STRATEGIES FOR CONCRETE PAVEMENT PRESERVATION

Interim Report

FHWA-HIF-18-025

Prepared For:

Federal Highway Administration
Office of Preconstruction, Construction and Pavements
1200 New Jersey Avenue SE
Washington, DC 20590

February 2019

U.S. Department of Transportation
Federal Highway Administration

Notice

Quality Assurance Statement

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-HIF-18-025	None.	None.

4. Title and Subtitle	5. Report Date
Strategies for Concrete Pavement Preservation	February 2019
	6. Performing Organization Code None.
7. Author(s) Tom Van Dam, Kurt Smith, Mark Snyder, Prashant Ram, and Nicole Dufalla	8. Performing Organization Report No. None.
9. Performing Organization Name and Address Applied Pavement Technology, Inc. 115 West Main Street, Suite 400 Urbana, IL 61801	10. Work Unit No. (TRAIS) None.
	11. Contract or Grant No. DTFH61-14-D-00006/0001
12. Sponsoring Agency Name and Address Federal Highway Administration 1200 New Jersey Avenue, SE Washington, DC. 20590	13. Type of Report and Period Covered Interim Report July 2017 – February 2019
	14. Sponsoring Agency Code None.

15. Supplementary Notes

FHWA Contracting Officer's Representative: Tom Yu

16. Abstract

The use of the term "preservation" for concrete pavements is somewhat of a misnomer in that many preservation treatments are actually stop-gap measures used to address visible signs of deterioration in the existing pavement. The treatments address the symptoms of the problem but may not directly address the underlying causes of the distress, which can progress until requiring even more significant rehabilitation, or perhaps even reconstruction.

This report redefines the term concrete pavement preservation as "preserving the existing concrete pavement structure to extend its service life for as long as possible, by arresting, greatly diminishing, or avoiding the pavement deterioration process." This can be achieved through three fundamental approaches: (a) designing and constructing pavements that remain structurally adequate and relatively distress-free throughout their service lives (i.e., using long-life concrete pavement), (b) using asphalt or concrete overlays as preservation treatments to maintain the functional performance of the pavement, and (c) maintaining the serviceability of the pavement using concrete pavement restoration (CPR) treatments.

This report reviews the primary factors affecting concrete pavement performance and strategies for concrete pavement preservation. A state-of-the-practice review on the approaches for evaluating the condition of concrete pavements that will help in developing long-term concrete pavement preservation strategies and the engineering economic analysis techniques that can be employed to evaluate the cost-effectiveness of various preservation strategies is also included.

17. Key Words	18. Distribution Statement
concrete pavements, pavement preservation, preservation strategies, long-life concrete pavements, concrete overlays, concrete pavement restoration, economic analysis	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161.

19. Security Classify. (of this report)	20. Security Classify. (of this page)	21. No of Pages	22. Price
Unclassified	Unclassified	79	None.

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		
in²	square inches	645.2	square millimeters	mm²
ft²	square feet	0.093	square meters	m²
yd²	square yard	0.836	square meters	m²
ac	acres	0.405	hectares	ha
mi²	square miles	2.59	square kilometers	km²
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft³	cubic feet	0.028	cubic meters	m³
yd³	cubic yards	0.765	cubic meters	m³
		NOTE: volumes greater than 1000 L shall be shown in m³		
		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 degrees)		
°F	Fahrenheit	5 (F-32)/9	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m²	cd/m²
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in²	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		
mm²	square millimeters	0.0016	square inches	in²
m²	square meters	10.764	square feet	ft²
m²	square meters	1.195	square yards	yd²
ha	hectares	2.47	acres	ac
km²	square kilometers	0.386	square miles	mi²
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m³	cubic meters	35.314	cubic feet	ft³
m³	cubic meters	1.307	cubic yards	yd³
		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 degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m²	candela/m²	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/in²

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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CHAPTER 1. INTRODUCTION

Background

Pavement preservation is a recognized strategy to cost-effectively maintain the serviceability of a pavement and extend its life. According to the Federal Highway Administration (FHWA) Pavement Preservation Expert Task Group (ETG), and as defined in the *Moving Ahead for Progress in the 21st Century Act* (MAP-21), pavement preservation employs a network-level, long-term strategy that enhances pavement performance by using an integrated, cost-effective set of practices (Geiger 2005). As shown in figure 1-1, this can be accomplished through preventive maintenance and minor rehabilitation activities that are designed to restore pavement functionality and extend pavement life without necessarily increasing structural capacity (Smith et al. 2014). However, it is recognized that the application of these preventive maintenance and minor rehabilitation treatments to pavements undergoing structural deterioration or materials degradation is, at best, a stop-gap measure that is merely delaying their ultimate failure. It is more desirable to apply preventive maintenance and preservation treatments to pavements that are (and are expected to remain) structurally adequate and durable.

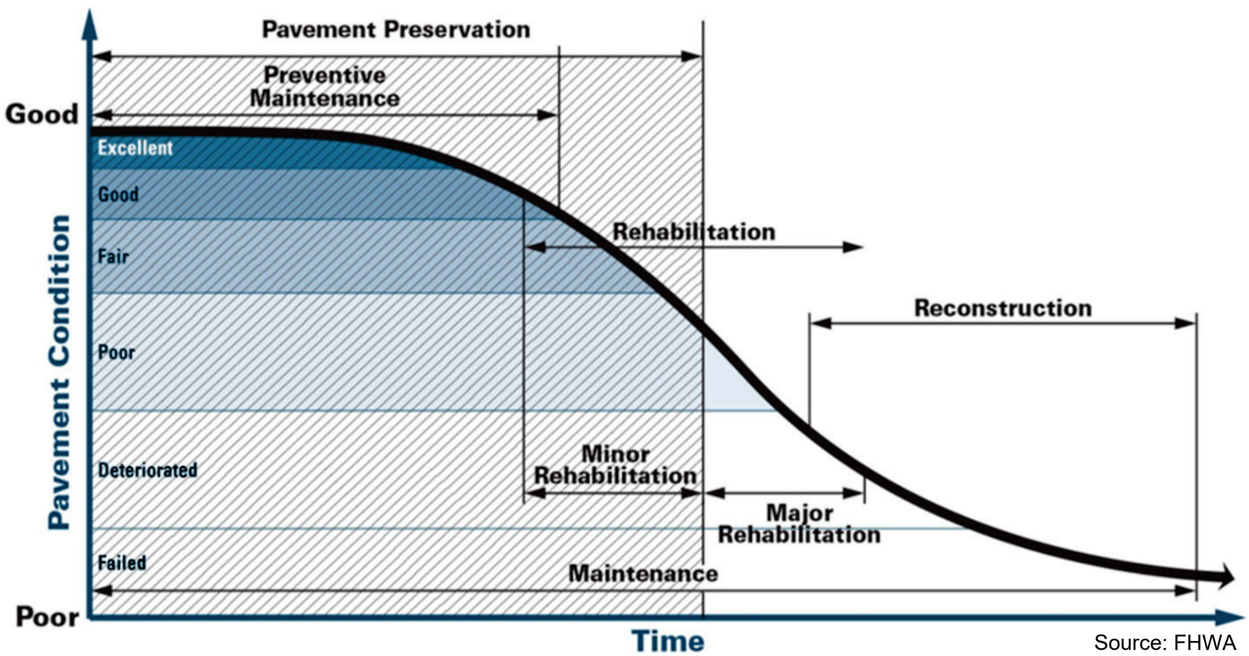

Figure 1-1. Pavement preservation activities and pavement condition (Smith et al. 2014).

Long service life is a hallmark of concrete pavements, and it is not uncommon for them to provide outstanding performance for 40 or more years. Such extended performance periods are easily achievable by concrete pavements when they employ durable materials and are properly designed and constructed. Throughout their service lives, these pavements will not require any significant rehabilitation other than the timely application of appropriate preservation treatments to maintain their functionality (e.g., smoothness, friction, quietness). When applied to the right concrete pavement at the right time, appropriate preservation strategies can be extremely successful at cost-effectively maintaining functionality and extending service life for 50 years or more.

Unfortunately, it is common today to design concrete pavements with the expectation that they will develop structural distress (e.g., slab cracking and joint faulting) within a relatively short time frame (e.g., 20 years), triggering the need for full-depth repairs, diamond grinding, slab stabilization, and other treatments to maintain their serviceability. Thus, the application of maintenance treatments and minor rehabilitation are not used to “preserve” the pavement structure, but instead are applied to concrete pavements that are already suffering structural distresses and other deficiencies to extend the service life for a relatively short amount of time. Although such treatments restore functionality in the short-term, they often do not address the underlying causes of the problems, so the performance of the pavement continues a downward spiral toward inevitable major rehabilitation or reconstruction.

Defining Concrete Pavement Preservation

In light of the above discussion, it is appropriate to reconsider the term “concrete pavement preservation” and define it as follows:

> *A strategy of extending concrete pavement service life for as long as possible by arresting, greatly diminishing, or avoiding pavement deterioration processes.*

This strategy can be achieved by:

- Designing and constructing concrete pavement that is durable and remains structurally adequate and relatively distress-free throughout a long service life (i.e., design and construct long-life concrete pavement, LLCP).
- Using overlays (asphalt or concrete) as a preservation treatment to maintain the structural capacity and serviceability of the existing concrete pavement.
- Maintaining the serviceability of the existing concrete pavement using concrete pavement restoration (CPR) treatments.

More details on each of these approaches are described in the following sections.

Designing and Constructing Long-Life Concrete Pavements

Concrete pavements that are ideal candidates for preservation are those that possess the ability to remain in service with little to no structural or materials-related distress for an extended performance period. Under SHRP2 Project R23, long-life pavements were defined as those providing 50 or more years of service without requiring major rehabilitation (Jackson, Puccinelli, and Mahoney 2014). Such long-life pavements are excellent candidates for preservation, as the basic elements of design, materials selection, and construction are such that major structural or materials failures are substantially deferred, allowing relatively non-invasive preservation techniques to be utilized to maintain functionality while effectively delaying the development of significant structural deterioration.

Although it is recognized that the construction of LLCPs typically comes at a higher initial cost, a life-cycle perspective reveals that reductions in future maintenance and rehabilitation costs, as well as reductions in user costs, can more than offset that increased initial investment. Thus, LLCP are considered a cost-effective alternative for most cases.

Using Overlays as a Preservation Treatment

Existing concrete pavements are commonly overlaid with hot-mix asphalt (HMA) or concrete as part of rehabilitation, and these are effective in restoring functionality (e.g., smoothness, skid resistance, reduced tire-pavement noise emissions) and in adding structural capacity. The approach supported in this project is a bit different, as it aims to establish guidance on the use of overlays as a preservation strategy to create a composite pavement with extended life. Under this scenario, the overlay will prevent or arrest the development of distress in the underlying concrete pavement, thereby preserving the overall pavement structure.

The use of overlays as a concrete pavement preservation strategy is not a common practice, but has been used on some projects. For example, several states (including Arizona, Texas, and now Nevada for the I-15 reconstruction as part of Project NEON) are using relatively thin rubberized asphalt overlays as part of new construction to provide desirable functional surface attributes to the concrete pavement (the primary focus is on noise reduction and demarcation of painted lane markings). The work conducted under SHRP 2 Project R21 demonstrated the effectiveness of this strategy beyond purely functional benefits in terms of constructability and rapid renewal (Rao et al. 2013). This work considered the application of conventional HMA and stone matrix asphalt (SMA) overlays, as well as porous HMA, asphalt rubber friction course (ARFC), and ultra-thin bonded wearing courses over new concrete pavements. Although such overlays do not significantly contribute to the pavement structure, they can have a mitigating effect on climatic impacts by reducing both the temperature and moisture gradients in the underlying concrete pavement. This, in turn, reduces critical pavement stresses and can improve ride quality over extended periods of time.

With regards to unbonded concrete overlays of concrete, most have been used to restore structural capacity to highly distressed concrete pavements, but there is no reason that they could not be used as a preservation strategy on a concrete pavement that is currently in good condition. The use of thin bonded concrete overlays of concrete have also demonstrated good success in correcting surface or structural deficiencies on many projects but there have also been some notable failures when applied to pavements in deteriorated condition or when effective bond was not achieved between the overlay and the underlying concrete pavement.

Maintaining Serviceability with CPR

Concrete pavement restoration is what is traditionally thought of as preservation. The techniques used (slab stabilization/slab jacking, partial-depth repair, full-depth repair, retrofitted edge drains, dowel bar retrofit, diamond grinding and grooving, and joint resealing and crack sealing) are well-documented in the literature (e.g., Peshkin et al. 2011; Smith et al. 2014). Smith et al. (2014) also discuss preventive maintenance and pavement preservation concepts and how the specific techniques can be combined to restore the functional and, to some degree, structural characteristics of the concrete pavement.

CPR can be the most economical solution for long-term concrete pavement preservation if the existing pavement is in sound structural condition and free of materials-related distresses (MRD), such as D-cracking or alkali-silica reactivity (ASR). If this is the case, non-invasive CPR preservation strategies can often be employed with minimal disruption to traffic to maintain pavement functionality for 50 years or more from the time of original construction.

However, when a concrete pavement is undergoing structural deterioration that cannot be slowed, CPR may not be the best strategy; a global treatment (such as a structural overlay or reconstruction) may provide the most cost-effective, long-term preservation approach. In such cases, the use of CPR may be appropriate as a "stop-gap" measure to temporarily maintain serviceability until the long-term strategy can be developed and implemented.

Project Objective

In light of the above discussion, the objective of this project is to develop guidelines for concrete pavement preservation with a focus on the long-term preservation of the existing concrete pavement structure. The emphasis is on developing life-cycle management strategies for concrete pavements that address the underlying causes of problems rather than just the symptoms, which are typically lag indicators of pavement performance. The guidelines ***will not*** provide detailed descriptions of specific treatment types, application procedures, or their benefits, as these are already well-documented in the literature.

Report Organization

This report consists of six chapters, including this introductory chapter. Brief descriptions of the chapters are provided below:

- **Chapter 2. Concrete Pavement Preservation Concepts** – This chapter covers fundamental concrete pavement preservation concepts and discusses the primary factors affecting concrete pavement performance. It also describes the capabilities and functions of some common concrete pavement preservation treatments, and presents considerations for designing new concrete pavements that can be preserved over long time periods.
- **Chapter 3. Evaluation of Existing Concrete Pavements** – This chapter summarizes the current state of the practice for the evaluation of structural adequacy, materials durability, and functional adequacy of concrete pavements.
- **Chapter 4. Strategies for Concrete Pavement Preservation** – Chapter 4 builds on the three primary strategies for concrete pavement preservation introduced in this chapter: long-life concrete pavements, overlays, and concrete pavement restoration. A summary of the available literature and the state of the practice of each of these three approaches is presented.
- **Chapter 5. Engineering Economic Analysis and Concepts for Strategy Selection** – This chapter reviews the economic analysis techniques that can be used to evaluate the impact of various concrete pavement preservation strategies. Literature highlighting the use of an alternate economic analysis approach (using the cost per lane-mile per year metric) is reviewed and summarized.
- **Chapter 6. Summary** – This chapter presents a brief summary of the key points raised in the report.
- **Appendix A. Bibliography** – A bibliographic listing of pertinent documents examined during the literature review is presented.

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CHAPTER 2. CONCRETE PAVEMENT PRESERVATION CONCEPTS

This chapter reviews the primary factors affecting concrete pavement performance and provides brief discussions on the benefits of pavement preservation; this includes a description of the capabilities and functions of common concrete pavement preservation treatments. A summary of the key considerations that go into designing new concrete pavements suitable to long-term preservation is introduced, with more details provided in chapters 3 and 4.

Key Factors Affecting Concrete Pavement Performance

Before discussing the need for pavement preservation, it is important to understand some of the key factors that drive concrete pavement performance. These include (but are not limited to): (a) design features, including structural design, joint load transfer, foundation support, and drainage; (b) durability of materials; and (c) construction techniques.

Design Features

Structural Design

An effective structural pavement design, including the integration of appropriate layer thicknesses, panel dimensions, and load transfer systems, helps reduce the risk of structural distresses that develop under traffic loadings and environmental forces. Mechanistic-empirical (M-E) pavement design methods can be used to predict specific distress types as a function of time, traffic and specific site conditions and, therefore, can develop pavement designs to meet site-specific conditions. M-E design procedures use mechanistic pavement responses (critical stresses, strains, and deflections), relate those responses to performance indicators (cracking, faulting, and roughness), and calibrate the performance indicators against actual field data to produce more accurate predictions. The designs that meet the specified performance criteria at a chosen level of reliability are considered feasible and can be further evaluated in terms of their life-cycle costs (LCC) or environmental impacts (NCHRP 2004).

Load Transfer Systems

An effective load transfer system is critical to the performance of jointed concrete pavements (JCP), especially on heavily-trafficked roadways. Joint load transfer refers to the ability of a joint to transfer traffic loading from one slab to the next and is an important design element for the reduction of pumping and faulting. It is accomplished through one or both of the following mechanisms:

1. Aggregate interlock, which refers to the mechanical interactions of aggregate particles at the abutting joint faces. This method is dependent on several factors, including the gradation, hardness and angularity of aggregates, concrete mixture proportions, concrete strength at the time of joint development, orientation of crack face, and joint width at the fractured concrete face. The joint width may be considered to be the most critical factor since the degree of interlock and shear capacity decreases significantly as joint width increases above 0.03 inches (0.8 mm) (FHWA 2018). Aggregate interlock is the primary means for load transfer for tied joints, and for transverse cracks in continuously reinforced concrete pavement (CRCP).
2. Mechanical load transfer devices (most commonly round, smooth, solid steel dowel bars) are placed across joints at the mid-depth of the slab. This method is not only highly effective at transferring load from one side of the joint to the next (which reduces pumping and faulting), but it also reduces tensile stresses in the slab corners (which can

prevent corner breaks). The smooth dowels do not restrict horizontal joint movement associated with the longitudinal expansion and contraction of the slabs caused by daily and seasonal changes in temperature and moisture (FHWA 2018).

Foundation Support

The design and construction of a robust foundation is essential to the long-term performance of any pavement structure. A foundation can be considered to consist of all layers that provide a platform for the riding surface. For concrete pavements, the foundation typically includes one or more base layers on top of the subgrade soil. Proper care and attention should be paid to the design and construction of the subgrade and base layers to ensure the effective structural capacity, stability, uniformity, durability, and smoothness of the concrete pavement over its service life (ACPA 2007).

Paving concrete typically has a 28-day flexural strength ranging from 550 to 750 lb/in^2 (3.8 to 5.2 MPa) or greater, and an elastic modulus ranging from about 4 to 6 million lb/in^2 (28,000 to 41,000 MPa), which helps to provide a high degree of rigidity. This rigidity enables concrete pavements to distribute loads over large areas of the supporting layers, meaning that the stresses transmitted to the underlying foundation materials are relatively low. Overall, it is the uniformity of support and not a high foundation strength that is important to concrete pavement performance (ACPA 2007).

Drainage

Drainage is one of the most important factors in pavement design. Water enters the pavement structure from above by infiltrating through cracks, joints, pavement surfaces, and shoulders, or from beneath as groundwater from a high water table, aquifers, and localized springs. When water is trapped within the pavement structure due to inadequate drainage, it often reduces the strength of unbound pavement layers and subgrade; this generates high hydrodynamic pressures under traffic loading that may pump out the fine material from under the pavement, resulting in loss of support. Drainage-related distresses in concrete pavements include pumping, faulting, corner breaks, punchouts, slab warping, and D-cracking (for freeze-thaw-susceptible aggregates).

Over the years, several different approaches have been used to try to reduce the adverse effects of moisture on the performance of pavements, including sealing the pavement joints, using moisture-insensitive materials (e.g., stabilized bases), and providing positive subsurface drainage to remove the water within the pavement. Effective designs may incorporate portions of each of these approaches.

Durability of Materials

Concrete pavement structural designs invariably assume that the materials used in the concrete are durable. However, unlike mechanical properties of concrete (e.g., strength), durability is not an inherent material property but rather a set of material properties that are required for the concrete to withstand the environment in which it serves (TRB 2013). Thus, material properties and environmental factors must be considered in concert when specifying and constructing durable concrete pavements (Van Dam 2016). Durability-related distresses such as D-cracking, deicer scaling, ASR, and sulfate attack could develop if not properly accounted for during the mix design process, and can result in premature failures even when the concrete pavement is structurally adequate.

Construction Techniques

Broadly speaking, the construction of concrete pavements involves the following general activities:

- Subgrade preparation.
- Hauling, placement, and compaction of pavement base and subbase layers.
- Proportioning and mixing concrete materials.
- Placement of embedded steel.
- Hauling and placement of the concrete materials.
- Finishing, texturing, and curing of the concrete pavement.

Achieving long service life is strongly impacted by the quality of the constructed pavement system. The potential gains realized through pavement design optimization and the use of durable materials and concrete mixtures can be nullified by poor construction quality and improper construction techniques.

Why is Preservation Important?

As described in Chapter 1, pavement preservation is a proactive and cost-effective approach for managing pavement assets. By methodically applying pavement preservation concepts, a number of significant benefits can be realized, including (Smith et al. 2014):

- Improved Pavement Condition – Effective preservation programs can help maintain pavements in good condition for longer periods of time, thereby delaying the need for major rehabilitation and reconstruction activities.
- Increased Safety – Pavements in better overall conditions are generally smoother and contribute to safer operating environments for the traveling public.
- Cost Savings – Cost savings are realized in the form of: (a) extended service lives requiring fewer and less expensive treatments; (b) decreased user costs resulting from reduced traffic delays; (c) lower vehicle operating costs (due to smoother roads), and (d) lower crash-related costs.
- Higher Customer Satisfaction – Preservation produces smoother and safer roads and requires shorter lane closure durations and less invasive procedures, all of which contribute to a higher level of satisfaction for the traveling public.

Concrete Pavement Preservation Treatments

Table 2-1 provides brief descriptions of common concrete pavement preservation treatments. These treatments use different types of materials (or no materials at all, in some cases) and can be used to address either localized deficiencies or more widespread issues.

Table 2-1. Common concrete pavement preservation treatments (adapted from Smith et al. 2014 and Peshkin et al. 2011).

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 (PDR)	Removal of small, shallow (upper one-third to one-half of the slab) areas of deteriorated concrete and subsequent replacement with a cementitious or proprietary repair material.
Full-Depth Repair (FDR)	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.
Dowel Bar Retrofit (DBR)	Placement of dowel bars across joints or cracks in an existing concrete pavement to restore load transfer.
Cross Stitching	Insertion of deformed bars into holes drilled at an angle through longitudinal cracks (or, in some cases, longitudinal joints) in an existing concrete pavement.
Slot Stitching	Grouting of a deformed bar into slots cut across a longitudinal joint or crack.
Diamond Grinding	Removal of a thin layer of concrete (typically 0.12 to 0.25 inches [3 to 6 mm] thick) 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 joint sealant materials, refacing and cleaning the joint sidewalls, and installing new material (liquid sealant and backer rod or preformed compression seal).
Crack Sealing	Sawing, cleaning, and sealing cracks (typically transverse, longitudinal, and corner-break cracks wider than 0.125 inches [3 mm]) in concrete pavement using high-quality sealant materials.

Capabilities and Functions of Concrete Pavement Preservation Treatments

For concrete pavement structures, the selection of appropriate preservation treatments is largely driven by the distresses and unique conditions of the pavement. Each treatment is directed at one or two primary deficiencies and, while they can be used as stand-alone treatments in many cases, a pavement preservation project often will employ several treatments in order to restore the serviceability of the pavement to a satisfactory level and minimize the potential for the distresses to reoccur. Table 2-2 summarizes the primary capabilities and functions of the common preservation techniques, while table 2-3 presents the general applicability of the various treatments based on the distresses in the existing pavement.

Table 2-2. Primary capabilities and functions of concrete pavement preservation treatments (adapted from Peshkin et al. 2011).

Treatment	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 or Reduce Hydro-planing	Improve Profile (Lateral Surface Drainage and Ride)	Improve Texture for Noise
Slab Stabilization		X			X			
Slab Jacking		X					X	
Partial-Depth Repair (PDR)	X			X			X	
Full-Depth Repair (FDR)	X	X		X	X		X	
Dowel Bar Retrofit (DBR)					X		X	
Cross Stitching/ Slot Stitching					X		X	
Diamond Grinding					X	X	X	X
Diamond Grooving						X		
Joint Resealing	X			X	X			
Crack Sealing	X			X				

The use of the term "preservation" for many concrete pavement treatments has been somewhat of a misnomer because most concrete pavement "preservation" treatments (shown in table 2-1) are often stop-gap treatments that attempt to slow down the deterioration of the existing pavement and delay the end-of-life activities. These treatments are typically applied only after an observed indication of a potential problem, most often manifested as a distress on the pavement surface. The treatments are then used to address the symptoms of the problem (e.g., removal of joint faulting by diamond grinding) but do not aim to prevent the problem from occurring in the first place (e.g., erodible foundation materials, poor load transfer, free water in the base).

Table 2-3. Applicability of concrete pavement preservation treatments based on distresses observed (adapted from Hall et al. 2001).

Distress	Slab Stabilization	Slab Jacking	PDR	FDR	DBR	Cross Stitching/ Slot Stitching	Diamond Grinding	Diamond Grooving	Joint Resealing	Crack Sealing
Corner Breaks			X	X						X[a]
Linear Cracking				X		X[b]				X[a]
Punchouts				X						
D-Cracking				X[c]						
Alkali Aggregate Reaction (AAR)				X[c]						
Map Cracking, Crazing, Scaling			X							
Joint Seal Damage									X	
Joint Spalling			X	X						
Blowup				X						
Pumping	X				X	X				
Faulting					X		X			
Bumps, Settlement, Heaves		X		X			X			
Polishing/ Low Friction								X		

Note: Many of these treatments are commonly done in combination to fully address all 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.

Designing Pavements For Preservation

The fundamental shift in the pavement preservation philosophy in this study revolves around the premise that concrete pavement preservation begins with the initial design of the concrete pavement structure. The objective is to design and construct long-life concrete pavements that are structurally adequate and largely distress-free over their service lives, and then apply preservation strategies that maintain pavement functionality without compromising the structural capacity of the underlying pavement structure. In other words, design the pavement to perform instead of designing the pavement for a certain level of distress.

Some of the key considerations that go into designing pavements for preservation over a long service life (>50 years) include:

- Design and construct effective load transfer systems, especially for pavements exposed to high volumes of heavy (truck) traffic.
- Use durable concrete mixtures and mixture components along with a robust QA program.
- Integrate effective structural layer thicknesses, panel dimensions, load transfer systems, and drainage systems such that the risk for structural distress through typical failure mechanisms is virtually eliminated (or significantly reduced). It should be noted that certain structural aspects can be provided in initial design/construction (including the use of composite pavement structures) or through staged construction (i.e., through the use of structural overlays later in life).
- Use high-performance foundation systems. Several concrete pavement failure mechanisms, such as pumping, faulting, longitudinal cracking, and settlements, can be traced at least in part to inadequacies of the pavement support system.
- Monitor pavement performance to identify lead indicators of pavement performance that can help to determine the appropriate type of preservation treatment that can be applied before the onset of distresses.
- Ensure adequate ride quality, safe levels of friction, and acceptable levels of tire-pavement noise throughout the service life of the pavement.
- Recognize that pavements may reach a condition where maintaining an adequate level of serviceability is no longer possible. Agencies can make this determination based on the remaining service life and overall economic considerations.

It is recognized that a vast majority of in-service concrete pavements have not been designed as long-life concrete pavements; thus, they may require a different approach for preserving and extending the service lives through a combination of overlays and CPR treatments. In some cases, complete removal and replacement of the existing pavement structure may be the most economical and environmentally sustainable solution from a life-cycle perspective.

The literature and state-of-the-practice review summarized in chapters 3, 4, and 5 document the information necessary for the development of this new approach for concrete pavement preservation. The focus of the review is to identify suitable approaches for pavements that are categorized as follows:

- Category 0 – Concrete pavements with no apparent or impending distresses (e.g., recently constructed pavements). These pavements are candidates for monitoring to identify the initiation and subsequent progression of structural distresses (in addition to functional distresses).
- Category 1 – Pavements that are not experiencing structural deterioration but require functional preservation.
- Category 2 – Pavements in which structural improvements (e.g., overlays) have been used to prevent or arrest the development of structural deterioration.
- Category 3 – Pavements in which the progression of structural deterioration is continuous and treatments are being applied only to maintain some acceptable level of serviceability until a major structural rehabilitation or reconstruction can be performed.

The next chapter (Chapter 3) focuses on techniques for evaluating the condition of existing concrete pavements that will then help in developing strategies for concrete pavement preservation (discussed in Chapter 4). Chapter 5 summarizes the economic analysis techniques that can be used to evaluate the cost-effectiveness of various pavement preservation strategies and help identify the life-cycle management strategies with the lowest practical life-cycle costs.

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CHAPTER 3. EVALUATION OF EXISTING CONCRETE PAVEMENT

Methods for evaluating the condition of an existing concrete pavement can be broadly divided into three categories: (a) evaluating structural adequacy, (b) assessing the durability of the pavement materials, and (c) assessing the functional adequacy of the pavement. The structural adequacy of the pavement describes the ability of the pavement to withstand repeated structural loading and includes measures of pavement layer thicknesses, layer strength and stiffness, degree and uniformity of support (including drainage), and joint load transfer efficiency. The durability of the concrete materials describes the ability of the materials used in the pavement to withstand environmental deterioration, remaining intact in the service environment. Finally, functional adequacy describes attributes of the pavement surface that includes measures of smoothness, noise, and safety (e.g., surface friction and hydroplaning potential).

Smith et al. (2014) summarized recommendations for pavement evaluation based on the pavement treatment being considered, as shown in table 3-1. Many of the data items recommended for collection can be categorized into one of the three categories listed above. Data items used to characterize structural adequacy include material properties, subgrade information, traffic loading and volume, distress, nondestructive testing (NDT), destructive testing, and drainage. Items that characterize concrete durability include material properties, climate, subgrade information, and destructive testing information. Data items associated with the functional adequacy of the pavement include friction and pavement profile (roughness), although many agencies also measure tire-pavement noise emissions.

Structural Adequacy

As described previously, the structural adequacy of the pavement describes the ability of the pavement to withstand repeated structural loadings over its design life and is indicated by such measures of the pavement layer thicknesses, layer strength and stiffness, and degree and uniformity of support, and joint load transfer (preferably quantified in terms of differential deflection). Structural inadequacy can be inferred by conducting a distress survey that summarizes the number and severity of structural-related distresses (e.g., transverse cracking, corner breaks); however, the method of collecting and categorizing this distress data varies widely across state highway agencies (SHAs). Furthermore, many SHAs consider deflection-based NDT to be a primary indicator of the current structural capacity and adequacy of a roadway.

Another key aspect of structural adequacy is the thickness of the pavement layers, which is most often determined from as-built plans or through coring and subsurface boring but can also be indicated by NDT methods such as ground-penetrating radar (GPR) and seismic methods. The methods of assessing structural adequacy discussed herein are based mainly on evaluations of data obtained from pavement distress surveys and deflection-based NDT methods, specifically through the use of the falling weight deflectometer (FWD). In addition, coring and boring are discussed along with other NDT methods used to characterize pavement layer parameters.

Table 3-1. Suggested data collection needs for concrete pavement treatment alternatives (Smith et al. 2014).

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/ Slot stitching
Pavement Design	X	X	X	X		X	X	X	X	X	X	X
Original Construction Data			(X)	(X)	(X)			(X)	(X)	(X)	(X)	(X)
Age	(X)	(X)	(X)	(X)	(X)			(X)				
Material Properties	(X)	(X)	X	X	X	(X)	(X)	X				
Subgrade			X			(X)	(X)	X	X	X		
Climate			X			X	X	X	X	X		
Traffic Loading and Volumes	X	X	X	X	X	(X)	(X)	X	(X)	(X)	X	X
Distress	X	X	X	X	X	X	X	X	X	X	X	X
Friction			(X)	(X)	(X)							
Crashes			(X)	(X)	(X)							
Potential NDT	(X)		X			X	X				X	(X)
Potential Destructive Testing/Sampling	X	X	X	(X)	(X)	(X)	(X)	X			(X)	(X)
Roughness			(X)	(X)	(X)	(X)	(X)					
Surface Profile			(X)	X	X	(X)	(X)					
Drainage	X		X	X	X	X		X	X			
Previous Maintenance	(X)	(X)	(X)	(X)	(X)	(X)		(X)	(X)	(X)		
Bridge Transitions	X	X	(X)						X			
Utilities	X		X			(X)	(X)	(X)				
Traffic Control Options	X	X	X	X	X	X	X	X	X	X	X	X
Vertical Clearances			X									
Geometrics			X									

X indicates required testing while (X) indicates recommended testing

Distress Surveys

SHAs generally collect pavement distress data to assess the condition of their pavement network. Those data are then used to develop a condition rating metric that quantifies the condition of their pavements and can be used to trigger rehabilitation needs. SHAs typically outline a testing schedule for distress surveys including the frequency of distress rating and the quality assurance required for this data.

Distress survey data describe the type, extent, and severity of distresses observed on the pavement surface. Pavement distress data are commonly used to "manage" pavement assets, where distress data are collected over time using an established methodology. In this way, distress progression can be tracked and preservation and repair appropriately scheduled. The method of data collection and the definitions of the distresses vary widely across state and local agencies. Distress surveys can be conducted either manually by trained distress raters or by automated distress data collection equipment. Automated distress collection uses equipment that collects and interprets distress data automatically, or equipment that captures images and videos that are later analyzed manually by trained distress raters in the office.

A comparison of automated and manual data collection methods is provided in table 3-2. A number of different types of equipment are in use for automated data collection, including the Road Excellent Automatic Logging (REAL) system by PASCO (van-mounted equipment that collects both road profile and distress images), the ARAN (a high speed, multi-functional and diverse road/infrastructure data acquisition vehicle that is used by a number of SHAs and includes six video cameras and a smart bar for rutting determination in asphalt pavement), the Pavedex PAS-1 (an inspection vehicle containing five cameras and a digitized crack quantification system), and Pathway Systems, Inc. (a digital inspection vehicle [DIV] used by the Minnesota DOT). These systems continue to evolve.

Table 3-2. Comparison of automated and manual data collection (Attoh-Okine and Adarkwa 2013).

Parameter	Automated Data Collection	Manual Data Collection
Time	Reduces data collection times	Longer data collection times
Safety	Much safer means of collecting data	Personnel at risk collecting data
Objectivity	Objective measurements	Usually subjective since it is dependent on the experience level of the personnel
Cost	Very expensive equipment cost	Relatively less expensive
Data size	Vast amounts of data collected and stored depending on capacity of equipment	Agencies may only be able to collect smaller amounts of data at a time
Data handling	Not subject to transcript errors	Subject to transcript errors
Employers	Suitable in agencies seeking to downsize number of employees	Source of employment for rating staff
Coverage	May cover footprint of data collection vehicle. Multiple runs sometimes needed to cover entire road width	Inspectors can cover entire width of road section relatively easier.

Distress surveys capture a wide variety of distresses, including those linked primarily to structural adequacy. For JCP, pavement distresses linked to structural adequacy include corner breaks, longitudinal and transverse cracking, joint faulting, and pumping, and these are described below (AASHTO 1993; Miller and Bellinger 2014; NCHRP 2004):

- Corner Break – A diagonal crack separating a portion of the slab at the corner, intersecting the adjacent transverse and longitudinal joints at an approximately 45-degree angle is a corner break. The length of each side is from 1 ft (0.3 m) to one-half the slab width on each side of the corner. Corner cracks are most often linked to heavy traffic loading, where poor load transfer across the joint has resulted in loss of support, often in the presence of water. Temperature curling and/or moisture warping may also be contributing factors.
- Longitudinal and Transverse Cracking – Singular cracks (not part of a network of cracks) that are predominantly parallel to the pavement centerline (longitudinal cracking) or predominately perpendicular to the pavement center (transverse cracking). There are a number of causes that can contribute to longitudinal and transverse cracking, including both design and construction factors. Cracking of this type that occurs after the pavement

has been in service for some years likely has a structural component, where heavy traffic loading combined with poor load transfer, temperature curling, moisture warping, and possibly loss of support in the sublayers contribute to fatigue failure in the concrete.

- Faulting – Faulting is an elevation difference that develops across a joint or crack. It results from repeated heavy traffic loading, poor joint load transfer, and reduced support conditions. As heavy vehicles move across the joint, the differential slab movement causes rapid movement of water, resulting in the erosion of material from beneath the second slab (leave slab). This material is ejected backwards under the first slab (approach slab), raising it with respect to the second. Faulting will continue to develop over time if the contributing factors are not addressed, affecting ride quality and ultimately may result in corner breaks and slab cracking due to loss of support.
- Pumping – Pumping is the ejection of water and material from beneath the slab at joints and cracks, frequently observed as the deposition of fine material left on the pavement surface. Pumping is related to faulting, in which repeated heavy traffic loading, poor load transfer, and subsurface water result in the erosion and ejection of supporting materials onto the pavement surface.
- Patch Deterioration – Patches are a portion (> 0.1 yd^2 [0.08 m^2]), or all of the original concrete slab, that has been removed and replaced, or additional material applied to the pavement after original construction. If the patch suffers corner breaks, transverse or longitudinal cracking, faulting, pumping, or spalling, the distress is considered to be primarily due to heavy traffic loading.

CRCP designs share some of the same structural distresses as JCP (such as pumping and patch deterioration), with some additional distresses as follows (AASHTO 1993; Miller and Bellinger 2014; NCHRP 2004):

- Longitudinal Cracking – Singular cracks (not part of a network of cracks) that are predominantly parallel to the pavement centerline. There are a number of causes that can contribute to longitudinal cracking in a CRCP, including both design and construction factors. Cracking of this type that occurs after the pavement has been in service for some years is often considered to have a structural component, where heavy traffic loading combined with loss of load transfer at transverse cracks and possibly loss of support in the sublayers results in fatigue failure in the concrete due to bending. Under these circumstances, longitudinal cracking is viewed as a precursor to punchouts.
- Punchouts – The area enclosed by two closely spaced (usually less than 2 ft [0.6 m] apart) transverse cracks, a longitudinal crack, and the edge of the pavement or longitudinal joints. Also includes “Y” cracks that exhibit spalling, breakup, or faulting. Punchouts are considered a primary structural failure in CRCP.

The presence of these structural distresses is indicative of a structural inadequacy in the concrete pavement, often the result of poor support, weak or thin PCC layers, poor load transfer, and poor drainage. These distresses can provide valuable information for evaluating a pavement for possible preservation and rehabilitation treatments.

Nondestructive Testing

NDT methods are applied to quantify specific pavement properties without damaging the pavement. A range of NDT methods are available for assessing the pavement, with deflection-based methods widely used for assessing the structural adequacy and other methods (such as pulse velocity and impact echo) used to determine in situ strengths (Crawford 1997). A summary of selected NDT devices is provided in table 3-3.

Table 3-3. Comparison of selected NDT methods (adapted from Smith et al. 2014).

Type of NDT testing	Load Transfer Efficiency	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	Yes	Yes
MIT Scan 2-BT	No	Yes	No	No	No
MIT Scan T2	No	No	Yes	No	No

Each NDT device collects raw data that is then analyzed (either externally or internally by the device itself) to estimate the pavement characteristic of interest. For example, an FWD applied load induces deflections that are measured at the pavement surface through contact sensors. External analysis of the load and deflection data, in combination with layer thicknesses, can provide backcalculated stiffness estimates for each pavement layer. Load transfer efficiencies or differential deflections across joints and the presence of voids at slab corners can also be determined. The MIRA device processes data internally to determine slab thickness, depth to steel, and slab stiffness.

Despite the valuable information produced by NDT methods, they are not widely used for network-level pavement condition data collection due to their relatively high cost and the time required for this type of testing (Pierce et al. 2013). There are a number of NDT methods that have been developed to collect data at highway speeds including GPR and deflection but this type of service is still viewed as being cost-prohibitive at a network level. That said, NDT is widely used at the project level for evaluation and its use is expected to continue to grow. The following sections describe some of the characteristics of selected NDT methods.

FWD Testing

FWD testing is the most commonly used method of nondestructive testing across most SHAs. The FWD is most often trailer-mounted, as shown in figure 3-1, although truck-mounted versions are also available. The FWD operates by dropping a mass onto the pavement surface through a buffer and load plate, generating an impulse load. This is illustrated in figure 3-2. A series of deflection sensors, one located within the load plate and others at multiple locations emanating outward, measure the induced deflections. A typical sensor spacing is also shown in figure 3-2. Together, the sensors measure the deflection basin generated by the applied load as illustrated in figure 3-3.

Figure 3-1. Falling weight deflectometer behind tow vehicle.

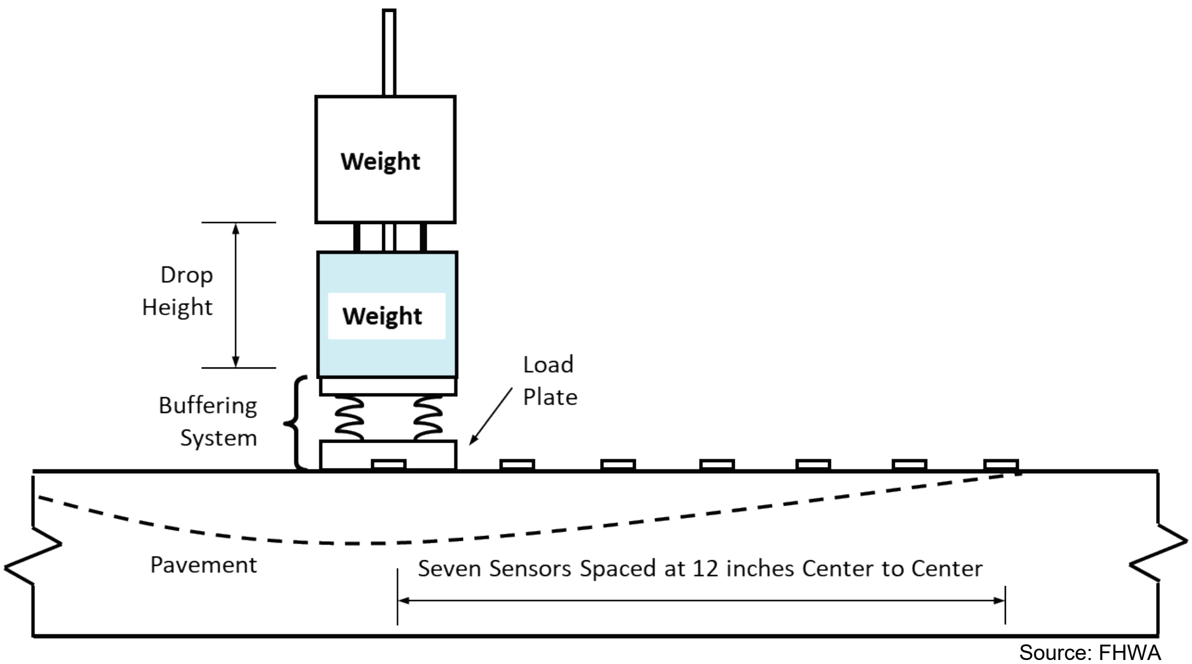

Figure 3-2. Schematic of FWD device (Smith et al. 2014).

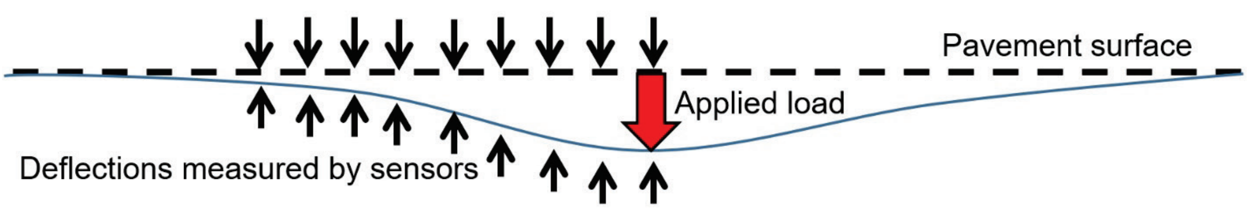

Figure 3-3. Typical FWD loading and measurement layout (adapted from TxDOT 2008).

It is most common to select an impulse load to simulate a typical moving truck wheel load (approximately 9,000 lbs [40 kN]). The magnitude of the impact load can be varied once the mass is selected by dropping it from different heights. The applied load is often varied below and above the target load during a test. For example, the Texas Department of Transportation typically uses four different load levels at each location in a series of drops in a test that typically lasts less than 2 minutes (TxDOT 2008).

Due to the relatively short time it takes to conduct a single nondestructive test at each location, it is not uncommon to test 200 to 400 locations with an FWD in a single day, depending on the type of testing (mid-panel testing vs. joint testing) and traffic control logistics. Data for project-level assessment are often collected at intervals of about 200 ft (60 m) and provide a valuable indication of structural variability that cannot be obtained from destructive testing. Several states, including Pennsylvania, Texas, and Minnesota, provide detailed testing requirements and recommendations for FWD testing.

The raw data collected by the FWD includes the magnitude of the impact load (collected by a load cell) and the surface deflection measurements. These data can be used to calculate a wide variety of properties relevant to evaluating the structural integrity of the concrete pavement including the stiffness of the concrete slab and degree of support, the load transfer efficiency and differential deflection across joints (to measure the effectiveness of the dowels or aggregate interlock), and evidence of loss of support (e.g., voids or similar subsurface structural variability that should be addressed prior to any rehabilitation treatments) (AASHTO 1993; TxDOT 2008; NCHRP 2004).

An appropriate testing plan must be selected based on the desired output. Testing plans vary based on the width of the type of pavement, the number of repeated passes in testing, and the desired information (for example, specific slab locations are required for calculating the load transfer efficiency or differential deflection between slabs). The FHWA Long Term Pavement Performance Program (LTPP) conducts FWD testing for a wide array of pavement experiments and has published a series of testing plans for various concrete pavements (Schmalzer 2006). A typical testing plan (load placement locations) for jointed plain concrete pavement (JPCP) is reproduced in figure 3-4. This figure shows that the combination of J4 and J5 can be used to calculate the load transfer efficiency or differential deflection between the two slabs from both directions. Measurement point J1 is taken in the exact middle of the slab and is often used for backcalculating slab stiffness and support, while J2 and J3 are taken on the edge of the slab to assess load transfer or differential deflection along the longitudinal joint (if the adjacent lane/shoulder is concrete) and for void detection. Details for conducting each type of analysis are presented in AASHTO (1993) and NCHRP (2004), as well as in several other sources.

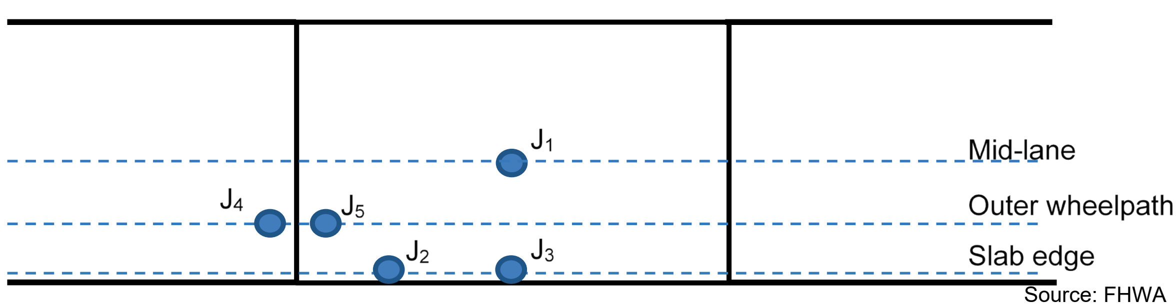

Figure 3-4. Recommended FWD testing plan for JPCP (Schmalzer 2006).

In the last several decades, work has progressed on the development of equipment capable of measuring deflections continuously along a project, at speeds ranging from about 5 mi/hr (3 km/hr) to near posted limits (Flintsch et al 2013). This allows for the entire length of the project to be investigated so that overall variability in deflection measurements can be captured while reducing traffic disruptions. However, to date, these devices have seen greater applicability on asphalt-surfaced pavements than on concrete pavements.

Other NDT Methods

In addition to deflection testing, other NDT methods are sometimes used. For example, ground penetrating radar (GPR) has been used for decades to characterize layer thicknesses (when the dielectric constants or permittivity of the individual layers are sufficiently different) and to determine the depth of embedded steel; standard GPR pavement testing procedures are described in AASHTO Standard R 37, *Standard Recommended Practice for the Application of Ground Penetrating Radar (GPR) to Highways*. By using GPR, the thicknesses of layers differentiated by different material types can be determined, which is an effective nondestructive alternative to coring. One major advantage of GPR is that it can be performed over an entire pavement project, but some limited coring is still required for calibration purposes. This allows for the observation of variations in thickness along the length of the pavement, which can be helpful in supporting backcalculation of FWD data and for dividing a long project into smaller sections for rehabilitation. And GPR can be used to discern structural measures of the base layers, such as void detection (Jung, Freeman, and Zollinger 2008). However, moisture levels in newly placed concrete pavements can attenuate the signal and make it difficult to develop reliable thickness readings at early ages (Ye and Tayabji 2009).

Magnetic imaging tomography (MIT) is becoming common as a quality assurance tool for new JCP construction. It has come into widespread use as a way of determining the location of the metallic dowel and tie bars in newly constructed concrete pavement, and a derivative approach uses a steel plate reflector placed on the base prior to paving to allow the determination of the slab thickness.

Ultrasonic tomography, performed using a multi-element ultrasonic imaging device (MIRA), has been used in pavement evaluation to accurately determine pavement thickness, the location and position of steel, and whether the sawed contraction joint has activated (Tran, Roesler, and Popovics 2017). Results from the MIRA device can help support the analysis of FWD data by verifying slab thicknesses.

Infrared thermography is used for detecting concrete defects (such as cracks, delamination, and concrete disintegration) in roadways and bridge structures, and has also seen use in identifying asphalt concrete segregation (Gucunski et al. 2013). However, for concrete pavements, the testing does not provide information on the depth of the flaw and the readings can be affected by surface anomalies and boundary conditions (Gucunski et al. 2013).

Drainage Surveys

Drainage is an important contributor to pavement performance. Poor drainage weakens the underlying pavement structure and contributes to the formation of distresses such as pumping, faulting, and corner breaks in JCP and punchouts in CRCP. Poor drainage in an existing pavement is not easy to correct but will continue to negatively influence pavement performance if not addressed.

Drainage surveys are typically conducted as part of a project-level distress survey. The first step is to determine if moisture-related distress is present and to what extent. For JCP, pumping, faulting, and corner breaks are all strong indicators that a drainage issue exists, as are punchouts in CRCP. The condition of ditches, drainage outlets (if present), and drainage inlets (if present) should be assessed during the visual survey. For outlets, evidence of free moisture flow should be sought as an indicator of potential blockage. The finding of voids at corner locations through the NDT evaluation is also an indicator of potential drainage issues.

More in-depth investigations should be conducted if the visual and/or NDT assessment indicates that drainage may be an issue. These investigations could include possible video inspection of the subsurface drainage system and destructive coring and sampling designed specifically to test the effectiveness of the subsurface drainage as well as to determine the moisture condition of extracted in-situ base, subbase, and subgrade.

Materials Durability

While measuring the structural integrity of a concrete pavement provides an indication of the potential pavement longevity, it is also imperative to consider the durability of the concrete materials. The presence or absence of MRD can factor into preservation decision-making. Materials durability can be tested primarily by two means: distress surveys for MRD and by acquiring samples for laboratory testing.

Materials-Related Distress Surveys

Different types of automated and manual distress surveys were previously discussed. However, in addition to the structural distresses that can be identified, a preliminary identification of MRD can also be obtained from either automatic or manual distress surveys. These distresses include (AASHTO 1993; Miller and Bellinger 2014; NCHRP 2004):

- Durability Cracking – Closely-spaced crescent-shaped hairline cracking pattern, concentrated in the vicinity of joints, cracks, and free-edges. Often associated with dark-colored staining and can lead to spalling. Caused by freeze-thaw deterioration of the coarse aggregate.
- Joint and Crack Spalling and Deterioration – Cracking, breaking, chipping, or fraying of the slab edges within 1 ft (0.3 m) from the joint or crack face. Can be caused by a number of factors, including poorly maintained sealant that has allowed for the infiltration of incompressible material into the joint, misaligned dowel bars, weak concrete at the joint, freeze-thaw deterioration of the aggregate or the hydrated cement paste, or some other durability issue.
- Polished Aggregate – Exposure of coarse aggregate on the pavement surface as a result of the wearing away of texture and surface mortar. Can occur due to heavy traffic loading abrading a concrete surface susceptible to abrasion, most often linked to poor wear resistance of the course aggregate. Can impact functionality by decreasing surface friction of the surface.
- Map Cracking Due to Alkali-Aggregate Reaction (AAR) – A network of cracks forming a pattern on the slab surface. Caused by a chemical reaction between the alkalis in the pore solution of the hydrated cement paste and either (a) reactive silica in the aggregate (known as alkali-silica reactivity, ASR) or (b) certain carbonate rocks (known as alkali-carbonate reactivity, ACR). The AAR mechanism is progressive in nature and can

contribute to disintegration and deterioration of the concrete, as well as to pressure-related effects such as spalling and blowups:

- Popouts – Small pieces of pavement broken loose from the surface, normally ranging in size from 1 to 4 inches (25 to 102 mm) in diameter, due to expansion of an underlying aggregate particle near the pavement surface. A broken piece of aggregate or aggregate socket is often visible at the bottom of the popout. Popouts can indicate poor aggregate freeze-thaw resistance, expansive aggregates, or may be an indicator of AAR.
- Blowups – A localized upward movement of the slab surface at a transverse joint or crack, often accompanied by shattering of the concrete in the immediate vicinity of the blowup. Blowups occur when expansion in the pavement cannot be accommodated at the joints. Most often occurring during hot weather events when improperly maintained joints cannot accommodate slab expansion. AAR may also be a cause of expansion that can lead to blowups.

Unlike some structural distresses whose direct cause can be addressed, many MRDs are inherent to the materials originally used in construction and therefore the effects can often only be mitigated through repair or replacement. For example, in the case of AAR, the only effective means of completely addressing this distress is to entirely remove the reactive aggregate, which is infeasible. Consequently, the continued impacts of MRD must be identified and mitigated through localized repair to maintain pavement serviceability until replacement is achievable. The only way to determine if an MRD is causing distress is through evaluation of extracted concrete cores in accordance with ASTM C856, *Standard Practice for Petrographic Examination of Hardened Concrete* using procedures such as those presented by Van Dam et al. (2002) and by Walker, Lane, and Stutzman (2006).

Field Sampling and Materials Testing

A key element of any evaluation is field sampling, which can establish pavement layer thicknesses and provide materials that can be tested to determine the relevant properties of the concrete and supporting layers. Pavement cores should be obtained in accordance with ASTM C42, *Standard Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete* or ASTM C823, *Standard Practice for Examination and Sampling of Hardened Concrete in Construction*. Van Dam et al. (2002) provides specific guidance for sampling and shipping concrete extracted from pavements that are potentially affected by MRD.

Following coring, additional field testing and sampling can be conducted on the underlying pavement layers. One test that has been found to be very useful is the dynamic cone penetrometer (DCP). The DCP is a handheld device in which a cone tipped rod is driven into the underlying untreated base, subbase, or soil by repeatedly dropping a 16.7 lb (7.6 kg) hammer from a fixed height of 22.6 inches (574 mm). Well-established correlations exist that relate the rate at which the cone penetrates the layer to the California Bearing Ratio (CBR). Base and soil materials can then be augured from the core hole for laboratory characterization. Alternatively, the soil can be tested and sampled *in situ* using split-spoon (split-barrel) or Shelby (Push) tubes.

Laboratory materials characterization is used to obtain properties of the materials extracted from the pavement. These include tests on the concrete and on the materials obtained from underlying bound and unbound layers. Concrete cores extracted from the concrete surface layer can provide insight into both structural and material characteristics. Structural characteristics that can be obtained from the cores include:

- Compressive Strength – In accordance with AASHTO T 22, *Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens*.
- Elastic Modulus – In accordance with ASTM C469, *Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression*.
- Indirect Tensile Strength – In accordance with ASTM C496, *Standard Test Method for Spitting Tensile Strength of Cylindrical Concrete Specimens*.
- Coefficient of Thermal Expansion – In accordance with AASHTO T 336, *Standard Method of Test for Coefficient of Thermal Expansion of Hydraulic Cement Concrete*.

Concrete can be evaluated for the presence of various MRDs using ASTM C856, *Standard Practice for the Petrographic Evaluation of Hardened Concrete*. This practice describes the microscopic evaluation of concrete through evaluation of the hardened cementitious paste, aggregates, air void system, and other attributes of the concrete in order to assess the general quality of the concrete, to determine causes of cracking, and to identify the presence of materials-related distresses. Additional testing that could be considered includes the determination of chloride penetration (ASTM C1152 or C1218) or a detailed air void system analysis in accordance with ASTM C457, *Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete.* Guidance on how to conduct a microscopic evaluation of concrete can be found in Van Dam et al. (2002) and Walker, Lane, and Stutzman (2006).

Samples from the unbound layers and the subgrade can be tested for properties such as:

- Soil density, moisture content, and moisture-density relationship (AASHTO T 99 and T 180).
- Soil classification, including particle size distribution and plasticity (AASHTO M 145).
- California Bearing Ratio (AASHTO T 193) or Hveem resistance value (AASHTO T 190).
- Triaxial strength (AASHTO T 296).
- Resilient modulus (AASHTO T 307).

These properties relate to the in-situ structural capacity of the pavement and reflect on anticipated future performance.

Functional Adequacy

Functional adequacy of the pavement describes characteristics that affect the suitability of the pavement to provide the traveling public with a smooth, quiet, and safe riding surface and includes measures of roughness, noise, and surface friction.

Roughness

Pavement roughness is a measure of the rideability of the pavement. It is a common acceptance item for new construction and is useful in tracking ride quality over time or in comparing rideability between projects. Subjective roughness surveys (windshield surveys) can be conducted by a trained rater driving over the surface in a vehicle that they are familiar with, rating both the ride quality and obvious distress. This can be a good starting point for assessing overall condition, to identify areas within a project that are exceptionally rough, and to potentially link the causes of roughness to the types and severities of visible distress. Such surveys lack the objectivity of automated surveys but have the advantage of being inexpensive and easily performed; moreover, they can provide valuable information for setting up a pavement preservation plan.

Objective roughness surveys are conducted using automated equipment specifically designed to collect pavement profile data, which can then be translated into a measure of roughness, such as the International Roughness Index (IRI). Today, pavement roughness assessment is most commonly performed using inertial road profiling systems (IRPSs), which measure actual pavement profiles and not a vehicle response to irregularities in the pavement (Smith et al 2014).

All IRPSs function similarly, although they can vary in accuracy and in the speed in which information is collected. The two key elements of an IRPS are the accelerometer(s) that are used to establish an inertial plane of reference and the noncontact distance-measuring devices (generally line lasers) that measure the distance between the accelerometer(s) and the pavement surface. In combination, these two inputs can be used to create the pavement profile.

A single vehicle, operating at highway speeds, commonly collects the profile data in both wheel paths of the pavement, which are then used to calculate a roughness statistic (commonly IRI). The IRI is based on a "quarter car" model, meaning that it calculates the response of a single wheel and sprung mass being "driven" at a prescribed speed over the measured profile. IRI is calculated in inches/mi and commonly reported over 0.1-mi (0.16-km) increments. Although each highway agency maintains its own criteria for levels of roughness that are considered "acceptable," the FHWA criteria are (FHWA 2017):

- Good – Less than 95 inches/mi (1.5 m/km).
- Acceptable – 95 to 170 inches/mi (1.5 to 2.7 m/km).
- Unacceptable – Greater than 170 inches/mile (2.7 m/km).

As a quarter car model, only a single profile (wheel path) is considered in the calculation of IRI. Some agencies will average the IRI from the two wheel paths to create the Mean Roughness Index (MRI) whereas others will apply the IRI model to an average of the two wheel path profiles to create a Half-car Roughness Index (HRI).

Specifications and practices regarding the measurement and evaluation of pavement roughness can be found in the following:

- AASHTO M 328 – *Standard Specification for Inertial Profiler*.
- AASHTO R 43 – *Standard Practice for Quantifying Roughness of Pavements*.
- AASHTO R 57 – *Standard Practice for Operating Inertial Profiling Systems and Evaluating Profiles*.

One important note is that the measured roughness of concrete pavements, particularly for short-jointed concrete pavements, will change over the course of the day or over a few days as the temperature of the surface varies with respect to the temperature at the slab bottom. This induces a temperature gradient resulting in differential expansion or contraction between the slab top and bottom, causing the slab to curl downward during the heat of the day or upward during the cool of the night. This temperature-induced slab curling can have significant effects on the pavement profile and calculated IRI and should be considered (FHWA 2013).

Surface Texture

The texture of the pavement surface is an important parameter, not only in how it influences surface friction, but also impacts vehicle fuel consumption and noise. Pavement texture can be separated into four components defined based on the maximum dimension of deviation (wavelength) from a true planar surface: roughness with wavelengths of 1.6 to 164 ft (0.5 to 50 m), megatexture with wavelengths of 1.6 ft to 2 inches (0.5 m to 51 mm), macrotexture with wavelengths between 2 inches and 0.02 inches (51 mm and 0.5 mm), and microtexture with wavelengths less than 0.02 inches (0.5 mm) (Snyder 2006). This section focuses on macrotexture and microtexture as these impact both tire-pavement generated noise and surface friction.

Macrotexture describes the overall texture of the pavement, which for concrete pavements is largely controlled by how the pavement is finished (Snyder 2006). Good macrotexture is needed for pavements carrying vehicles traveling at speeds of 50 mi/hr (80 km/hr) or greater, as it reduces the water film thickness and helps prevent hydroplaning (Hibbs and Larson 1996). Macrotexture of asphalt-surfaced pavements and concrete pavement without directional textures (i.e., tining, diamond grinding, grooving) can be assessed using the same IRPS equipment discussed previously. Yet, most concrete pavements have some type of directional texturing, a feature constructed into concrete pavement surfaces to provide macrotexture to improve surface friction. A direct measure of macrotexture thus needs to be used to assess concrete pavements with directional texture, such as ASTM E965, *Standard Test Method for Measuring Pavement Macrotexture Depth Using Volumetric Technique*. In part, due to the lack of an automated means for data collection, studies linking macrotexture to vehicle fuel efficiency or tire-pavement noise often do not include concrete pavements with directional texture.

Microtexture on concrete pavements is typically provided by the fine aggregate (sand) in the mortar (Snyder 2006). It has little contribution to tire-pavement noise or vehicle fuel consumption but plays a key role in providing surface friction through adhesion to the tire for vehicles operating at speeds below 50 mi/hr (80 km/hr) (Hibbs and Larson 1996).

Tire-Pavement Generated Noise

The noise generated from the interaction between the tire and the pavement surface has been the focal point of numerous studies over the last decade or so (Rasmussen et al. 2010). There are several mechanisms that are occurring as a vehicle tire passes over a pavement surface that generates noise (Sandberg 2001) with the textural characteristics of the surface being a major contributor. Pavement noise is most commonly measured by AASHTO T 360, *Standard Method of Test for Measurement of Tire/Pavement Noise Using the On-Board Sound Intensity (OBSI) Method*. The transverse tining pattern commonly used in the past to create macrotexture in concrete pavement was found to create significantly higher noise levels than other pavement textures, and thus it is now common to longitudinally tine concrete to reduce tire-pavement generated noise. More specific guidance on texturing to reduce tire-pavement noise in concrete pavements includes (Rasmussen et al. 2012):

- Avoid texture patterns with intervals of 1 inch (25 mm) or greater.
- Avoid extremely smooth surfaces with little macrotexture.

- Textures should point down (e.g., grooves) rather than up (e.g., fins); the former is sometimes referred to as negative texture.
- Grooving or tining should be in the longitudinal direction.
- Transverse grooves should be closely spaced and randomized whenever possible.

Jointing also plays a significant role in pavement noise (Rasmussen et al. 2012). Transverse joints should be narrow and single cut rather than widened, reservoir-style cuts and should avoid excess joint sealant that protrudes above the pavement surface. During texturing and curing, tire-pavement noise can be reduced by minimizing the buildup of latency on the tining equipment, ensuring the consistent tracking of texturing equipment; and providing multiple applications of curing compound.

For existing pavements, diamond grinding and/or grooving can be performed to minimize pavement noise. Larger, heavier grinding equipment will have the control necessary to consistently impart the texture at the intended depth and lateral coverage, minimizing any variability in the height of concrete and minimizing the vibration (Rasmussen et al. 2012). The combined use of flush grinding and grooving (see figure 3-5), in what is commonly referred to as the Next Generation Concrete Surface (NGCS), has been shown to provide a very quiet concrete surface (Guada et al. 2013).

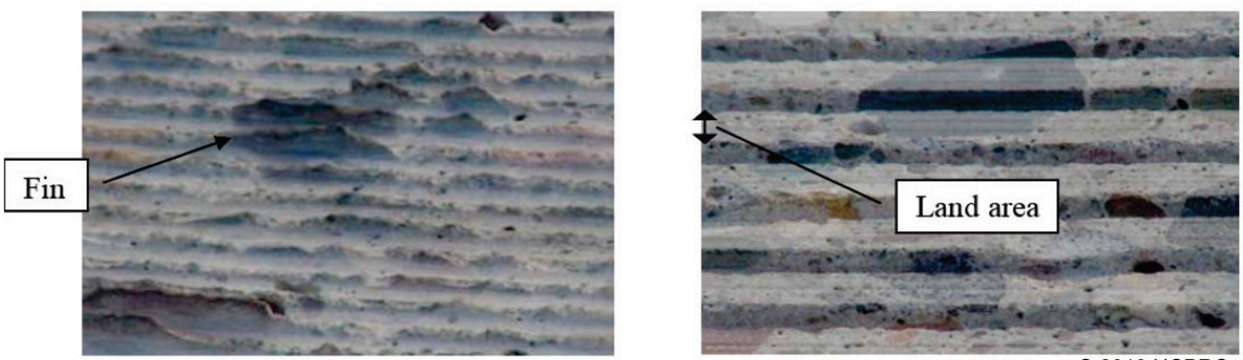

Figure 3-5. Conventional diamond grinding (left) and the NGCS (right) (Guada et al. 2013).

Surface Friction

Surface friction is of paramount importance to the traveling public as it heavily influences the ability of a vehicle to stop upon application of the brakes. Research has shown that roughly 14 percent of all crashes occur in wet weather and of those nearly 70 percent are preventable with improved pavement surface friction (Larson, Scofield, and Sorenson 2005). Moreover, inadequate friction can contribute to accidents under dry conditions as well due to unusual traffic movements and braking actions that can occur in intersections and work zones.

Surface friction has traditionally been measured using one of the four basic types of full-scale pavement surface friction devices (Henry 2000; Snyder 2006):

- Locked-Wheel Testers – Simulate emergency braking for vehicles without anti-lock braking system. Test conducted in accordance with ASTM E274, *Standard Test Method for Skid Resistance of Pavement Surface Using a Full Scale Tire.* The wheels (either ribbed or smooth) on a trailer-mounted unit are locked up on a wetted pavement surface. The measurement is reported as a “skid number” or SN, which is the measured value of friction multiplied by 100. When reporting SN, it is indicated whether the tire was ribbed or smooth and at what speed the test was conducted. Braking only occurs over small segments of the test site.

- Side-Force Testers – Designed to simulate a vehicle’s ability to maintain control in curves. The tire is mounted at an angle to the direction of motion and is allowed to rotate freely. Side force is developed and measured perpendicular to the direction of motion. Can be operated over the entire length of the test site.
- Fixed-Slip Testers – Used to simulate the braking ability of a vehicle with antilock brakes. Fixed-slip testers operate at a constant slip, typically between 10 and 20 percent. Can be operated over the entire length of the test site.
- Variable-Slip Testers – Similar to the fixed-slip tester except that the slip is varied in accordance with an established set of slip ratios. Testing is conducted in accordance with ASTM E1859, *Standard Test Method for Friction Coefficient Measurements Between Tire and Pavement Using a Variable Slip Technique.*

It is noted that skid resistance is not just a function of the pavement surface, but instead reflects the interaction between the pavement and the tires. As a result, many factors come into play including whether the pavement is dry or wet, roadway surface contaminants, temperature, vehicle speed, tire type, inflation, and wear, among other factors. Surface friction is required for pavement safety, yet it is not temporally static, typically decreasing over time as the pavement surface becomes smoother due to aggregate polishing and general surface deterioration or abrasion (Attoh-Okine and Adarkwa 2013).

Summary

Various methods are employed for project level pavement evaluation as a precursor to preservation or rehabilitation. Many state agencies combine a quantified measure of structural distress with some kind of quantified measure of ride quality to create an overall pavement quality metric, which is used as a basis for recommending preservation or rehabilitation treatments. Table 3-4 presents rating systems used by a select number of SHAs to determine maintenance and rehabilitation actions (Papagiannakis et al. 2009).

Some patterns and consistencies in the treatment plans based on ratings can be seen across the agencies. Despite inconsistencies in the exact calculation of the pavement performance scale (for example, some agencies considered only IRI, others quantified their structural distress values and into the rating), poor performing pavements generally required major rehabilitation. On the other hand, while some agencies require preventive maintenance for very good pavements, others make no recommendation for action.

Although not specific to pavement preservation, NCHRP (2004) presents a detailed description of how to approach collecting and compiling data for designing recommended rehabilitation alternatives. The document provides a checklist that can be used to step through the various elements of an evaluation and concludes with an approach to overall condition assessment to identify key problems and prepare feasible, cost-effective, rehabilitation alternatives. Approaches considered include determination of overall structural adequacy, drainage adequacy, material structural and durability adequacy, and functional adequacy.

Table 3-4. Rehabilitation and maintenance procedures based on pavement performance for select SHAs (Papagiannakis et al. 2009).

State	Condition Rating Scale	Maintenance and Rehabilitation Actions
Alabama	0 - 100	• Overlay at score of 55.
California	1 = Excellent 2 = Good 3 = Fair 4 = Poor 5 = Very Poor	• Preventive maintenance for 1 and 2. • Major rehabilitation or replacement for 3 to 5.
Colorado	RSL >11 = Good RSL 6 - 10 = Fair RSL 1 - 5 = Poor RSL 0 = Due	• Needs rehabilitation when RSL is 0.
Delaware	4 - 5 = Very Good 3 - 4 = Good 2.5 - 3 = Fair 2 - 2.5 = Poor <2 = Very Poor	• Routine Maintenance for 4 to 5. • Preventative maintenance for 3 to 4. • Rehabilitation for 2.5 to 3. • Reconstruction 2.5 or less.
Florida	0 = Worst 6 = Not deficient for speed limit < 55 mph 6.4 = Sound condition 10 = Best	• Major Rehabilitation or Replacement for 6 or less. • Preventive Maintenance 6 to 10.
Georgia	100 - 75 = Excellent/good 70 - 75 = Fair <70 = Poor/bad	• Rehabilitation for 70 to 75. • Resurfacing for less than 70.
Illinois	7.6 - 9.0 = Excellent 6.1 - 7.5 = Good 4.6 - 6.0 = Fair 0 - 4.5 = Poor	• Preventive maintenance for 6.1 to 9. • Repair in short term for 4.6 to 6. • Immediate major rehabilitation for less than 4.5.
Iowa	0 - 39 = Poor 40 - 60 = Fair 60 - 80 = Good 80 - 100 = Excellent	• Reconstruction for less than 39. • Major rehabilitation for 46 to 60. • Preventive maintenance for 60 to 100.
Kansas	1 = Excellent 2 = Fair/good 3 = Poor	• Routine maintenance for 2. • Rehabilitation for 3.
Michigan	4 - 5 = Poor 3 - 3.5 = Fair 1 - 2.5 = Good	• Major rehabilitation or replacement for 4 to 5. • Preventive maintenance for 1 to 3.5.
Missouri	Acceptable PSR Marginal PSR Unacceptable PSR	• Preventive maintenance for acceptable PSR. • Asphalt surface treatments for marginal PSR. • Rehabilitation for unacceptable PSR.
Ohio	0 - 40 = Very poor 40 - 55 = Poor 55 - 65 = Fair to poor 65 - 75 = Fair 75 - 90 = Good 90 - 100 = Very good	• Major rehabilitation or replacement for 55 or less. • Preventive maintenance for 75 to 90.
South Carolina	0 - 1.9 = Very poor 2.0 - 2.6 = Poor 2.7 - 3.3 = Fair 3.4 - 4.0 = Good 4.1 - 5.0 = Very good	• Major rehabilitation or replacement for 2.6 or less. • Preventive maintenance for 3.4 to 4.0.
Tennessee	0 - 1.0 = Very poor 1.0 - 2.5 = Poor 2.5 - 3.5 = Fair 3.5 - 4.0 = Good 4.0 - 5.0 = Very good	• Mandatory field review performed for 1.0 or less. • Added to the resurfacing program for 1 to 2.5. • Eligible for resurfacing program for 2.5 to 3.5. • Maintenance for 3.5 to 4.
Wisconsin	0 - 19 = Very good 20 - 39 = Good 40 - 59 = Fair 60 - 79 = Poor 80 or more = Very poor	• Preventive maintenance for 39 or less. • Major rehabilitation or replacement for 59 or more.

RSL: Remaining Service Life; PSR: Present Serviceability Rating

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CHAPTER 4. STRATEGIES FOR CONCRETE PAVEMENT PRESERVATION

As described in previous chapters, a number of preservation strategies can be used to contribute to the extended service life of concrete pavements. First, beginning in the pavement design stage, agencies can adopt long-life concrete pavement designs that provide extended life with very little additional cost investment; this reduces future maintenance and rehabilitation requirements and minimizes the associated traffic disruptions due to lanes closures. And in later stages of the concrete pavement, agencies can consider strategies such as CPR and overlays to preserve and enhance the structural and functional life of the existing pavement. For the purpose of concrete pavement preservation as defined herein, these later CPR and overlay strategies have a strong temporal element, as the techniques themselves are not different from what are currently used but the timing of their application is shifted to more effectively preserve the pavement structure while maintaining functionality. A discussion of these strategies is presented in this chapter.

Long-Life Pavement Design and Construction

The concept of long-life (40 years or more) concrete pavement design has its roots in the "zero maintenance" design concept introduced by Darter and Barenberg (1977). This was a pioneering mechanistic-based design methodology that incorporated a stress ratio approach for calculating fatigue damage and also considered the impact of thermal curling stresses. Even with the release of the empirically based *AASHTO Guide for Design of Pavement Structures* (AASHTO 1993), work continued on the development and refinement of mechanistic-based design procedures (e.g., PCA 1984, Thompson and Barenberg 1992; and Darter, Hall, and Kuo 1995).

In 2004, the NCHRP issued the *Mechanistic-Empirical Pavement Design Guide (MEPDG)* (NCHRP 2004), which is now in its second edition (AASHTO 2015) and is available as the AASHTOWare *Pavement ME Design* software. The MEPDG has been nationally calibrated and continues to be refined for local and regional conditions as highway agencies work to implement the procedure. Still, it is realized that the adoption of mechanistic-empirical design procedures alone does not ensure long life, but instead provides a tool that can be used to assess the potential long-term performance capabilities of various concrete pavement designs.

While designing concrete pavements for long life is the first step in preserving concrete pavements, ensuring that the intent of the design is specified and constructed is the second step. Tayabji and Lim (2007) summarize the requirements expected from long-life concrete pavements as follows:

- The original concrete service life is 40+ years.
- The pavement will not exhibit premature construction and materials-related distress.
- The pavement will have reduced potential for cracking, faulting, and spalling.
- The pavement will maintain desirable ride and surface texture characteristics with minimal preservation activities (if warranted) for ride and texture, joint sealing, and minor repairs.

Tayabji and Lim (2007) go on to describe how long-life concrete pavements are readily achievable by refining current design and construction practices and by requiring improved construction quality control.

During a scan tour of long-life concrete pavements in Europe and Canada, researchers identified a number of practices used by those countries to achieve long-life, including (Hall et al. 2007):

- Use Deep, High-Quality Foundations – Unbound granular materials used in European subbases are generally better quality materials than those typically used for select fill and granular subbases in the United States. Recycled concrete is frequently used in European pavements as a base material.
- Pay Attention to Mix Design Components – One key to long-lasting concrete pavements in Europe is the great attention paid to cement and concrete mixture properties; this attention to detail consistently produces strong, dense, and durable concrete.
- Use a Geotextile Interlayer – Thick, geotextile interlayers prevent the concrete slab from bonding to the cement-treated base. European interlayers were typically thicker than those found in the United States.
- Use Two-Lift Construction – This allows for better quality aggregates to be used in the top layer and lower quality aggregate (including recycled aggregate materials) to be used in the lower layer, providing cost savings and conservation of materials. This approach is commonly used in Germany, Austria, Belgium, and the Netherlands.
- Use Catalog Designs – Many European countries use catalog designs more frequently than the United States. Design catalogs are used to select pavement thicknesses and other base pavement design features. This allows for iteration with field experience and local climate to prevent overdesigning pavements.
- Use Low-Noise Exposed Aggregate Surfacings – Exposed aggregate surfacings are used in the top course of two-lift pavements in some densely populated areas of Europe for reduced noise emissions while also providing good friction and durability.

Long-Life Concrete Pavement Design Features

Although concrete pavement design often focuses on slab thickness, a number of key concrete pavement design features are critical to long-term performance and should be considered in the pavement design process. As early as 1977, several key design features were recommended to help achieve long-term performance (Darter and Barenberg 1977):

- Dowel bars.
- Short joint spacings.
- Good subdrainage.
- Adequate joint sealing.
- Increased slab thickness.
- Increased foundation support.

The AASHTO MEPDG directly considers many of these factors, ensuring that their impacts on performance are accounted for in the design process.

The Minnesota DOT conducted a study to establish long-life guidelines for recommended design features based on a review of the long-life pavements in the state (Tutumluer, Ziao, and Wilde 2015). General recommendations from that study are summarized in table 4-1.

Table 4-1. Recommended design features for Minnesota long-life pavement design (Tutumluer, Ziao, and Wilde 2015).

Design Feature	Material	Typical Dimensions
Slab Thickness	HPCP mix	12-13 inches (305 to 330 mm)
Base course	MnDOT Class 5 material	4 inches (102 mm)
Subbase	MnDOT select granular	Minimum 36 inches (914 mm)
Transverse joints	Preformed elastomeric compression seal	15 ft (4.6 m) spacing, perpendicular to the direction of traffic
Dowel bars	Corrosion resistant	1.5-inch (38 mm) diameter, 15-inch (381 mm) long bars spaced 12 inches (305 mm) center to center
Texture	Astro-turf or broom drag	1/25 inch (1 mm) average depth using ASTM E965

Table 4-2 presents long-life concrete pavement design features and characteristics used in selected countries (Tutumluer, Ziao, and Wilde 2015). Most countries use JPCP for their long-life pavements, with the exception of Belgium (which uses CRCP). The selected countries consistently sought to improve the drainage conditions of the pavement structure through variations in cross slope or by using drainable base courses. Many countries also construct a robust foundation system.

Under a recent SHRP2 study, researchers developed the following characteristics and features for long-life pavements (Jackson, Puccinelli, and Mahoney 2014):

- For JPCP:
  - Design lives range from 50 to 60 years.
  - Slab thicknesses range between 11.5 and 13 inches (292 and 330 mm).
  - Joint spacings are 15 ft (4.6 m), with load transfer provided through corrosion-resistant dowels.
  - Maximum water-to-cementitious materials ratio (*w/cm*) between 0.40 and 0.44.
- For CRCP:
  - Design lives range from 30-40 years.
  - Slab thicknesses range between 13 and 15 inches (330 and 381 mm).

Although developed with new pavement construction in mind, these characteristics are also applicable to long-life concrete overlay systems (Jackson, Puccinelli, and Mahoney 2014).

Foundation Support

Although the foundation has a relatively minor contribution to the required slab thickness for a set of design conditions, these layers are important to long-term performance. To begin with, premature failures in concrete pavements are often the result of excess moisture beneath the slab, which contributes to pumping, joint faulting, erosion and loss of support, and slab cracking. An effective foundation system that either removes the excess water or is relatively insensitive to its effects will provide stable slab support and resist erosion. It has also been observed that characteristics of the slab-subsurface interface under moving wheel loads is the primary contributor to erosion of the subbase layer (Jung and Zollinger 2011). The erosion resistance is related to the type of foundation materials, the fines content, and the stabilizer (if used).

Table 4-2. Long-life concrete pavement designs in selected countries (after Tutumluer, Ziao, and Wilde 2015).

Country	Type	Design Features/Characteristics	Reference
Australia	CRCP, JPCP	(a) Typically use CRCP and a design thickness catalog (b) JPCP debonded from base layer to allow for free curling and warping of concrete slab (c) A lean concrete base with minimum compressive strength of 725 lb/in^2 at 42 days (no induced joints and limited construction joints) (d) A minimum 12 in subbase of select material with CBR > 30% after 4-day soak, PI<12, and compressive strength of 145 lb/in^2 (top 6 inches subbase is stabilized with 2% hydrated lime if soaked CBR < 30%)	Vorobieff and Moss (2006)
Austria	JPCP	(a) Typical high volume (18 to 40 million design axle loads) pavement 10 in JPCP on 2 in bituminous interlay over 18-inch unbound base or 8-inch cement-stabilized base (b) 18 to 20 ft joint spacing (c) Two-lift concrete slab with recycled/inexpensive aggregate used in the bottom 8 inches and more wear resistant aggregate in the upper 1.5 inches with an exposed aggregate surface	Hall et al. (2007)
Belgium	CRCP	(a) Minimal maintenance in-service life of 30 years and expected life span of 40 to 50 years (b) 2.5-inch asphalt base over 8-inch lean concrete base over 8- to 31.5-inch sand subbase (over specially compacted 3.3 to 4.9 ft subgrade) (c) The asphalt base prevents reflective cracking from the subbase (d) High strength concrete base, asphalt base course, excellent bond between asphalt layer and upper and lower concrete layers, and high-quality concrete contribute to good performance (e) Side gutters, sufficient cross slope, thick layer of drainage sand, and drainage pipes on the edge of the shoulder lead to good drainage	Caestecker (2006)
Canada	JPCP	(a) Standard design in Ontario is doweled JPCP with a 14-ft widened outside lane (b) Use 1993 AASHTO Guide and Canadian Portland Cement Association ME rigid pavement design methods (c) 4-inch asphalt treated, open-graded drainage base (0.75-inch maximum aggregate size and 1.8% asphalt content) (d) Open-graded cement-treated base as an alternative (e) JPCP over 6-inch granular base over granular subbase of varying thickness in Quebec for frost heave prevention (f) 11-inch CRCP over open-graded cement-stabilized base as an alternative in Quebec	Hall et al. (2007)
Germany	JPCP	(a) 8.6- to 13.8-inch JPCP over base and frost protection layers (b) Use geotextile between concrete slab and cement-stabilized bases to debond these two layers	Hall et al. (2007)

1 in = 25.mm
1 ft = 0.3048 m
1 lb/in^2 = 6.89 kPa

In addition, because concrete slabs distribute loading over large areas, it is the uniformity of the foundation support (and not the strength of foundation support) that is important to concrete pavement performance (ACPA 2007). Moreover, the foundation layers also provide other key benefits, including frost resistance and a stable working platform that can contribute to increased initial smoothness.

Table 4-3 summarizes the foundation considerations of selected highway agencies, based on the work done by Tutumluer, Ziao, and Wilde (2015). Most agencies use stabilized bases for their long-life concrete pavements, with many also providing positive drainage and granular subbases.

Table 4-3. Long-life concrete pavement foundation designs for selected states (after Tutumluer, Ziao, and Wilde 2015).

State	LLCP Type	Foundation Design Considerations	Reference
California	CRCP, JPCP	(a) 4-inch cement-treated (3.5%) base over 8-inch granular (b) Cement-treated (4%) base over A-2-4 granular subbase (c) Using either cement- or asphalt-treated base for good drainage	Rao, Darter, and Pyle (2006)
Florida	JPCP	(a) Well-performing pavements: good subbase drainage; (b) Poorly-performing pavements: poor subbase drainage (causing pumping and corner breaks) (c) 4-inch asphalt/cement treated permeable base over a stabilized subgrade (asphalt separation layer in between for fines migration prevention) (d) 5-ft subgrade/embankment composed of special select material meeting AASHTO A-3 soil classification (e) Use an edge drain in all concrete pavements	Armaghani and Schmitt (2006)
Illinois	CRCP	(a) Use the 1993 AASHTO design guide (b) Hot mix asphalt stabilized base (c) 12-inch aggregate subbase layer (1.5-inch and 8-inch maximum aggregate size for the top 3 inches and bottom 9 inches, respectively)	Winkelman (2006)
Iowa	JPCP	(a) Limited data available about subgrades (b) Longitudinal cracking is one of the most common distresses for low and medium volume roads (c) Subgrade failure can cause longitudinal cracking	Ceylan, Cable, and Gopalakrishnan (2006)
Texas	CRCP	(a) 6-inch cement-stabilized base with 1-inch asphalt layer on top, or 4-inch asphalt-stabilized base (b) Pumping and erosion of the base caused punchout distress	Won et al. (2006)
Virginia	N/A	(a) Blocked drains caused premature failures even for concrete and materials of high quality (b) Good pavement condition: no clogged drains (c) 4-inch AASHTO #57 aggregate bases over 6-inch cement-treated soil (10% cement by volume) (d) 4-inch dense-graded base for shoulders (e) Use edge drains	Hossain and Elfino (2006)
Washington	JPCP	(a) Joint faulting due to poor underlying conditions and lack of dowels (b) Currently used base layer: 4-inch dense-graded HMA over 4-inch crushed stone (c) Cement-treated base caused severe joint faulting, pumping, and cracking (d) Asphalt-treated base caused minimal joint faulting (which occurred due to stripping because of low asphalt content, i.e., 2.5 to 4.5%)	Muench et al. (2006)

1 in = 25.mm
1 ft = 0.3048 m

The erodibility of the underlying foundation layers can greatly reduce the performance of an otherwise well-designed pavement. Jung et al. (2009) provide a list of tests that can be used to assess the erosion properties of the sublayers (see table 4-4). Although all erosion tests have some weaknesses, introducing a quantifiable level of erosion performance could help ensure the use of improved foundation materials. Table 4-5 summarizes erosion considerations in selected concrete pavement design procedures (Jung et al. 2009).

Table 4-4. Tests for erosion properties of the foundation layers (Jung et al. 2009).

Test Method	Features	Strengths	Weaknesses
Rotational shear device	Stabilized test samples	Easy and precise to control shear stress	No consideration of crushing or compressive failure
Jetting device	Pressurized water at an angle to the upper surface of unstabilized samples generating weight loss over time	Easy to test	Shear stress is not uniform and inaccurate. Overestimation of weight loss by coarse aggregates loss
Brush test device	Rotational brush abrasions generate fines. An erosion index is defined as the ratio of the weight loss to that of a reference material	Easy to test. Consider durability of wet and dry cycle. Relative erodibility of each material is defined using an erosion index	Long test time and overestimation of weight loss by coarse aggregate loss
Rolling wheel erosion test device	Wheel movements over a friction pad on sample induce erosion. Measure average erosion depth after 5000 wheel load applications	Simulate field conditions for flexible pavement. No coarse aggregate loss	Voiding of the subbase under concrete slab cannot be considered. Sample saw-cut can damage sample surface

Table 4-5. Erosion considerations in selected concrete pavement design procedures (Jung et al. 2009).

Procedure	Features	Strengths	Weaknesses
PCA	Provide erosion factor as a function of the slab thickness, composite k value, dowel, and shoulder type	Consider erosion analysis in design procedures as the most critical distress in rigid pavement performance	Proposed composite design k-values for treated bases are overestimated and need discrimination for different stabilization levels
1993 AASHTO	Composite modulus of subgrade reaction considers the loss of support (LS) due to the foundation erosion	Accounts for structural degradation of support due to erosion using LS factor	k-value obtained from the chart is overestimated and LS is insensitive to various stabilized materials
NCHRP 1-37A MEPDG	Classified erodibility of subbase materials is utilized in NCP faulting prediction models as well as erosion width estimation of CRCP	Employed the erodibility class based on the type and level of stabilization along with compressive strength	Erodibility class is determined based on dry brush test results and strength even though erosion occurs mostly under saturated conditions
TxDOT	Select one from two types of stabilized subbase and require minimum 7-day compressive strength	Historical performances and erosion resistance are demonstrated as good	Costly excessive design regardless of subgrade and environmental condition

Other Design Features

Other concrete pavement design features, such as slab size, lateral edge support (tied concrete shoulder or widened slab), and dowel bar spacing and size, can also play a prominent contributing role in the performance of long-life concrete pavements (Smith and Hall 2001). For example, Khazanovich et al. (1998) evaluated data from the Long-Term Pavement Performance (LTPP) program to identify key design and construction features that had the greatest effect on concrete pavement longevity; these factors are summarized in table 4-6 by concrete pavement type. For jointed pavements, joint load transfer, joint spacing, and slab lateral support were recognized to play an important role.

Table 4-6. Key design and construction features affecting concrete pavement performance (Khazanovich et al. 1998).

Pavement Type	Key Design and Construction Features Affecting Performance (in decreasing order of importance)
JPCP	Initial smoothness Joint load transfer Subdrainage Base type Slab widening Joint spacing Slab thickness Slab modulus/strength
JRCP	Initial smoothness Joint load transfer Subdrainage Joint spacing Slab thickness
CRCP	Initial smoothness Steel percentage Subdrainage Slab strength Surface texture Base type

In 2007, Caltrans evaluated strategies for long-life JPCP concrete pavements, specifically evaluating: (a) the adequacy of structural design options (such as tied shoulders, doweled joints, and widened slabs) with respect to joint distress, fatigue cracking, and corner cracking; (b) the durability of concrete slabs made with cements meeting the requirements for early-opening-to-traffic; and (c) the effects of construction and mix design variables on the durability and structural performance of pavements. Using these objectives as a guide, Caltrans developed the following recommendations for long-life concrete pavements (Jones, Harvey, and Kohler 2007):

- Use a mechanistic-empirical design procedure.
- Use the axle load spectra instead of equivalent standard axle loads (ESALs) since the load spectra produces a better design estimate.

- Use dowel bars to mitigate joint faulting.
- Limit joint spacing to a maximum of 15 ft (4.6 m) to delay transverse cracking.
- Use widened truck lanes or tied concrete shoulders to improve fatigue cracking performance.
- Use nonerodible treated base types to minimize transverse faulting and corner cracking.
- Avoid the use of stiff bases in climates with large temperature gradients since they increase the tensile stresses in the slab.
- Include the coefficient of thermal expansion as a design parameter and its impact should be evaluated within a range comparable to the expected conditions of the pavement.
- Develop a new design specification for asphalt-treated bases since current specifications are designed against rutting performance when it should consider moisture sensitivity and friction between the slab and base.
- Use flexural strength to determine the pavement performance rather than compressive strength. While the requirement of 300 lb/in^2 (2.1 MPa) for opening strength should remain, increasing the minimum opening strength to 400 lb/in^2 (2.8 MPa) is recommended for minimizing early cracking. The 90-day required strengths of 650 to 800 lb/in^2 (4.5 to 5.5 MPa) are still recommended.
- Retain the requirement of no more than 0.053 percent drying shrinkage after 7 days.
- Recognize that concrete set time is not the most important variable for construction productivity in typical construction closure lengths.
- Continue the use of ASTM C1260 to test aggregate alkali-silica reactivity.
- Use ASTM C1293 for a wide variety of aggregate types for alkali-silica reactivity.
- Adopt and enforce the ACI Building Code 318 guidelines for sulfate resistance.
- Establish a failure criterion of less than 25 percent loss of strength at 28 and 63 days of sulfate exposure relative to the 7-day strength.
- Use bond-breaking layers with bases with a high frictional resistance (lean concrete bases or open-graded mixes) to reduce tensile stresses in the slab.
- When possible, pave during cool, moist conditions to reduce built-in curl.
- Use construction productivity analysis tools to determine the critical path for construction.
- Conduct traffic analyses to maximize the number of lanes that can be closed to traffic during construction.
- Continue strict monitoring of the water/cement ratio.

Based on a statewide performance study of long-life pavements in Minnesota, researchers provided the following recommendations for long-life concrete pavements (Tutumluer, Ziao, and Wilde 2015):

- Use two-lift construction approach to building pavements.
- Develop pavement design catalogs.

- Use better quality materials in pavement subbases.
- Pay greater attention to cement and concrete mixture properties.
- Use a geotextile interlayer to prevent concrete slabs from bonding with the cement-treated base.
- Use exposed aggregate surfaces to reduce noise.

Quality of Construction

As described earlier, good construction is needed to help ensure the pavement design achieves its performance expectations. Quality assurance (QA) activities must be performed during each stage of the pavement construction process to ensure that the materials and workmanship meet or exceed the project specifications. Pavement systems that are constructed in accordance with the specifications are likely to achieve or exceed the intended design life and exhibit lower life-cycle maintenance costs.

Some of the main aspects of concrete pavement construction for which QA is essential in order to achieve long life include plant certification, stringline setup and maintenance, proper equipment setup and hauling, proper placement of concrete to minimize segregation and maintain a constant load ahead of the paver, proper materials QA (e.g., monitoring mixture consistency through air, slump, unit weight testing), proper consolidation without overvibration, proper selection and application of curing compounds, and timely and effective joint sawing. Best practices for concrete pavement construction are available from the ACPA (ACPA 2008; ACPA 2010).

Summary of Long-Life Design and Construction

The design and construction of long-life concrete pavements do not require any special or exceptional measures, but instead rely upon applying common techniques with diligence and attentiveness. For long-life concrete pavements, the slab thickness is typically slightly increased to enhance structural load-carrying capacity and also to accommodate future grinding. Greater attention is paid to the supporting layers, ensuring that they are nonerodible and provide uniform support. Other design features are implemented to reduce stress, including the universal use of doweled joints, shorter joint spacings, and widened slabs or tied shoulders. Concrete materials are selected to be durable and the resulting concrete mixtures should exhibit low shrinkage and reduced permeabilities. Effective paving specifications are used to ensure high-quality construction.

Composite Pavements Through Overlays

As previously mentioned, many SHAs use major rehabilitation in the form of overlays (either PCC or HMA) as a treatment of pavements that are structurally degraded and in poor condition. However, using an overlay while an existing pavement is still in “good” condition can greatly extend the life of the pavement and thus can be considered a preservation strategy contributing to long life. Jackson, Puccinelli, and Mahoney (2014) provided critical features and limitations of various methods of PCC renewal options. These are presented in table 4-7, which lists three primary means of overlaying existing concrete pavement: unbonded concrete overlays, bonded concrete overlays, and HMA overlays.

Table 4-7. Critical features and limitations of different pavement overlay options (based on Jackson, Puccinelli, and Mahoney 2014).

Approach	Critical Features	Limitations
Unbonded concrete overlay over existing concrete	- Overlay thickness is critical to performance - Repair locally failed areas - 1.5-inch (38 mm) diameter rust-resistant dowels - 15 ft (4.6 m) maximum joint spacing - Interlayers should not trap water - Thicker HMA interlayer performed better - Adequate drainage	- Significant surface elevation increase - Consistent foundation support when widening - Consistent drainage when widening
Bonded concrete overlay over existing concrete	- Overlay thickness is critical to performance - Locally failed areas must be repaired - Stable subbase - Joints must match existing - Adequate drainage	- Existing pavements with materials-related distress are not good candidates - Existing pavements with voids are not good candidates - Working cracks can cause debonding of overlay - Service life up to 35 years
HMA over existing concrete[1]	- Good foundation support - Adequate drainage - No evidence of pumping - Existing pavement is structurally adequate - Absence or repair of major defects - Good bond of concrete pavement and HMA	- Concrete pavement has to be in good condition with few major defects (which would require repair) - Inadequate bonding can lead to poor performance - Unproven for the 50-year life

1 This study only considered HMA on CRCP as a long-life design.

Concrete Overlays

Concrete overlays can be either bonded or unbonded. These overlay types provide a number of benefits, including (Harrington and Fick 2014):

- Overall cost-effectiveness.
- Rapid and convenient construction.
- Minimal maintenance requirements.
- Enhanced surface characteristics (ride quality, surface friction, noise emissions).

A summary of primary advantages and disadvantages of each overlay type is provided in table 4-8, with additional details provided in the following sections.

Table 4-8. Primary advantages and disadvantages of concrete overlay options (Jackson, Puccinelli, and Mahoney 2014).

Rigid Pavement Renewal Approach	Advantages	Disadvantages
Unbonded concrete overlay over concrete	- Very good long-term performance with minimal maintenance or rehabilitation - Insensitive to existing pavement condition - Best documented record of projects in place that have achieved long life	- Significant surface elevation gain - Placement or cure time may make work zone management difficult
Bonded concrete overlay	- Smallest vertical elevation gain	- Unlikely to be viable for service lives longer than 35 years

Unbonded Concrete Overlays

Unbonded concrete overlays of concrete are almost exclusively used to rehabilitate concrete pavements that are in relatively poor condition (Harrington and Fick 2014). In these cases, the existing concrete is treated like a stiff base course and the new concrete is separated from it through either a separation layer (e.g., asphalt interlayer) or a thick geosynthetic fabric. The purpose of the separation is to permit the new concrete surface to act independently from the underlying pavement. Unbonded concrete overlays can range in thickness from about 6 inches (152 mm) to 10 inches (254 mm) or more depending on the design conditions.

Bonded Concrete Overlays

Bonded concrete overlays of concrete should be applied only to existing concrete pavements in good condition. These overlays are intended to structurally enhance the existing pavement or to correct a functional deficiency that cannot be addressed by another means (Harrington and Fick 2014). A bonded concrete overlay relies largely on the original pavement and sublayers for structural support with the overlay primarily serving as an improved riding surface. As such, a critical element in the performance of the overlay is that it is effectively bonded to the existing pavement.

During construction, concrete mixture characteristics, surface preparation, construction, and environmental impacts must be carefully monitored to ensure overall good bond. Bonded concrete overlays are thinner than unbonded concrete overlays, typically in the range of 3 to 4 inches (76 to 102 mm). The following factors are noted to significantly affect the performance of bonded concrete overlays of concrete (Harrington and Fick 2014):

- The structural integrity of the underlying pavement.
- The effectiveness of the bond permitting the two layers to move monolithically to maintain the bond.
- The overlay jointing and curing techniques.

Because of the higher risk associated with bonded concrete overlays, designers should approach their use with caution.

HMA Overlays

HMA overlays of concrete pavements have been a common rehabilitation treatment for many decades. The new HMA surface corrects surface deficiencies such as faulting, spalling, and poor surface friction while restoring ride quality and reducing noise. However, traditional HMA overlays of concrete pavements can have some limitations, particularly in terms of the likelihood of existing structural distress in the concrete pavement reflecting through the overlay. This can significantly detract from pavement performance and will necessitate repeated maintenance and rehabilitation treatments over the years. Fracturing of the slab prior to the placement of the HMA overlay (either through crack/break and seating or rubblization) can help minimize the occurrence of reflection cracking.

Rao et al. (2013) studied the use of new asphalt surfaces on new concrete as part of composite pavement systems. The new asphalt surfaces are typically thin (1 inch [25 mm] or less) and could include stone matrix asphalt (SMA), porous HMA, asphalt rubber friction course (ARFC), or ultra-thin bonded gap-graded asphalt rubber hot mix. These thin surfaces are placed on a new concrete pavement to provide specific functional characteristics (e.g., smoothness, surface friction, and reduced noise). The asphalt surface also serves as an insulator that will lower the temperature gradient in the underlying concrete slab and reduce thermal stresses, which potentially could result in reductions in the designed slab thickness (Rao et al. 2013). Finally, the asphalt-based surfaces are relatively easy to rehabilitate through milling and replacement, thus minimizing future traffic disruptions while maintaining good functionality.

Concrete Pavement Restoration (CPR)

Over the last 40 years or so, CPR has been used to effectively improve the performance and extend the service life of concrete pavements. CPR is considered a better and more cost-effective option in areas with localized distress that would benefit from targeted improvements rather than receiving an overlay over the entire project (ACPA 1997). Compared to overlays, CPR can be used to address the underlying mechanisms of the distress (and not just the symptoms), thus providing for improved performance. And when diamond grinding is performed as part of the CPR operation, the functional service of the pavement can be quickly and effectively restored.

As presented previously in table 2-2, CPR refers to a host of concrete pavement treatments that are combined, as needed, to improve the performance of an existing concrete roadway, including (Smith 2005; Smith et al. 2014):

- Joint Resealing – The installation of a sealant material into the joint, which minimizes infiltration of both water and incompressible materials and thereby helps reduce the risk of pumping, faulting, and spalling.
- Slab Stabilization – The application of a cementitious grout or polyurethane material below the slab to fill voids. This reduces corner deflections and consequently reduces pumping and faulting.
- Partial-Depth Repairs – The removal and replacement of small near-surface areas of an existing concrete slab (depths up to one half the slab thickness) to restore the integrity of joints damaged by such distresses as joint spalling or D-cracking.

- Full-Depth Repairs – The removal and replacement of a portion of an existing concrete slab through its entire thickness to address distresses such as medium- or high-severity cracking, corner breaks, and blowups.
- Load Transfer Restoration – The installation of dowel bars in slots cut across joints or cracks in a post-construction mode so as to increase the load transfer and control pumping and faulting.
- Diamond Grinding and Grooving – The removal of a thin layer (typically less than 0.25 in [6 mm]) of the pavement surface to restore smoothness and friction (diamond grinding) or the addition of grooves to the pavement surface to reduce hydroplaning potential and wet-weather accidents (diamond grooving).

The Washington State DOT, in a report on the performance of its long-life pavements, noted that the serviceability of many of its concrete pavements was extended through the timely application of CPR, as documented in table 4-9 (Muench et al. 2010). Through these successful CPR interventions, 38 percent of the agency's 2300^+ lane miles of concrete pavement are over 35 years old and still in service, despite having been designed for a 20-year performance period.

Table 4-9. WSDOT restorative actions for concrete pavements (Muench et al. 2010).

New Construction Age (Years)	No Rehabilitation (lane-miles)	HMA Overlay (lane-miles)	Diamond Grinding (lane-miles)	Dowel Bar Retrofit and Diamond Grinding (lane- miles)
0-10	54.9	-	-	-
11-20	298.6	1.3	1.1	-
21-30	520.0	10.6	1.0	20.09
31-40	268.7	49.2	43.2	259.2
41-50	220.5	219.3	101.8	71.0
51-60	9.5	105.5	0.8	-
61-70	12.9	193.9	6.6	-
71-80	31.6	659.8	3.0	-
81-90	5.2	359.3	6.5	-
91-100	-	104.8	-	-
Total	1421.8	1703.7	164.0	350.3

It can be seen that the most common restorative action for older concrete pavements (60^+ years) is using an HMA overlay to improve the riding surface. The most common restorative action for pavements less than 40 years old was dowel bar retrofit with diamond grinding.

Summary

Three broad preservation strategies, working in concert, can be used to help ensure the longevity of concrete pavements. The first is to design and construct concrete pavements for long-life, which requires the selection of durable materials, the development of a durable concrete mix design, the selection of design features that reduce load- and environmentally induced stresses, and the effective construction of the pavement. The second strategy involves the use of overlays, which have traditionally been used to restore functionality to deteriorated concrete, but in this context they are applied to structurally sound concrete pavements as a way to extend pavement life. The third strategy involves the application of CPR techniques, not in a reactive manner, but instead applied to structurally sound concrete pavements as a means to maintain a high level of serviceability.

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CHAPTER 5. ENGINEERING ECONOMIC ANALYSIS AND CONCEPTS FOR STRATEGY SELECTION

Introduction

Economic analysis provides a way of comparing the expected benefits from a proposed investment (generally quantified in monetary terms) with the cost of that investment, thereby providing an understanding of the value to be expected for the cost incurred. Engineering economic analysis (EEA) applies these concepts and methods to engineering problems to aid in developing decisions for a "best" course of action. In pavement network applications, these decisions usually involve the identification of a "preferred alternative" from candidate courses of action for the design, maintenance, preservation, rehabilitation, or reconstruction of a particular project (Markow 2012).

Unlike private-sector investments, public-sector projects and services (such as the construction and maintenance of streets and roads) typically do not generate income streams for owner-agencies; the economic value of highway projects and services is generally considered in terms of benefits to the public (e.g., reductions in congestion and travel time, etc.) or avoidance of potential costs to either the agency or the users (e.g., reducing risks of crashes or preventing deterioration of the facility).

Many pavement-related benefits and costs of the type described above are not directly measurable in monetary units. Guidance in the assignment of monetary values to user costs, traffic delay costs, accident and injury costs, etc. is increasingly available from many sources, but some benefits and costs are not easily quantified (e.g., the value of relative levels of service for any given traffic volume or composition). Therefore, economic analysis alone is often insufficient for selecting project-level or network-level strategies and it is often necessary to also consider (or integrate) these sorts of non-monetary factors into the strategy selection process.

This chapter identifies general methods and techniques currently in use for engineering economic analysis and pavement management strategy selection and describes specific strengths and concerns with each approach.

EEA – Discounted (LCCA-Based) Cash Flow Methods

Several methods have been developed for evaluating and comparing streams of monetized costs and benefits over a given analysis period, including present worth of costs (PWC), net present value (NPV), equivalent uniform annual cost or value (EUAC or EUAV), benefit-cost ratio (B/C), and internal rate of return (IRR). The first three are most commonly used in analyses to support pavement management decisions and are discussed in more detail below. All four methods are based on life-cost analyses (LCCA)—an analysis of discounted cost and benefit streams—and will produce consistent results (i.e., will result in selection of the same alternatives or strategies) when applied correctly (Riggs and West 1986).

It should be noted that many agencies have developed or adopted standardized monetary values for user travel time, crash damages, environmental impacts, and other considerations for which monetary values are not directly or easily obtained. The use of these values extends the applicability of all of the EEA methods described here.

Net Present Value (NPV) or Present Worth of Costs (PWC)

The Net Present Value (NPV) method discounts all projected costs and benefits to convert them into present currency values (e.g., 2019 dollars) and sums them to produce a present worth of costs (PWC), present worth of benefits (PWB), and/or net present value (NPV = PWB-PWC). All alternatives being considered must be evaluated over the same analysis period (except as described below) and the alternative with the highest NPV is the most economically favorable alternative. If all alternatives being considered have similar benefits, the benefits can be excluded from the analysis and PWC values can be compared to determine which alternative is the most cost-effective (i.e., which has the lowest PWC).

The NPV or PWC approach is commonly used to select from among various pavement preservation, maintenance, rehabilitation and/or reconstruction strategies that are believed to provide comparable levels of performance or benefits to the project's users.

If the alternatives have different analysis periods, the NPV or PWC values can be divided by their respective analysis periods to determine equivalent uniform annual values or costs (EUAVs or EUACs), which can then be compared to determine the most cost-effective alternative. The use of EUAV or EUAC analyses implies that the projected costs and benefits are repeated in subsequent analysis periods so that the computed EUAV or EUAC value is valid beyond the end of the selected analysis period. Pittenger et al. (2011) suggest that EUAC analyses may be particularly well-suited for evaluating pavement preservation strategies because preservation techniques tend to be relatively short-term in nature and EUAC values are more easily related to annual maintenance budgets than PWC values.

Another approach for computing NPV or PWC values when competing alternatives have different analysis periods is to use the same analysis period for all candidate alternatives (generally the shortest of those being considered) and to include the remaining value of each alternative at the end of the analysis period (i.e., salvage value of materials or value of remaining service life) as a "benefit" or "negative cost" at the end of the analysis period.

If benefits vary among the candidate alternatives (e.g., if they provide different levels of service), then the alternatives cannot be compared solely on the basis of costs, and the methods described above may not be appropriate means of comparison. If all benefits can be expressed monetarily, then the benefits can be considered in the same analysis as the costs, discounted similarly, and a decision can still be made based on the results of the analysis and the overall objective (e.g., to maximize net benefits or minimize net costs). Another option for analyzing monetarily expressed costs and benefits is benefit-cost analysis (BCA), which is described below.

NPV and EUAC analyses are direct options that will yield consistent and correct results for the selection of pavement design, maintenance, preservation, and rehabilitation activities when performed using reasonable inputs, especially for the costs and timing of all future activities and the selection of the discount rate (which is discussed later in this chapter).

Benefit-Cost Ratio (B/C)

In simplest terms, the benefit-cost ratio (B/C) approach involves computing a ratio of net overall benefits to net overall costs and selecting alternatives (for project-level analyses) or projects (for network-level analyses) with the highest computed ratios of benefits-to-costs. The variants of this approach that are most commonly applied in pavement management applications are described below.

B/C typically requires that all benefits and costs that differ between alternatives be expressed monetarily. Costs and benefits that are common to all considered alternatives can be omitted from the analysis, whether they can be monetized or not. For example, if all alternatives offer the same level of serviceability for the same levels of traffic over the analysis period, then that benefit can be excluded from the analyses and the analyses can proceed with consideration of all other costs and benefits that can be expressed in monetary units.

The present worth of benefits (PWB, computed using conventional economic analysis techniques) is divided by the present worth of costs (PWC) to obtain the simple benefit-cost ratio. In the analyses of project-level alternatives, any alternative with B/C > 1 may be considered acceptable and the option with the highest B/C is often preferred. In network-level analyses, projects may be selected for programming based on the overall ranking of project B/C values.

Unfortunately, this approach can lead to the selection of alternatives and strategies that are incorrect and inconsistent with those developed using conventional NPV or EUAV analyses. Consistent economic analysis results can be assured through a process called "incremental" analysis (Riggs and West 1986). This process involves computing B/C for each competing alternative, discarding those with B/C <1 (i.e., costs exceed benefits), ranking the remaining alternatives from least costly to most costly, and then, beginning with the least cost alternative, compute the incremental B/C ratios with the next higher cost alternative (i.e., compute ΔB/ΔC as the change in benefits between the two alternatives divided by the change in cost between the two alternatives). When ΔB/ΔC <1, the added costs outweigh the added benefits and the higher cost alternative is rejected. When ΔB/ΔC >1, the added benefits outweigh the added costs and the lower cost alternative is rejected in favor of the higher cost alternative. The process continues with increasingly higher cost alternatives until only one alternative remains. The remaining alternative may or may not be the one with the highest simple B/C.

Because of their relative simplicity, PWC and EUAC analyses are generally preferred over B/C for project-level economic analyses.

Internal Rate of Return (IRR)

The discount rate that can be applied to a stream of benefits and costs to produce NPV = 0 is referred to as the internal rate of return (IRR). This value represents the theoretical effective rate of return (based on the assumed and monetized benefits) based on the monetized costs invested in the strategy over the analysis period. Simple IRR analyses select the alternative that provides the highest IRR, but, like B/C, this type of analysis must be conducted incrementally to ensure the selection of strategies that are correct and consistent with NPV and EUAV analyses. IRR is rarely used in pavement management activities.

Key Issues in Discounted Cash Flow Methods

There are several limitations to the use of discounted cash flow methods (i.e., life-cycle cost analyses or LCCA) in pavement strategy selection that, if not understood and used correctly, can provide false support for poor choices. The accurate estimation of life-cycle activity costs and timing is the most important factor in conducting a good pavement LCCA (Hallin et al. 2011). Additional important considerations are described below.

Discount Rate

The "real" discount rate, which represents the combined effects of interest and inflation rates, is used to estimate the present value of all future costs (and benefits, if applicable). Higher discount rates reduce the present value of future costs by a greater amount than do lower discount rates; a zero discount rate values future costs the same as current costs; and negative discount rates increase the present value of future costs above those of current costs.

The discount rate used in most LCCA is sometimes called the "real discount rate" and can be defined as follows:

$$"Real"\ Discount\ Rate = \frac{Nominal\ Discount\ Rate - Inflation\ Rate}{1 + Inflation\ Rate} \qquad \textit{Equation 5-1}$$

where:

"Nominal" Discount Rate	=	the cost of funds (interest rate) or the potential return on an investment forgone (opportunity cost); and
Inflation Rate	=	the rate of future increases in costs due to the erosion of monetary value.

It is worth noting that this equation suggests that it is the difference between "interest" and "inflation" that is most relevant and that the absolute values of these two parameters are relatively unimportant!

The rate of inflation for the goods and services required to design, construct, and maintain pavements may be significantly different from the official government rate of inflation for consumers. For example, the National Park Service (citing U.S. Bureau of Labor Statistics data) has noted that the U.S. Consumer Price Index (which measures the prices that consumers pay for a "fixed basket" of goods and is the most common measure of inflation in the U.S.) rose 3 percent annually while the producer price index for highway and street construction (PPI-HSC) had risen almost 10 percent annually (NPS 2009). The CPI has risen by 0.1 to 3.0 percent annually since 2010 and rose by 1.9 percent in 2018 (US DOL BLS 2018); the BLS discontinued the PPI-HSC metric in 2010, so more recent direct comparisons with the CPI are not readily available. However, it still seems clear that the use of a discount rate that is based on market interest rates and the rate of inflation of a standard consumer shopping basket is inappropriate for economic analyses in the highway and heavy construction industries. Furthermore, the use of "standard" general discount rates may lead to the selection of construction, rehabilitation, maintenance, and preservation alternatives that are not the most cost-effective.

In addition, Mack (2012) suggests that different materials and paving products (e.g., asphalt and concrete paving materials) may undergo different rates of inflation and should, therefore, be evaluated using different discount rates.

The traditional public-sector view of the discount rate is that it should reflect "social opportunity costs"; in other words, the "interest" part of the equation should represent the potential return (i.e., additional benefits to the public) that would have been realized by spending the funds in a different sector of the economy (e.g., on public schools or social programs or other budget items). It is very difficult to quantify the relative values of investing in one area of the public sector versus any other, so most public sector economists use a generalized discount rate that is based on the returns provided by government bonds (e.g., Treasury Bill yields) and forecast overall inflation. In the U.S., these figures have historically resulted in the use of a discount rate of between 3 and 4 percent, depending upon the length of the selected analysis period (although recent discount rates computed in this manner would have been much lower, or even negative – see OMB Circular A-94).

Possible fallacies in this view include the following:

- Government agencies typically invest very little (if any) money at prevailing interest rates; budgets are typically spent during the budget cycle and accrue no long-term interest.
- Most highway funds come from taxes and tolls with some funding from the sale of bonds. Therefore, the cost of money to public sector agencies is typically very low.
- In many instances, highway funds are "dedicated" and savings produced by selecting one design or rehabilitation alternative over another cannot be used for other purposes such as education, social programs, etc. Therefore, there is no "cost of opportunity forgone."

Therefore, for many highway agencies, "nominal discount" or "interest" rates approach zero.

The most accurate economic analysis results will be obtained using a discount rate that reflects the actual cost of funds to the agency (e.g., the average rate of return on portions of the income stream that are invested, or the weighted average cost of money when bonding is used for funding) and the actual rate of inflation for the appropriate segment of the economy (and the appropriate geographic region of the country). If the "interest" portion of the discount rate (i.e., opportunity cost or cost of funds) is low and the rate of inflation for construction services is relatively high (compared to the CPI), the real discount rate may approach zero (and may even be negative). This concept may seem foreign, but it may be most appropriate, especially in an economic environment where the cost of money is low (or zero).

The above discussion just scratches the surface of the issues and arguments that surround the choice of an appropriate discount rate for EEA of pavement management activities. A thorough understanding of the factors that influence discount rate is important, however, because selected discount rate values can significantly impact the economic analysis and can result in the selection of different competing alternatives.

End-of-Analysis (Residual) Value: Salvage Value vs. Remaining Service Life Value

As noted previously, it is often necessary (i.e., when performing NPV analyses on alternatives with different analysis periods) to truncate an analysis period for one alternative to match the analysis period of another alternative. In such cases, one generally assigns a "remaining service life" or "residual" value (generally a benefit or negative cost) to the pavement cost stream at end of the shortened analysis period to capture the value of the remaining pavement life (see figure 5-1). If the pavement's service life has been fully consumed at the end of the analysis period, one can (and should) determine the "salvage" value of the materials that will be derived from the pavement structure. If the existing pavement material will not be removed and replaced, the "salvage value" may be computed as the value of the existing pavement as a support layer for an overlay at the end of the analysis period (i.e., recycling or "repurposing" the pavement in place).

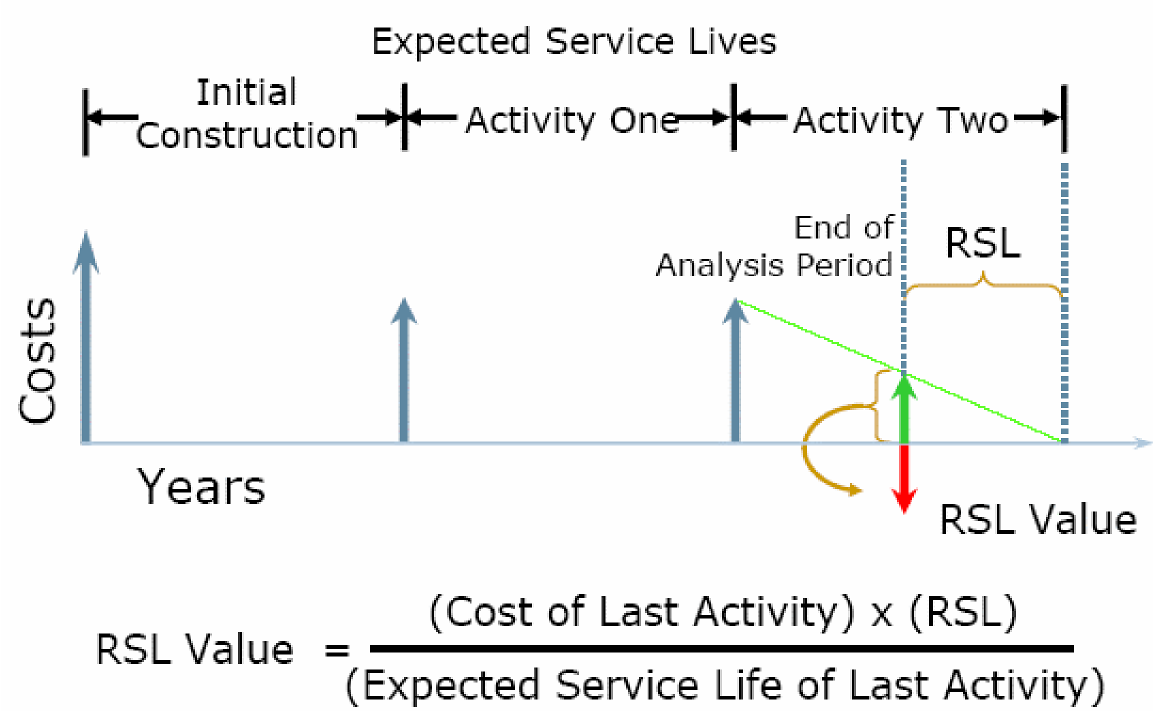

Figure 5-1. Illustration of determination of remaining service life (residual) value (ACPA 2011).

These options are mutually exclusive for any given economic analysis; that is, no analysis should include both a salvage value and a remaining service life value. Whichever end-of-analysis value is selected (if any), it should reflect what the agency realistically expects will be done with the pavement structure at the end of the analysis period. ACPA (2011) and West et al. (2012) provide summaries of U.S. state highway agency practices concerning the inclusion of salvage and remaining service life values in their LCCAs.

Salvage values may be properly applied as either a positive cash flow (benefit) at the end of the current project analysis (e.g., when an agency retains the material for unspecified future use) or as a reduction in cost at the beginning of the next project analysis. If different materials from the same project are used in different ways, portions of the salvage value may be allocated to both places.

User Cost Estimates

User costs originate primarily from vehicle operating costs, delay costs, and crash costs (Walls and Smith 1998). The value of road users' time is a subject of great debate. User delay costs are generally computed in consideration of vehicle class, trip type (urban or rural), and trip purpose (business or personal). Details concerning the computation of user costs can be found in NCHRP (2004), and software for computing these costs is a part of the FHWA *RealCost* LCCA program (FHWA 2010) and the CA4PRS software (Caltrans 2011).

While there is no doubt that user costs should be considered in pavement decision-making processes, LCCA of these costs is often computed and considered separately from LCCA of agency costs because: 1) their quantification is subject to debate and uncertainty, 2) user costs "do not debit agency budgets as do agency costs" (FHWA 2002), and 3) computed user costs on some projects can drive the decision process to options that the agency cannot afford.

Deterministic LCCA vs. Probabilistic LCCA

The use of fixed values for all LCCA inputs (e.g., activity timing, costs, discount rate) to produce a single output value is referred to as the deterministic approach to LCCA. While this approach is relatively simple and requires few inputs, it fails to adequately account for either the variability in actual initial costs and discount rates over time or the uncertainty in the timing and costs of planned maintenance and rehabilitation activities. Furthermore, the output of a single value (i.e., PWC or EUAC) without some statement to qualify that value may imply a degree of certainty in the conclusion that is inappropriate (FHWA 2010). Sensitivity analyses can provide a sense of the accuracy of deterministic LCCA results.

The probabilistic approach to LCCA is more realistic in that it uses statistical descriptions of the probable distribution of values for each input (e.g., a mean and standard deviation for each normally distributed input value) to account for the input-associated variability that creates uncertainty in the outputs of the analysis, which helps quantify the risk in any decisions that are made based on the outputs. A distribution of output values is produced to provide users with information for understanding the variability of the results and the confidence that can be placed in the analysis.

Key Strengths of LCCA-Based Methods

- LCCA is widely accepted and recognized as a tool for aiding in making strategy selection decisions.
- LCCA provides reasonable guidance in decision-making when all inputs are known with reasonable certainty.
- PWC and EUAC analyses are direct, relatively simple, and provide consistent, unambiguous results.

There are many well-developed LCCA tools available to facilitate the analyses, with the FHWA's *RealCost* (FHWA 2010) being most prevalent (Rangaraju, Amirkhanian, and Guven 2008).

Key Disadvantages of LCCA-Based Methods

- BCA and IRR analyses must be conducted using a more complicated incremental analysis to produce results that ensure the most cost-effective use of funds (and consistency with PWC and EUAC methods).
- LCCA-based methods cannot directly consider differences in benefits (or costs) that cannot be accurately quantified monetarily. For example, at the project level, two strategies that begin and end with the same level of serviceability may have very different impacts on the traveling public if one serviceability curve is a smooth or linear change over time and the other is a saw-tooth (because of frequent minor treatments). Similarly, at the network level, LCCA alone cannot differentiate the relative benefits of two projects with identical costs and performance results when one carries a high-volume of commercial traffic and the other carries a lower volume of mixed traffic. Other potential decision factors that may be difficult to accurately monetize include user costs, environmental impacts, value of crash/fatality avoidance, etc., although guidance in quantifying and monetizing many of these factors can be found in publications from the Federal Highway Administration (FHWA 2010) and the World Roads Association (WRA–PIARC 2012).

- Deterministic LCCA inputs can be easily manipulated to obtain results that justify pre-determined strategy selections.
- Minimizing LCC at the project level does not necessarily result in network-level optimization, especially when funding needs outstrip available budgets.

Cost-Effectiveness Analyses (CEA)

Cost-effectiveness analysis (CEA) is an evaluation technique that produces a cost-effectiveness ratio (CER) that is obtained by dividing the effectiveness (E) of the treatment by the cost (C) of the treatment. For example, the cost-effectiveness of road safety measures can be defined as the number of accidents prevented per unit cost of implementing the measure. The costs of each treatment and its effects (e.g., the need for additional activity as a result of implementing the treatment) must be expressed in terms of NPV or EUAV. CEA can be interpreted as units of effectiveness obtained for each unit of cost incurred, so the higher the ratio, the more effective the treatment (WRA–PIARC 2012).

This type of analysis is most useful for comparing projects and strategies where the benefits are not easily quantified in monetary terms (e.g., the value of crash avoidance, reductions in fatalities, reductions in travel time, etc.), so it complements conventional EEA or LCCA and may be used in addition to (or in lieu of) them.

CEA can be applied to evaluate a single benefit (as in the example above) or to evaluate and balance several types of benefits in a single analysis using a concept called weighted cost-effectiveness and a technique called Multi-Criteria Analysis (MCA). Details concerning the calculation and use of CEA and MCA can be found in WRA–PIARC (2012) and Markow (2012) and are not repeated here because of their apparent limited applicability to the evaluation of pavement preservation strategies (except as a supplemental factor to economic analysis in decision-making).

Key Strengths of CEA

- Calculating CEA generally requires only a quantification (but not the monetary value) of the general benefits of a measure or treatment and the cost of implementation. This is typically much easier and subject to less debate than developing monetary estimates of value for a full LCCA-based economic analyses.
- Weighted CEA or MCA can be used to compare the overall cost-effectiveness of any given strategy with respect to several non-monetized criteria (e.g., injury accidents avoided, lives saved, travel time saved, etc.). The benefits of multiple criteria may accrue over different analysis periods, requiring the use and summation of annualized costs (similar to the development of EUAC) for each criterion to produce an overall cost-effectiveness value.
- CEA offers the potential to highlight the safety effects of various treatments (e.g., diamond grinding versus resurfacing), compared to more comprehensive LCCA where the apparent importance and impact of safety measures may be reduced relative to the economic impact of other factors.
- CEA may be most appropriate for small projects, such as consideration of safety improvement measures for local road projects.

Key Disadvantages of CEA

- It can be difficult to estimate benefits over an appropriate unit of implementation where the benefits do not accrue uniformly. For example, construction project limits may include several intersections with different potentials for crashes and, therefore, different potential improvements. Analyses could be conducted over the entire project or over subprojects that represent individual intersection locations.
- Weighted CEA require the development of weighting factors, which are often established subjectively and have an associated potential risk of bias.
- CEA can be used for ranking various treatments, but (unlike LCCA-based analyses) cannot be used for determining whether a project should be undertaken or not.
- CEA alone generally disregards the effects of safety measures on mobility measures and the environment, and these effects are often of sufficient importance to drive the decision-making process.

Recent Concepts for Strategy Selection Based on EEA

Basic LCCA techniques (i.e., PWC, EUAC, B/C and IRR) can easily be used to identify economically preferred project-level strategies and can also be used to offer insights into the effects of network-level allocations of limited resources. These applications are described in most economic analysis texts as well as in many pavement industry- and government-published reports and documents. These approaches are assumed to be common practice and are not repeated here.

The following sections describe other more recently developed concepts for pavement project-level strategy selection and network-level activity programming.

Remaining Service Interval (RSI)

RSI is not an alternative approach to economic analysis. It is an approach for developing project and network maintenance and rehabilitation strategies by focusing on when and what treatments are needed to provide an acceptable level of service (LOS) over the analysis period at the lowest life-cycle cost (LLCC)—typically the lowest PWC or EUAC. It is based on the idea that pavements are repairable systems that do not "die," so the concept of a remaining service "life" is inappropriate. RSI holds that pavements are better defined in terms of a string of time intervals to the next preservation, rehabilitation, and/or reconstruction activities, as shown in the inset table for hypothetical "Pavement Section 1" in figure 5-2.

The structured sequence of preservation, rehabilitation, and reconstruction actions associated with a specific pavement section is selected using pavement performance models and conventional LCC considerations to provide acceptable levels of service over the analysis period (life cycle) at the lowest life-cycle cost, which can be computed using the cash flow diagram shown in the top portion of figure 5-2.

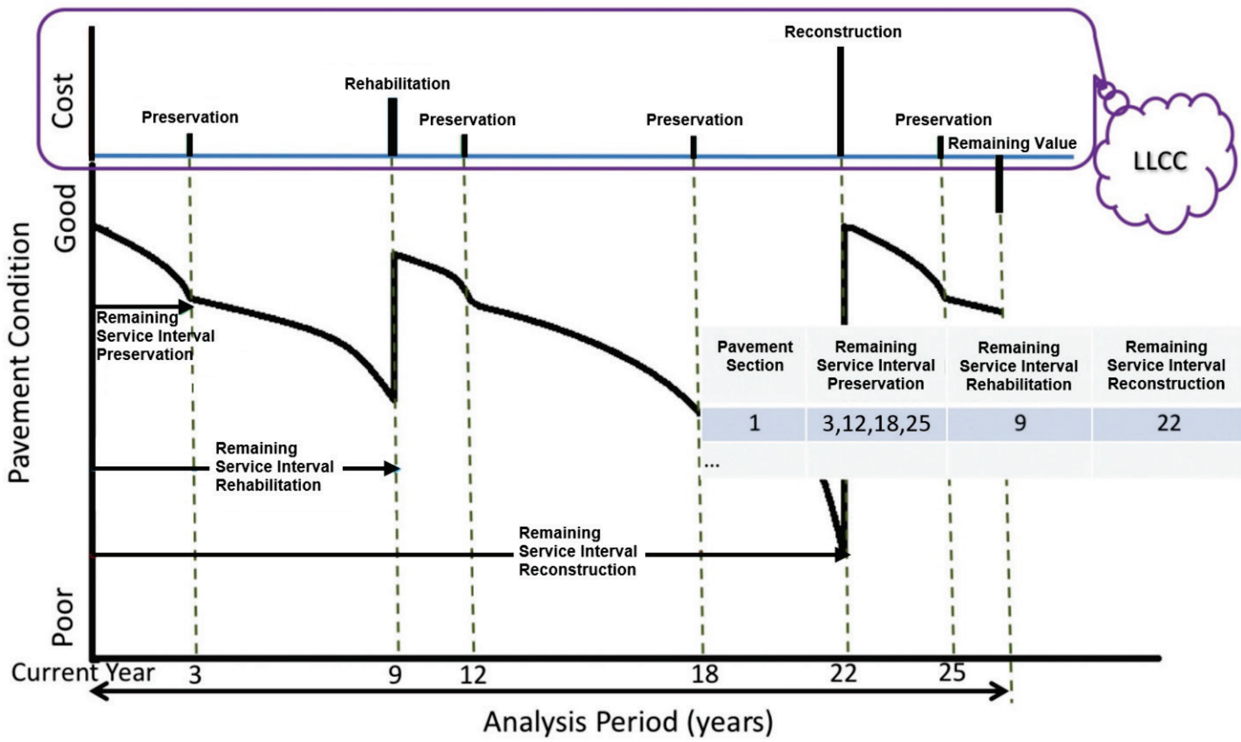

Figure 5-2. Illustration of RSI concept (FHWA 2016).

For example, a demonstration of project-level RSI concepts was performed for a specific pavement section in California by evaluating candidate treatment strategies using mechanistic-empirical pavement analysis software (CalME) to predict pavement performance. If the predicted performance met previously defined LOS requirements over the analysis period, the predicted performance and associated activity costs were used as inputs to the LCC analysis. If predicted performance failed to meet LOS requirements at any time during the analysis period, the treatment types and/or timing were changed and reanalyzed. This process resulted in the identification of acceptable treatment sequences and their costs, and allowed quantification of potential financial impacts of applying treatments too early or too late (or of applying less-effective treatments).

Network-level applications of the RSI concept have been promoted as representing "the ideal management system, where decision-making consider[s] the optimal treatment selection, not based on thresholds, but based on identifying the optimal treatment type and timing while maintaining an acceptable or above-acceptable LOS" (FHWA 2016). Network-level validation of RSI concepts has been performed using Maryland State Highway Administration models, which resulted in preservation treatments generally being applied when pavements were in better condition and rehabilitation treatments generally being applied when pavements were in worse condition.

It is important to emphasize that the RSI concept does not provide alternative techniques for assessing network health or for making budget allocation decisions.

Strengths

- RSI appears to offer a better approach for determining ("optimizing") and communicating future preservation, maintenance, rehabilitation and reconstruction needs than using a single value parameter, such as remaining service life (RSL), which is generally unrelated to the time until the next pavement treatment in an optimized strategy (FHWA 2016).

- RSI offers a way to evaluate the potential service-level impacts of alternate budget scenarios (network level) and alternate treatment strategies (project level).
- RSI concepts can help agencies move from "worst first" programming strategies to LLCC-driven strategies, which should lead to lower annual network costs and more consistent network conditions over the analysis period (FHWA 2016).

Weaknesses

- RSI relies on existing techniques for economic analysis (i.e., LCCA), which suffer from sensitivity to selected discount rate, assumptions concerning projected activity costs, etc. These issues can be addressed to some extent using stochastic (rather than deterministic) LCCA.
- While RSI can be used to identify strategies with the LLCC for *acceptable* levels of service, it does not (in its current forms and applications) appear to offer the ability to maximize units of service purchased at any given LCC or funding level (i.e., it does not consider the area under the performance-time curve in selecting the preferred strategy).
- Similarly, RSI does not (in its current forms and applications) appear to be able to differentiate between the amounts of service provided by competing alternatives (i.e., it does not consider the number and classification distribution of vehicles that use a given facility that is subject to a strategy-specific performance-time curve).
- It is unclear whether RSI techniques have been normalized for project length, which would seem to be required for network-level applications.
- RSI does not appear to be a good tool for making budget allocation decisions.

Cost-of-Ownership (a.k.a. dollars/lane-mile/year or DLMY analyses)

DLMY analyses treat pavement investment decisions as an optimization problem where the goal is to maintain a network of roads in the best possible acceptable condition considering limited available funding. The item to be optimized (maximized) in this approach is the amount of service or pavement performance provided and the primary constraints are the available resources for network preservation, rehabilitation, and reconstruction—money—and the lower limits of acceptable performance (e.g., limiting thresholds of acceptable ride quality, structural capacity, noise, safety, etc.).

The concept is based on work by Galehouse and Sorenson (2007), who noted that maintaining a network of x lane-miles of roadway for 1 year requires exactly x lane-mile-years of purchased service life (through maintenance, preservation, and construction investments) annually to maintain the network at the same service level. The purchase of additional service life will increase the overall level of the network service while shortfalls in annual service life purchases will result in decreases in overall network service level. Therefore, the objective of the DLMY analysis can be assumed to be the maximization of lane-mile-years of acceptable service that are "purchased" for any given budget level or per dollar spent. Conversely, one could minimize the quantity "$/lane-mile-service year." Either approach would result in the selection of strategies that "purchase" the greatest amount of service for a given level of budget.

It is worth noting that the computation of DLMY described above is essentially an EUAC analysis that has been normalized for project length and performed at a discount rate of 0 percent over an analysis period that is assumed to include no other direct costs or other monetized events.

Galehouse and Sorenson (2007) provides an illustrative example of using DLMY concepts to evaluate baseline network replenishment conditions (i.e., determining whether the current allocation of funds to reconstruction, rehabilitation, and preservation activities is sufficient to maintain the network status quo by providing a number of lane-mile-years of added service each year that is equal to the number of lane-miles in the network). Upon determining that the baseline allocation of activities in this hypothetical network falls 1654 lane-mi-years (2662 lane-km-years) short of what is needed to maintain the network status quo, 9 lane-mi (14 lane-km) of reconstruction work and 5 lane-mi (8 lane-km) of rehab work are removed from the program (at a cost reduction of $6.1M and a reduction of 345 lane-mi-years [555 lane-km-years] of serviceability) in favor of 31 lane-mi (50 lane-km) of concrete resealing, 16 lane-mi (26 lane-km) of thin HMA overlay, 44 lane-mi (71 lane-km) of microsurfacing, 79 lane-mi (127 lane-km) of chip seal, and 506 lane-mi (814 lane-km) of crack sealing (at a cost increase of $5.6M and an increase of 1999 lane-mi-years [3217 lane-km-years] of serviceability). This example demonstrates how DLMY can be used to rebalance network programs to prevent deterioration of the overall network by deferring work on some of the pavements that are already in poor condition in favor of preserving other pavements in good or better condition.

The DLMY approach promulgated by Galehouse and Sorenson (2007) considers only the initial cost of each treatment, whether a 2-year pavement preservation technique or a reconstruction with a 30-year expected service life. This implies that no additional work is required to achieve the design service life or life extension of the treatment. While this negates the need for discounting the cash flow (and eliminates the uncertainties associated with selecting an appropriate discount rate and the timing and nature of future maintenance and rehabilitation activities), it is probably not an appropriate assumption for longer-term strategies (although it may be reasonable for short-term fixes like many pavement preservation activities).

Li et al. (2017) report that Washington State has adopted a slight variation of this type of approach to managing their asphalt pavement network and have found that "cost-effectiveness [i.e., DLMY] is a very useful performance measure that can be used for making good pavement management decisions." The key performance measure used is Equivalent Uniform Annual Cost (EUAC) per lane-mile (or $/lane-mile/year, DLMY). The use of EUAC suggests that future costs may be considered over the service life or life extension provided by each strategy and it requires the selection of an appropriate discount rate. Li et al. (2017) report that Washington State uses a discount rate of 0 percent to provide a simple indicator of annual costs of ownership while a discount rate of 4 percent is used when comparing alternative strategies to consider the time value of money. "Acceptable" pavement performance thresholds are established for pavement roughness, rutting, faulting, and friction, and an effort is made to identify strategies that provide acceptable performance while maximizing service life additions within the allocated budget (or, alternatively, to minimize the unit cost of service life additions). Annual cost savings to WSDOT are estimated to approach $80M/year by 2025.

Similar principles have been adopted by at least a few roadway management agencies in other countries (e.g., Scotland and the Shetland Islands) in developing Road Asset Management Plans (RAMPs) to develop pavement life-cycle plans (i.e., preservation strategies) that help "to make long-term predictions of network deterioration and maintenance needs" and allow them to compare the impact of alternative life cycle plans and adopt the approach that delivers the required service but also makes the best use of resources." (Transport Scotland 2007).

Strengths

- DLMY offers a way to evaluate the potential service-level impacts of alternate budget scenarios (network level) and alternate treatment strategies (project level).
- DLMY approaches can help agencies move from "worst-first" programming strategies to strategies that greatly increase the "cost-effectiveness" of each monetary unit spent on pavement reconstruction, rehabilitation, maintenance, and preservation while ensuring that pre-determined minimum performance thresholds are met and overall network conditions are maintained or improved. This will generally be accomplished by allowing some pavements in poor condition to remain in poor condition in favor of maintaining and preserving many more lane-miles of pavement in good or better condition.
- In its simplest form (i.e., when based solely on initial strategy costs and design service life or service life extension for each strategy with no consideration of future activities or the time-value of money), DLMY may be best suited for developing and managing pavement preservation strategies, which typically have relatively short performance life expectations. The economic analysis of these types of activities using DLMY are unlikely to be greatly affected by failing to address and consider downstream activities.

Weaknesses

- When DLMY concepts are implemented with consideration of downstream activities (e.g., consideration of periodic maintenance activities that are required to reach the design service life for reconstruction options), they rely on existing techniques for economic analysis (i.e., LCCA), which suffer from sensitivity to selected discount rate, assumptions concerning projected activity costs, etc. These issues can be addressed to some extent using stochastic (rather than deterministic) LCCA in the DLMY analysis.
- As described by Galehouse and Sorensen (2007), DLMY does not offer the ability to consider any other costs or benefits (e.g., user costs, reductions in traffic delay costs, safety benefits, environmental impacts, etc.).
- The DLMY analysis does not reflect differences in the quality of service provided by any given strategy (i.e., it assumes that the quality of 1 lane-mile-year of service life extension obtained through a specific pavement preservation activity (say, joint resealing) is the same as the quality of 1 lane-mile-year of service life extension obtained through any other preservation activity (say, diamond grinding). In other words, the benefits provided by any given increase in service life are assumed to be equal for any given strategy applied to any given pavement in the network. It seems unlikely that this is generally true.
- DLMY techniques do not (in published theory or practice to date) appear to offer the ability to maximize units of service purchased at any given LCC or funding level (i.e., it does not consider the area under the performance-time curve in selecting the preferred strategy).
- Similarly, DLMY does not (in its current forms and applications) appear to be able to differentiate between the impact of improved service levels on competing projects given the numbers and classification distributions of vehicles that use each facility.

- Currently described and documented DLMY techniques do not appear to consider pavement category or function (e.g., urban interstate vs. rural local road) in establishing priorities for resource allocation.

Summary

Discounted cash flow analyses (i.e., PWC, EUAC and B/C), also known as life-cycle cost analyses (LCCA), are typically used to evaluate agency (and sometimes other) costs associated with alternative strategies for designing, constructing, rehabilitating, maintaining and preserving pavement systems over a specified analysis period. PWC and EUAC analyses are the most easily conducted and will provide consistent ranking and selection of alternatives for any given set of inputs. B/C analyses are sometimes preferred when the value of benefits varies between alternatives and can be monetized for evaluation, but incremental B/C analyses (i.e., examining the ratio of incremental benefits received for each increment of additional cost) are often required to rank and select alternatives consistently with NPV and EUAV analyses.

All LCCA are subject to significant limitations, particularly with regard to assumptions concerning the nature, cost, and timing of future maintenance and rehabilitation activities and concerning the selection of appropriate discount rates. Inaccurate or inappropriate estimates of activity timing and cost and/or discount rate can lead to the selection of strategies that are not the most cost-effective. Stochastic (probabilistic) LCCA can provide insights to the sensitivity of the strategy selection to input assumptions. User costs and benefits are important, but should generally be excluded from agency cost analyses and should be discounted differently and considered separately.

Two newer approaches to project-level strategy selection and network-level programming have been developed: Remaining Service Interval (RSI) and Cost of Ownership or Dollars per Lane-Mile per Year (DLMY). Both have a basis in LCCA techniques. The DLMY approach seems to offer the most promise for widespread implementation, particularly for pavement preservation analyses, possibly with some modifications.

In simplest form, DLMY attempts to select the alternative or strategy that maximizes the amount of service "purchased" (expressed as $/lane-mile/year) for the cost of treatment. At the network level, allocation of resources to various projects is adjusted to ensure that the number of lane-mile-years of service purchased annually is at least equal to the number of lane-miles in the network; this ensures maintaining (or improving) the overall status quo. Discount rate is irrelevant (or is effectively zero) when the treatment cost is spread only over the time until the next treatment, which eliminates a major concern with conventional LCCA over long analysis periods. This may be a reasonable approach for short-term pavement preservation activities, but is probably inappropriate for new construction and longer-life rehabilitation activities (e.g., concrete overlays) which typically require periodic maintenance and rehabilitation to achieve their design service lives.

DLMY (and most other strategy selection approaches) are also typically limited by their current inability (in practice, at least) to differentiate between quantity of service (e.g., $/lane-mile/year) and quality of service (e.g., $/lane-mile/[area under the serviceability-time curve for one year]). It may also be reasonable to attempt to normalize these values for the level and composition of traffic using each facility (e.g., recognizing that there is generally more value to maintaining a higher level of serviceability on a high-volume road with heavy commercial traffic than there is on a low-volume local road). Effort should be devoted to incorporating these concepts for DLMY and other analyses to extend their general usefulness.

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CHAPTER 6. SUMMARY

Background

Concrete pavement preservation is defined in this report as "preserving the existing concrete pavement structure to extend its service life for as long as possible, by arresting, greatly diminishing, or avoiding the pavement deterioration process." This can be achieved through three fundamental approaches:

1. Designing and constructing concrete pavement that is durable and remains structurally adequate and relatively distress-free throughout a long service life.
2. Using overlays (asphalt or concrete) as a preservation treatment to maintain the structural capacity and serviceability of the existing concrete pavement.
3. Maintaining the serviceability of the existing concrete pavement using CPR treatments.

This report reviews the primary factors affecting concrete pavement performance and the strategies for concrete pavement preservation. The approaches for evaluating the condition of existing concrete pavements and the economic analysis techniques that can be employed to evaluate the cost effectiveness of various preservation strategies are also included.

The literature review summarized in this report documents the information necessary for the development of an approach for concrete pavement preservation that is consistent with the definition for concrete pavement preservation. The review summarizes suitable approaches for pavements that are categorized as follows:

- Category 0 – Concrete pavements with no apparent or impending distresses (e.g., recently constructed pavements). These pavements are candidates for monitoring to identify the initiation and subsequent progression of structural distresses (in addition to functional distresses).
- Category 1 – Pavements that are not experiencing structural deterioration but require functional preservation.
- Category 2 – Pavements in which structural improvements (e.g., overlays) have been used to prevent or arrest the development of structural deterioration.
- Category 3 – Pavements in which the progression of structural deterioration is continuous and treatments are being applied only to maintain some acceptable level of serviceability until a major structural rehabilitation or reconstruction can be performed.

Evaluation of Existing Concrete Pavement

A project-level pavement evaluation is required as a precursor to preservation or rehabilitation. Many SHAs combine a quantified measure of structural distress with some kind of quantified measure of ride quality to create an overall pavement quality metric that is used as a basis for recommending preservation or rehabilitation treatments.

Some patterns and consistencies in the treatment plans based on ratings can be seen across the agencies. Despite inconsistencies in the exact calculation of the pavement performance scale (for example, some agencies consider only IRI, whereas others may integrate their structural distress values into the rating), poor performing pavements generally require major rehabilitation or complete reconstruction. While some agencies require preventive maintenance for very good pavements, others make no recommendation for action.

Strategies for Concrete Pavement Preservation

Three broad preservation strategies, working in concert, can be used to help ensure the longevity of concrete pavements. The first is to design and construct concrete pavements for long-life, which requires the selection of durable materials, the development of a durable concrete mix design, the selection of design features that reduce load- and environmentally induced stresses, and the effective construction of the pavement. The second strategy involves the use of overlays, which have traditionally been used to restore functionality to deteriorated concrete; but in this context, they are applied to structurally sound concrete pavements as a way to extend pavement life. The third strategy involves the application of CPR techniques, not in a reactive manner but instead applied to structurally sound concrete pavements as a means to maintain a high level of serviceability, including ride quality, noise reduction, and surface friction.

Engineering Economic Analysis for Strategy Selection

Several different methods and techniques are currently in use for engineering economic analysis and pavement management strategy selection. Two newer approaches related to project-level strategy selection and network-level programming were reviewed: Remaining Service Interval (RSI) and Cost of Ownership or Dollars per Lane-Mile per Year (DLMY). Both have a basis in LCCA techniques, with the DLMY approach offering promise for widespread implementation, particularly for pavement preservation analyses.

The DLMY approach (and most other strategy selection approaches reviewed in Chapter 5) are typically limited by their current inability (in practice, at least) to differentiate between quantity of service (e.g., $/lane-mile/year) and quality of service (e.g., $/lane-mile/[area under the serviceability-time curve for one year]). It may be reasonable to attempt to normalize these values for the level and composition of traffic using each facility (e.g., recognizing that there is generally more value to maintaining a higher level of serviceability on a high-volume road with heavy commercial traffic than there is on a low-volume local road). Efforts should be devoted to incorporating these concepts for DLMY and other analyses to extend their general usefulness.

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