

- **Unbonded on Concrete Pavement**—Isolated panels where movements indicate potential nonuniform subgrade support or severe MRD, as well as badly shattered panels and tenting panels (early stages of blowups), may require FDR. An asphalt or geotextile interlayer between the overlay and existing pavement is required.
- **Unbonded on Asphalt and Composite Pavements**—Direct placement without milling is appropriate if rutting is less than 51 mm (2 in.). Surface distortions of at least 51 mm (2 in.) should be spot-milled to

less than 25 mm (1 in.) to ensure minimum overlay thicknesses throughout the overlay. An adequate layer of asphalt (76 mm [3 in.]) should remain to ensure the asphalt will function as a uniform base.

- **Preparation of Existing Surface**—Preparation of an existing concrete pavement is very different from preparation of an existing asphalt surface.
  - *Unbonded on concrete pavement*: A thin interlayer (usually a 25-mm [1-in.] asphalt layer or nonwoven geotextile fabrics) must be placed on the existing concrete pavement to prevent the existing surface from bonding with the overlay. The interlayer also provides a shear plane that helps prevent reflective cracking up into the overlay.
  - *Unbonded on asphalt and composite pavements*: The pavement surface should simply be swept to remove debris. Remaining small particles are not a problem.
- **Placement Temperature**—If the asphalt surface is very hot (49°C [120°F] or higher), it may draw water from the overlay, making it more susceptible to shrinkage cracking. The asphalt surface can be cooled by water mist immediately before overlay placement, as long as no standing water remains when the overlay is placed.
- **Joint Sawing and Curing**—Stiff support provided by the existing pavement can increase stresses caused by overlay curling and warping. Spacing joints more closely than on normal concrete pavement projects helps reduce these stresses and the related potential for cracking. Proper timing of saw cutting and thorough, timely curing on all surfaces also help reduce potential stress development.
  - *Unbonded joint spacing on concrete pavements:*

    < 152 mm (6 in.): 1.8-m by 1.8-m (6-ft by 6-ft) panels.

    > 152 mm (6 in.): 2 times overlay thickness in inches to give the spacing in feet (not to exceed a maximum of 4.6 m [15 ft]).
  - *Unbonded jointing spacing on asphalt pavement:*

    < 152 mm (6 in.): 1.5 times overlay thickness in inches to give the spacing in feet.

    > 152 mm (6 in.): 2 times overlay thickness in inches to give the spacing in feet.

Construction of Geotextile Interlayer Fabric

Before placing the geotextile, the surface of the existing pavement should be swept clean of loose material with either a mechanical sweeper or an air blower. Then conventional placement practices and procedures should be followed for placing the interlayer.

Both pervious (geotextile fabric or open-graded asphalt) and impervious (densely graded HMA) types of interlayers must drain at the pavement edges or risk trapping water, which can be very damaging. The layer can either be daylighted at the edges (allowing the egress of water) or terminate in a subdrain or other layer (allowing the water to flow away from the pavement structure).

In general, the following construction practices have resulted in successful installations of geotextile interlayers:

- Place the material as shortly before paving as possible (ideally no longer than 2 to 3 days) to reduce the potential for it to be damaged.
- Before placing the nonwoven geotextile material, do the following:
  - Repair the existing pavement to correct any significant cracking.
  - When faulting is greater than 6 mm (0.25 in.), or as specified by the engineer, the faulting may be reduced by milling.
  - Sweep the pavement surface clean.
- Roll the material onto the base or other surface, keeping the nonwoven geotextile tight with no wrinkles or folds.
- Roll out sections of the material in a sequence that will facilitate good overlapping, prevent folding or tearing by construction traffic, and minimize the potential that the material will be disturbed by the paver.
- Overlap sections of the nonwoven geotextile material a minimum of 152 mm (6 in.) and a maximum of 254 mm (10 in.), and ensure that no more than three layers overlap at any point; see offsets shown in Figure 11.17.

- Ensure that the edge of the material along drainage areas extends at least 102 mm (4 in.) beyond the pavement edge and terminates above, within, or adjacent to the pavement drainage system.
- Secure the material with pins (nails) punched through 51- to 70-mm (2.0- to 2.75-in.) diameter galvanized discs placed 1.8 m (6 ft) apart or less, depending on conditions; see Figure 11.18.
- A limited number of projects have used an adhesive for securing the geotextile to the existing pavement; for more information see http://multimedia.3m.com/mws/mediawebserver?mwsId=SSSSSuH8gc7nZxtU5x_948_GevUqe17zHvTSevTSeSSSSSS--.
- Construction traffic on the geotextile should be limited to only that necessary to facilitate concrete paving. If construction traffic is placed on the geotextile, precautions should be taken to mitigate tears and wrinkles in the fabric; these include the following:
    - Leaving temporary gaps in the geotextile where trucks are crossing and making sharp turns.
    - Minimizing sharp turns and heavy braking that can cause tearing and wrinkling.
    - Reducing the travel speed of construction traffic.
    - Considering the use of asphalt interlayers in situations with tight radii, such as interchange ramps; in such areas, the geotextile may require numerous cuts and overlaps.

Figure 11.17. Overlap of nonwoven geotextile material interlayer (Harrington and Fick 2014)

Figure 11.18. Fastening nonwoven geotextile interlayer (Harrington and Fick 2014)

Strength Criteria for Opening to Traffic

Bonded Systems

If proper surface treatment, curing, and sawing are employed in the construction of bonded concrete overlays, the bond strength at the time of opening should be adequate if 3.7 MPa (540 lbf/in.^{2}) flexural or 24.8 MPa (3,600 lbf/in.^{2}) compressive strength is achieved. As a rule of thumb for bonded concrete overlays, the bond tensile strength may be on the order of 2 to 10 percent of the compressive strength and the bond shear strength approximately 4 to 20 percent of the compressive strength.

Unbonded Systems

Because unbonded overlays are essentially a concrete pavement on a high-quality base, it is appropriate and somewhat conservative to use opening strength criteria that are commonly used for conventional paving. For example, a minimum flexural strength (third point) of approximately 2.3 MPa (340 lbf/in.^{2}) or 12.4 MPa (1,800 lbf/in.^{2}) compressive strength can be used for noninterstate traffic.

Payment of Concrete Overlays

Payment is typically based on two items: square yards and cubic yards. The surface is measured to account for the square-yard surface area, and batch tickets are collected to account for the cubic-yard concrete volume, including variable depths.

Key Points for Concrete Overlay Construction

Normal concrete paving construction practices can be used to complete concrete overlay projects as quickly and efficiently as any other paving method; key factors are summarized in Table 11.7.

Table 11.7. Construction Considerations for Bonded and Unbonded Overlays (Harrington and Fick 2014)

Construction Consideration	Bonded Overlays of Concrete	Bonded Overlays of Asphalt or Composite	Unbonded Overlays of Concrete	Unbonded Overlays of Asphalt or Composite
1. Mixture Design				
Aggregate:				
Physically and chemically stable and durable	X	X	X	X
Well-graded mix	X	X	X	X
Match aggregate thermal properties with existing pavement	X			
Maximum aggregate size should be D/2.5 in relation to the new overlay thickness	X	X	X	X
Use conventional mixtures with Type I or II cement.	X	X	X	X
Use fly ash and slag to reduce permeability with w/cm ratio not to exceed 0.45.	X	X		
Use water reducer to help maintain w/cm ratio and desired slump, as well as to increase strength.	X	X	X	X
If accelerated construction is desired, accomplish this through careful scheduling and diligent execution; accelerated concrete mixtures should only be used in limited areas where early opening cannot be achieved through other means.	X	X	X	X
Fibers may be used to increase the "toughness" of concrete (measure of its energy-absorbing capacity), improve resistance to deformation, hold concrete together in case of cracking, and serve as an insurance policy that protects the surface from unseen base conditions.	X	X		
Verification testing in the laboratory of nonstandard mixes (trial batches) and specifications of testing at temperatures representative of site conditions is encouraged to flag any mix problems.	X	X	X	X
2. Grade Control				
Centerline profile only (as-built) with uniform finished cross section	X	Mill and concrete overlay		
Three-line profile (edges and centerline) when cross slope varies or surface distortions exist	X	Little or no milling	Inlays only	Inlays only
Measure off existing pavement or top of milled surface to set stringline or form. Adjust individual points up to produce a smooth line.	X	X	X	X
Survey 100–500 ft (30.5–152 m) cross sections when shouldering, foreslopes, and backslopes need adjusting. If the existing profile grade is irregular, additional centerline elevations may be necessary for grade corrections in certain locations for smoothness.			X	X
Survey bridge tie-ins or bridge clearance conditions and extreme superelevations.	X	X	X	X
To prevent thicker asphalt separation layer and thus compaction, stability, and grade control issues, use concrete to make up any 3 in. (75 mm) or greater variances in grade and a nominal 1 in. (25 mm) asphalt separation layer.			X	

Table 11.7. Construction Considerations for Bonded and Unbonded Overlays (continued)

Construction Consideration	Bonded Overlays of Concrete	Bonded Overlays of Asphalt or Composite	Unbonded Overlays of Concrete	Unbonded Overlays of Asphalt or Composite
3. Preoverlay Repairs for Uniform Support				
Minimal minor repairs of surface defects. Remove deteriorated area and replace with overlay.	X			
An engineer should observe final condition of subbase pavement prior to overlay construction. For minimal isolated distress that causes some loss of structural integrity that cannot be overcome with milling, thicken the overlay in this area.		X		
Replace isolated areas of subbase pavement when there is evidence of active movement.			X	X
Joint deterioration with little or no faulting can be bridged with the overlay.			X	
To widen the roadway, excavate the shoulder to allow for the widened thicker section to be placed with the overlay.	X	X	X	X
Fill cracks in the HMA with sand or flowable mortar when the crack width exceeds the maximum coarse aggregate size used in the concrete overlay mixture.		X		X
4. Surface Preparation				
Surface roughness for bonding:				
Shotblasting (even after milling)	X			
Milling to remove significant distortions or reduce high spots		X		X
Surface cleaning:				
Sweeping followed by high-pressure airblasting (waterblasting may be needed to remove dirt tracked onto surface)	X	X		
Surface sweeping only			X	X
Maintain a clean and dry surface.	X	X		
Sprinkle (mist) the existing pavement when the surface temperature exceeds 120°F; use compressed air to remove any standing water directly ahead of the concrete-placing operation	X	X	X	X
Place nominal 1 in. (25 mm) asphalt layer to separate concrete layers and prevent bonding. When heavy truck traffic is anticipated, it is advisable to consider a drainable asphalt layer and drainage system.			X	
If the existing asphalt surface of a composite pavement section remains intact, it can serve as a separation layer.				X
5. Concrete Placement				
When the surface temperature of the asphalt is at or above 120°F (49°C), surface watering can be used to reduce the temperature and minimize the potential for shrinkage cracking. No standing water should remain at the time the overlay is placed.		X	X	X
The bonding of the overlay can be affected by the climatic conditions at the time of placement. Significant stresses that develop due to rapid changes in temperature, humidity, and wind speed may reduce the bond strength under severe conditions. HIPERPAV can predict interface bond stress based on numerous factors.	X	X		
Feeding concrete consistently into the paver requires an adequate number of batch delivery trucks. The number of trucks will often dictate the slipform or placement speed. The entire cycle of mixing, discharging, traveling, and depositing concrete must be coordinated for the mixing plan capacity, hauling distance, and spreader and paving machine capabilities. Extra trucks may be needed as the haul time increases.	X	X	X	X
Do not track paste or dirt onto the existing surface ahead of the paver because it can cause bond failure.	X	X		

Table 11.7. Construction Considerations for Bonded and Unbonded Overlays (continued)

Construction Consideration	Bonded Overlays of Concrete	Bonded Overlays of Asphalt or Composite	Unbonded Overlays of Concrete	Unbonded Overlays of Asphalt or Composite
The manner in which the crew deposits concrete in front of the paving operation is an important factor for creating a smooth pavement surface in overlay projects. Placement in front of slipform paver should be done in small overlapping piles so as to minimize lateral movements.	X	X	X	X
Properly established, secure, and maintained stringline is very important for smoothness; constant and continuous paving prevents interruptions that lead to bumps.	X	X	X	X
Tiebars may be appropriate in an open-ditch situation when constructing 3- to 6-ft (0.9- to 1.8-m) widening units and overlay thickness is 5 in. (125 mm) or greater. Normally, tiebars are not used for lane widening to prevent cracking from stresses due to differential expansion and contraction between lanes.	X	X		
Dowel bar use should follow full-depth pavement requirements. **Pavements less than 7 inches thick should not use load transfer dowels.** When used for thicker pavements, they should be located approximately in the mid-third of the overlay thickness. Isolated thicker sections should not dictate a change in basket height or dowel bar insertion depth.		X	X	X
Texturing needs to be performed at the right time so as not to disturb setting of the concrete. Shallow longitudinal tining or burlap/turf are two effective textures. Burlap/turf drag has shown adequate friction with a quiet surface when hard sands are used in the mix.	X	X	X	X
6. Curing to Prevent Rapid Loss of Water from Concrete				
Proper curing of bonded and thin unbonded overlays is particularly important because they are thin with a large surface compared with the volume of concrete. The curing rate may be increased from the normal rate to provide additional protection. Standard curing compound rates may be used for thicker unbonded overlays.	2 times normal	1.5–2 times normal	1.5–2 times normal	1.5–2 times normal
During hot weather, steps should be taken to reduce the evaporation rate from the concrete. For significant evaporation, provide a more effective curing application, such as fog spraying, or apply an approved evaporation retarder.	X	X	X	X
Adequate curing of overlays on a stiff support system (especially on underlying concrete pavement) is important to minimize curling and warping stresses.	X	X	X	X
7. Joints				
Joint spacing for concrete overlays requires special consideration for each type.				
Joints are to be matched with underlying concrete to prevent reflective cracking.	X			
When feasible, it is a good policy to mismatch joints and/or cracks to maximize load transfer from the underlying pavement. Some states that have not intentionally mismatched joints, however, have not experienced any adverse effects.			X	X
Slab dimensions match the underlying pavement.	X			
The recommended joint pattern for bonded overlays of asphalt should not exceed 1.5 times the overlay thickness in inches.		X		
For overlays less than or equal to 6 inches thick, the slab dimensions (in feet) should not exceed 1.5 times the overlay thickness in inches (e.g., 4 in. x 1.5 ft/in. = 6 ft).			X	X
For overlays greater than 6 inches, the slab dimensions (in feet) should not exceed 2.0 times the overlay thickness in inches, not to exceed 15 ft.			X	X
Because of the potential for higher curling and warping stress from a rigid underlying pavement, shorter than normal spacing is typical.			X	X

Table 11.7. Construction Considerations for Bonded and Unbonded Overlays (continued)

Construction Consideration	Bonded Overlays of Concrete	Bonded Overlays of Asphalt or Composite	Unbonded Overlays of Concrete	Unbonded Overlays of Asphalt or Composite
Joint sawing:				
The timing of sawing is critical. Sawing joints too early can cause excess raveling. HIPERPAV may be useful in helping to predict the appropriate time window for joint sawing, based on the concrete mix design, construction times, and environmental conditions.	X	X	X	X
Sawing must be completed before stresses exceed the strength developed. Sawing too late can lead to uncontrolled cracking.	X	X	X	X
Transverse joint saw-cut depth for conventional saws	Full depth + 0.50 in. (13 mm)	t/3	T/4 min.-T/3 max.	
Transverse joint saw-cut depth for early-entry saws	Full depth + 0.50 in. (13 mm)	Not < 1.25 in. (32 mm)	Not < 1.25 in. (32 mm)	
Longitudinal joint saw-cut depth	T/2 (at least)	T/4 – T/3	T/4 – T/3	
Transverse joint width must be equal to or greater than the underlying crack width at the bottom of the existing transverse joint.	X			
Sealing:				
Seal joint using low-modulus hot-pour sealant with narrow joint.	X	**	*	*

Some agencies have experienced problems with asphalt stripping of the separation layer, particularly under heavy truck traffic and high speeds; therefore, sealing is important in these conditions. On lower-speed roads without a heavy traffic loading, some agencies successfully do not seal.

**Joint between overlay and curb and gutter

Ch 11. Concrete Overlays

8. Troubleshooting

Most of the chapters in this Preservation Guide provide tables for troubleshooting regarding a particular preservation technique. Concrete overlays are a different preservation technique in that they are a new pavement surface placed on top of an existing pavement. The best way to address and prevent problems using a concrete overlay is to follow the techniques described herein and in detail in Chapter 3 of the *Guide to Concrete Overlays, Sustainable Solutions for Resurfacing and Rehabilitating Existing Pavements* (Harrington and Fick 2014).

As stated in the introduction of this chapter, nearly all the documented cases of premature overlay failure can be traced to some violation of the principles of providing uniform support for the overlay and understanding the movements of the underlining pavement. Not following these principles can result in "picking the wrong project" for a concrete overlay and greatly increasing the risk of poor performance. The selection of the appropriate project and following the guidelines outlined in Table 11.7 will contribute to a successful concrete overlay project.

9. Summary

Concrete overlays can be used to provide solutions for a range of issues and deficiencies in existing pavement structures. Thinner overlays (typically 51–102 mm [2–4 in.] thick) can be effective preservation treatments. Overlays equal to or greater than about 127 mm (5 in.) are considered more structural overlays and are for more major rehabilitation.

Many of the changes and improvements in concrete overlay technology during the last 5 years have been in improved design procedures, detailed construction guidelines, and relevant specifications. Among the major advances has been the better definition of how the existing pavement should be evaluated and prepared in advance of a concrete overlay; others have to do with improved methods of placement and expanded use of synthetic fibers. Fiber-reinforced concrete resurfacings have been on the increase also because the fibers contribute to the performance of thin concrete resurfacings. The improved performance comes from the increase in concrete structural integrity through improved toughness and durability of the concrete.

Major research projects have been completed providing long-term solutions of bonded and unbonded overlays of existing pavements, both concrete and asphalt. These efforts, along with favorable pricing conditions and a national focus on training outreach and technical guidance development, have led to greater acceptance and increased use of high-quality concrete overlays.

Many agencies are adopting sustainability considerations in their pavement management decisions. Quantifying the impact of pavement decisions on the primary sustainability factors of environment, social, and economics is exceedingly difficult; however, by looking at the sustainable benefits of concrete overlays from a qualitative perspective, the following can be concluded:

- Preserving the existing pavement has a minimal impact on the environment (no waste products are produced).
- User delays during construction are reduced as compared to reconstructing a pavement.
- Concrete overlays are capable of maintaining their smoothness and frictional characteristics for many years, which provides significant societal benefits.
- Concrete overlays typically have a lower life cycle cost than asphalt overlays of equivalent design life.

Concrete overlay pavement systems can be sustainable for a wide range of design life choices. Rather than removing and reconstructing the original pavement, the owner maintains and builds equity in it, realizing a return on its original investment as long as the original pavement remains part of the system.

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Chapter 12

Strategy Selection

1. Learning Outcomes

This *Preservation Guide* has discussed in detail a variety of concrete pavement preservation and restoration techniques. These range from relatively simple and straightforward treatments, such as joint resealing, to more involved techniques, such as concrete overlays. Thus far, however, only limited guidance has been provided on determining which treatment (or which combination of treatments) is appropriate for a given concrete pavement project.

The selection of an appropriate preservation or restoration treatment for a given concrete pavement project requires a systematic, step-by-step approach that considers all relevant factors. This chapter outlines a recommended procedure that can be used to select the most appropriate treatment types or strategies. Upon successful completion of this chapter, the participants will be able to accomplish the following:

- Describe the treatment selection process.
- List the factors that might enter into the selection process.
- Describe the pavement deficiencies that are best addressed by the different preservation treatments.
- Describe how the benefits and costs of alternative treatment strategies are computed as part of a cost-effectiveness analysis.
- Describe the process used to select the preferred treatment strategy.

2. Introduction

Across the country, the maintenance and rehabilitation of the existing highway network has become a central focus. The need to maintain the nation's already-constructed network is essential to the economical operation of the overall transportation system. Because of ever-tightening financial conditions, however, accomplishing that task has become more and more challenging. As a result, both the traveling public and highway agencies are seeking better solutions to their mutual concerns about the operating conditions on the nation's roads. The incorporation of pavement preservation is viewed to be essential to this process because these techniques have been shown to be effective at delaying more costly and invasive rehabilitation procedures, thereby extending pavement service lives, minimizing traffic disruptions, reducing the work zone safety risks to both workers and highway users, and minimizing life-cycle costs.

Determining the right treatment for the right pavement at the right time can be a complex problem that requires simultaneously evaluating a number of different influencing factors, including costs and performance. This chapter provides information about the types of factors that should be considered when selecting an appropriate preservation 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 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.

3. Treatment Selection Process

Overview of the Selection Process

Chapter 2 discussed the importance of 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 treatment is most ideal in terms of cost effectiveness and other considerations. Although such data can be effective in screening projects for preservation, more information about a project is usually needed to confirm that preservation is appropriate and to help identify candidate preservation treatments. 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 on-site pavement evaluation 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 for the pavement) and then ultimately to determine how to best address those deficiencies (i.e., the best course of action). For example, if the 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 activities.

If more global structural or material problems exist that were not indicated by the pavement management data, then the pavement section is more likely suited for a structural overlay or perhaps even complete reconstruction (in the most severe case). Because discussion of reconstruction is outside the scope of this guide, this chapter focuses on the selection of the most appropriate concrete pavement preservation treatments, including the use of concrete overlays.

At the project level, the process of determining the most appropriate pavement preservation activities for concrete pavements is a fairly straightforward one. Based on a collective review of a number of recently published documents, 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; Anderson, Ullman, and Blaschke 2002; NCHRP 2004; Peshkin et al. 2011):

- Conduct a thorough pavement evaluation.
- Determine causes of distresses and deficiencies.
- Identify treatments that address deficiencies.
- Identify constraints that could influence treatment selection.
- Develop feasible treatment strategies.
- Assess the cost-effectiveness of the alternative treatment strategies.
- Select preferred strategy.

Each of these different 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 the pavement. Overall, the pavement evaluation procedures focus on determining both the structural and functional adequacy of the current pavement. As described in Chapter 3, the structural condition refers to the ability of the pavement to carry current and future traffic loading, whereas the functional condition refers to the ability of the pavement to provide a smooth and safe riding surface for the users. The structural condition of the pavement is determined from the results of the condition and drainage surveys, deflection testing, and any material sampling and testing. The functional condition is primarily determined by reviewing the results of any roughness and friction testing (or, if appropriate, noise testing). Table 12.1 presents a summary of the different condition parameters included in an evaluation and the different testing methods used to assess them.

Table 12.1. Areas of Overall Condition Assessment and Corresponding Data Sources (adapted from NCHRP [2004])

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	✓	✓				
Shoulders Adequacy	✓		✓			✓
Variability along Project	✓	✓	✓	✓		✓

Step 2: Determine Causes of Distresses and Deficiencies

One of the most important steps of the treatment selection process is to collectively review 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. By knowing the underlying causes of the distresses that are observed, appropriate preservation treatments can be identified.

Table 12.2. Concrete Pavement Distress Types and Causes (adapted from Hall et al. [2001] and Miller and Bellinger [2003])

Distress	Causes	Notes
Linear cracking (transverse, longitudinal, or diagonal)	Fatigue damage, often in combination with slab curling and/or warping; drying shrinkage; improper transverse or longitudinal joint construction; or foundation 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	Fatigue damage, 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 available.
Alkali-aggregate distress	Compressive stress building up in slab, due to swelling of gel produced from reaction of certain susceptible aggregates with alkalis in the cement	Alkali-aggregate reaction includes ASR and ACR.
Map cracking and crazing	Alkali-aggregate reaction or overfinishing	Hairline cracks in upper surface of slab are cosmetic but can deteriorate into scaling.
Scaling	Overfinishing, 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	Loss of adhesion of sealant to joint walls, extrusion of sealant from joint, infiltration of incompressibles, oxidation of sealant, and cohesive failure (splitting) of the sealant are all considered joint seal damage.
Joint spalling (also called joint deterioration)	Compressive stress buildup in the slab (due to incompressibles or alkali aggregate reaction); D-cracking; misaligned or corroded dowels; poorly consolidated concrete in vicinity of joint; or damage caused by joint sawing, joint cleaning, cold milling, or grinding	Joint spalling includes cracking, breaking, chipping, or fraying of slab edges within 0.3 m (1 ft) of transverse or longitudinal joint.
Blowups	Compressive stress buildup in the slab (due to infiltration of incompressible, or alkali-aggregate reaction)	A blowup may occur as a shattering of the concrete for several feet on both sides of the joint, or an upward buckling of the slabs.
Pumping	Excess moisture in the pavement structure, erodible base or subgrade materials, and high volumes of high-speed, heavy wheel loads	Pumping can lead to loss of support beneath the slabs and the development of faulting. Dowel bars and nonerodible bases can 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 2–3 mm (0.08–0.12 in.).
Roughness caused by curling/warping	Moisture gradients through the slab thickness, 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	These detract from riding comfort and at high severity may pose a safety hazard.
Polishing	Abrasion by tires	Polished wheelpaths may pose a wet-weather safety hazard.
Popouts	Freezing in coarse aggregates near the concrete surface	This is a cosmetic problem rarely warranting repair.

Step 3: Identify Treatments That Address Deficiencies

The main objective of the third step is to identify the pavement preservation treatments (or series of preservation treatments) that would be potentially useful at addressing one or more of the identified pavement deficiencies. It is important to remember that the scope of this document is limited to 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 contained in Chapters 4 through 11, a summary of their general applications is presented in Table 12.3.

Table 12.3. Concrete Pavement Preservation Treatments Based on Distress (adapted from Hall et al. [2001]).]

	Concrete Pavement Preservation Treatment											
Distress	Slab Stabilization	Slab Jacking	Partial-Depth Repair	FDR	Retrofitted Edgedrains	DBR	Cross Stitching/ Slot Stitching	Diamond Grinding	Diamond Grooving	Joint Resealing	Crack Sealing	Thin Concrete Overlay
Corner breaks			✓	✓							✓[a]	
Linear cracking				✓			✓[b]				✓[a]	
Punchouts				✓								
D-cracking				✓[c]								✓[c]
Alkali-aggregate reaction				✓[c]								✓[c]
Map cracking, crazing, scaling			✓									✓
Joint seal damage										✓		
Joint spalling			✓	✓								✓
Blowup				✓								
Pumping	✓				✓	✓	✓					
Faulting						✓		✓				✓
Bumps, settlements, heaves		✓		✓				✓				✓
Polishing/ low friction								✓	✓			✓

Note: Many of these treatments are commonly done in combination to fully address the pavement deficiencies.

[a] Cracks with limited vertical movements.

[b] Longitudinal cracks only.

[c] On 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.

In general, the following sequence of checks can be used to help identify those treatments that may be appropriate for a given project:

1. **Assess Slab Support Conditions**—When assessing the support conditions of concrete slabs, it is important to test for voids at slab corners, as well as test the load transfer efficiency at transverse joints. One good indication that there is a slab support problem is the presence of pumping (i.e., the presence of fine material on the pavement surface at 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 dowel bar retrofit.

2. **Correct Localized Distress That Is Contained in the Upper Half of the Slab Thickness**—In concrete pavements, it is not uncommon to have localized areas of distress that are contained in the upper half of the slab thickness. Common distresses in this category include joint spalling, or map cracking, crazing, or scaling. If any of these distresses are present in an amount or severity that requires attention, a partial-depth repair is typically the best treatment to correct the distress. A thin concrete overlay, however, may also be a suitable solution for a surficial problem that is widespread over an entire project.

3. **Correct Localized Distress Not Contained 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, transverse cracking, or MRD), 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, then crack sealing may be a suitable solution.

4. **Correct Functional Distresses**—Many otherwise sound concrete pavements may be exhibiting 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 often an effective treatment option.

5. **Assess Joint Sealant Condition**—One final step in the strategy selection process is to assess the performance of the joint sealant. In general, if the original concrete 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 treatment alternatives have caused the effectiveness of the joint sealant to be compromised to a significant extent (e.g., 25 percent 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 specific pavement preservation project before it is opened to traffic.

Step 4: Identify Constraints and Key Selection Factors

After compiling a list of possible effective treatments under Step 3 and before proceeding further in the treatment selection process, it is important to check those possible effective treatments against a list of any project-specific constraints or other key selection factors that may come into play. Some of the potential 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, safety).
- Environmental impact (e.g., contamination, greenhouse gas emissions generated during construction, etc.).
- Conservation of natural resources (i.e., recycling, reuse).
- User impacts during service (e.g., smoothness and friction levels, noise emissions).

- Worker safety during construction.
- Availability of needed equipment and materials.
- Competition among providers of materials.
- Agency policies.

It is important that all outside constraining factors be identified at this point of the selection process to avoid conducting unnecessary work in the upcoming steps.

Step 5: Develop Feasible Treatment Strategies

A treatment strategy is a plan that defines what treatments to apply and when to apply them over a selected time period. For example, a strategy using only one treatment could be to conduct diamond grinding every 8 to 10 years for the next 25 years. Another strategy could be to install retrofitted dowel bars, followed by diamond grinding during the same construction project. It is not uncommon to concurrently conduct more than one of the concrete pavement preservation activities in a single project because the various concrete pavement preservation activities complement each other. Therefore, the purposes of this step are the following:

- Determine all of the different activities that need to be conducted to best address the pavement's needs.
- Determine if it is best to conduct the activities concurrently or to apply the individual activities at different times in the future.

As mentioned previously, it is not uncommon for different treatments to be used concurrently in a single project. If used concurrently, however, it is important to conduct those activities in a logical construction order that maximizes the effectiveness of each individual treatment while protecting any previously performed repairs (ACPA 2006). For example, full- and partial-depth repairs, dowel bar retrofitting, and slab stabilization activities should always be conducted prior to diamond grinding. Delaying diamond grinding until after these other activities have been conducted maximizes the resulting smoothness associated with diamond grinding. A summary of the logical order of conducting pavement preservation techniques concurrently on a project is displayed in Figure 12.1 (ACPA 2006). Obviously, not every project will require every treatment, but it is recommended that this sequence of treatment applications be maintained when performed concurrently on a single project.

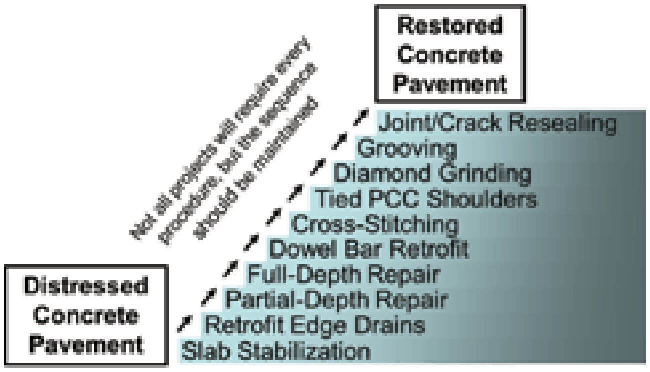

Figure 12.1. Recommended sequence of performing preservation activities concurrently on a given project (ACPA 2006)

Each individual treatment combination or treatment timing scenario can be considered a separate treatment strategy for the pavement. Although there is usually an obvious choice for the most appropriate strategy, competing strategies can be objectively compared by considering the overall cost-effectiveness of each strategy.

Step 6: Assess the Cost-Effectiveness of the Alternative Treatment Strategies

Because the concrete pavement preservation treatments address different pavement deficiencies, cost-effectiveness analysis techniques are not typically needed to help select appropriate strategies. Where cost-effectiveness results do become important, however, is when the concrete pavement preservation treatments and strategies are being considered along with more extensive rehabilitation techniques (i.e., overlays) or reconstruction. A cost-effectiveness analysis provides an objective method of comparing the costs associated with different treatments applied at different times over the life of a pavement. These results are of particular interest to those 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 a long-term approach to analyzing cost-effectiveness, one that considers the performance benefits of one or more treatments and the associated costs of applying those treatments. The approach is referred to as the benefit-to-cost ratio (BCR) analysis method.

The BCR analysis method combines the results of individual evaluations of treatment benefits (B) and treatment costs (C) to generate a benefit-to-cost (B/C) ratio (Peshkin et al. 2011). The B/C ratios of alternative treatment strategies (including a "do-nothing" strategy, if desired) are then compared, and the strategy with the highest ratio is deemed the most cost effective. Since the analysis is performed over a long period covering the life-cycle of a pavement (usually at least 25 to 40 years beginning from the original construction), the costs and performance characteristics of the existing pavement and all future projected preservation and rehabilitation treatments associated with each strategy must be estimated.

As mentioned in Chapter 2, pavement management databases provide the best source of data for modeling pavement performance and developing service life estimates of the original pavement, the preservation-treated pavement, and the rehabilitated pavement. Historical condition data in the form of overall condition indicators and individual distress parameters can be obtained 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 those 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 need for structural improvement (i.e., major rehabilitation or reconstruction) and/or functional improvement (i.e., friction or smoothness restoration).

In the BCR method, the benefits associated with a particular treatment strategy are evaluated from the standpoint of benefits accrued to the highway 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. The top portion of Figure 12.2 illustrates the assessment of benefits using the

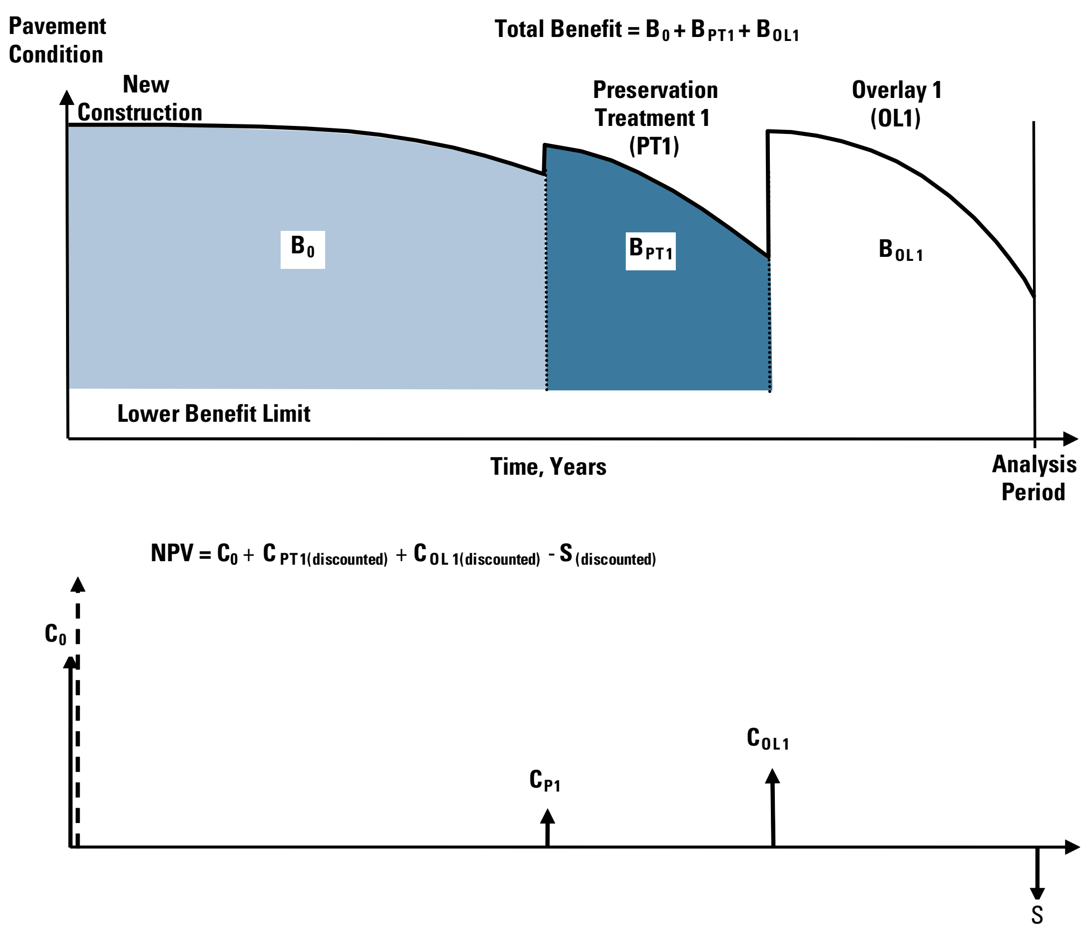

Figure 12.2. Illustration of benefits and costs associated with a pavement preservation treatment strategy (adapted from Peshkin et al. [2011])

area-under-the-performance-curve approach. A treatment strategy with a greater area under the curve yields more benefit through higher levels of condition or serviceability/smoothness provided to the highway users. More detailed information on computing pavement performance benefits using the area-under-the-performance-curve approach is available in an NCHRP report (Peshkin, Hoerner, and Zimmerman 2004).

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 preservation and rehabilitation treatments as those used in computing benefits. A discount rate is used to convert the costs of the future projected preservation and rehabilitation treatments (and any salvage value at the end of the analysis period) to present-day costs. 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 treatment strategy. The bottom portion of Figure 12.2 illustrates the stream of costs included in the LCCA. These costs occur in accordance with the preservation and rehabilitation treatment timings established and used in the analysis of benefits. They 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 B/C ratio 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, the performance curves can be developed from available historical condition data, historical preservation and rehabilitation treatment records, or expert opinion. The strategy with the highest B/C ratio is deemed the most cost effective.

Although most state highway agencies have a standardized procedure for conducting LCCA, detailed information on all aspects of the 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 has produced a software program called RealCost that completely automates the LCCA methodology as it applies to pavements (FHWA 2004).

Step 7: Select the Preferred Treatment Strategy

Decision Factors

A detailed cost-effectiveness analysis can be one part of the decision-making process, but by itself it does not necessarily identify the most optimal alternative. The highest B/C ratio option may not be practical when other considerations, such as available budgets, network priorities, environmental factors, and agency and customer preferences, are taken into account. 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 engineering needs 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 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 factors applicable to a given highway 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 the different factors and identifying the preferred strategy is through a strategy decision matrix (Peshkin et al. 2011). In a strategy decision matrix, various selection factors are identified for consideration and each factor is assigned a weighting. The weightings are then multiplied by rating scores given to each strategy, based on how well it satisfies each of the selection factors. The weighted scores of each strategy are then summed and compared with the weighted scores of the other strategies. The one with the highest score is then recognized as the preferred strategy. Illustrative examples of the decision matrix approach can be found in several references (Hallin et al. 2011; Peshkin et al. 2011).

Sustainability Considerations

As described in Chapter 1, sustainability considerations are being included by a growing number of 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 highway agency. As a whole, sustainability simply means providing good engineering.

Pavement preservation is inherently a sustainable activity, in that it employs low-cost treatments to prolong or extend the life of the pavement. In doing so, it helps in delaying major rehabilitation activities and thereby conserves energy and virgin materials while reducing greenhouse gas emissions by using lower-cost and low-environmental-impact techniques to maintain roads in good condition. Furthermore, well-maintained pavements provide a smoother, safer, and quieter traveling surface over a significant portion of their life, resulting in higher vehicle fuel efficiencies, reduced crash rates, and lower noise impacts on surrounding communities, which also positively contribute to their 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 summary is presented in an FHWA document (FHWA 2013). 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.

4. Summary

This chapter describes several basic steps that can be used to determine the most appropriate treatment strategy for a given concrete pavement project. The process begins with conducting a pavement evaluation and determining the causes of any observed distress. Next, treatments that address the identified deficiencies are selected (in a logical sequence to maximize the effectiveness of all the treatments) and filtered using any outside constraints that have been identified. After applying any outside constraints, feasible treatment strategies (i.e., combinations of treatments) are determined and a cost-effectiveness analysis is conducted for each strategy, whereby the benefits and costs associated with applying the treatments over a long analysis period are computed. Finally, the appropriate strategy is selected using a strategy decision matrix that systematically and rationally considers the results of the cost-effectiveness analysis and other important economic and noneconomic factors.

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