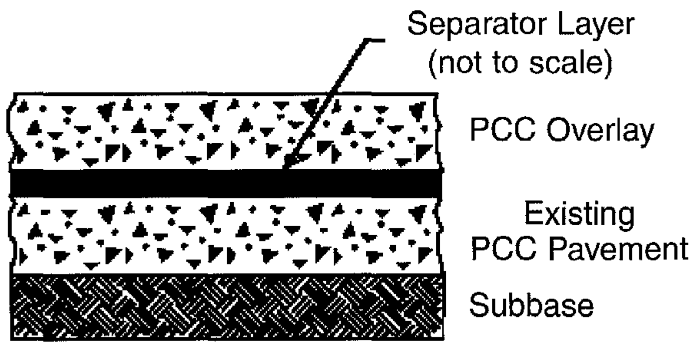
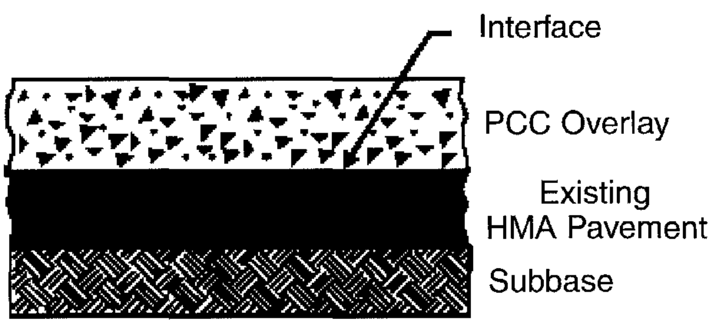
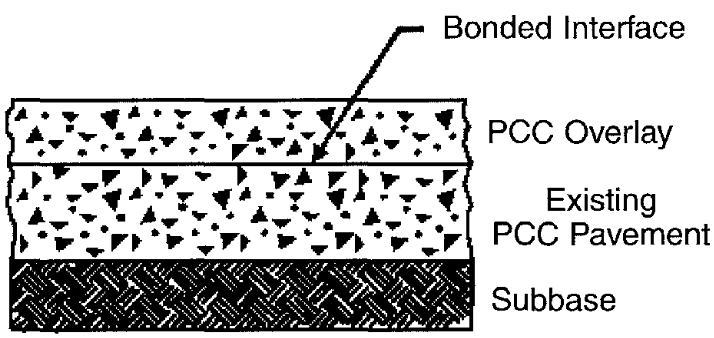
PORTLAND CEMENT CONCRETE OVERLAYS

STATE of THE TECHNOLOGY SYNTHESIS







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Federal Highway Administration

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16. Abstract

This report presents the latest information on the design, construction, and performance of portland cement concrete (PCC) overlays. It describes the four types of PCC overlays that are commonly used in highway pavement applications: bonded PCC overlays, unbonded PCC overlays, conventional whitetopping, and ultra-thin whitetopping. Recommended applications, critical design elements, current overlay design methodologies, recommended construction practices, and performance highlights are described for each overlay type. Information is also provided on the selection of PCC overlays as possible rehabilitation alternatives for existing pavements. Taken together, this document addresses the current "state of the technology" of PCC overlays placed on both existing PCC pavements and on existing hot-mix asphalt (HMA) pavements.

As described in this document, there has been significant progress over the last decade in improving the performance of PCC overlays. However, there remain several critical design and construction areas that are currently not adequately addressed, and these suggested future research needs are listed in this report.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

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

APPROXIMATE CONVERSIONS FROM SI UNITS

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

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

(Revised September 1993)

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CHAPTER 1. INTRODUCTION

1.1 Background

Portland cement concrete (PCC) overlays are increasingly being used as a rehabilitation technique for both existing PCC and hot-mix asphalt (HMA) pavements. The use of PCC overlays offers the potential for extended service life, increased structural capacity, reduced maintenance requirements, and lower life-cycle costs when compared with other overlay alternatives.

PCC overlays have been used to rehabilitate existing PCC pavements since 1913, and to rehabilitate existing HMA pavements since 1918 (Hutchinson 1982). Beginning in about the mid-1960s, more and more highway agencies began to search for alternative means of rehabilitating existing pavements, and the use of PCC overlays increased significantly (McGhee 1994). The last decade has seen an even greater increase in the use of PCC overlays, spurred by improvements in PCC paving technology. For example, innovations such as the use of zero-clearance pavers, fast track paving concepts, and high-early strength PCC mixtures have greatly increased the ability of PCC overlays to serve as a viable rehabilitation alternative that can meet almost any paving demand.

In parallel with the increased use of PCC overlays, significant research aimed at advancing the state of the knowledge of PCC overlays has been conducted. One impetus for this research was the Intermodal Surface Transportation Act (ISTEA) of 1991, which included a provision under section 6005 allocating designated funding for the assessment of thin bonded PCC overlays and surface lamination technology. The goals of the assessment were to evaluate the feasibility, costs, and benefits of the techniques in terms of minimizing overlay thickness, initial laydown costs, and time out of service, and also to maximize life cycle durability. As part of this effort, the Federal Highway Administration (FHWA) participated in the funding of 12 test and evaluation projects throughout the country, the performance of which are documented in a recent report (Sprinkel 2000).

Another example of ongoing studies of PCC overlays are those being conducted under the FHWA's Long-Term Pavement Performance (LTPP) program. The LTPP program is divided into two complementary studies: the General Pavement Studies (GPS) and the Specific Pavement Studies (SPS). Under GPS-9, the performance of unbonded PCC overlays is being investigated, and currently 14 projects are being evaluated. Under SPS-7, the performance of four bonded PCC overlay projects is being studied. The long-term monitoring of these projects is expected to provide valuable information on the design and construction of these types of PCC overlays.

Yet another example of PCC overlay research has been in the area of resurfacing HMA pavements with PCC, a process referred to as "whitetopping." In particular, several studies on the use of ultra-thin whitetopping (UTW), a very thin (50 to 102 mm [2 to 4 in]) layer of PCC bonded to an existing HMA pavement, have been conducted, and in the last decade this rehabilitation technique has evolved from a radical rehabilitation concept to a mainstream rehabilitation alternative. Several studies on whitetopping overlays are currently being conducted under the research auspices of the Innovative Pavement Research Foundation (IPRF).

1.2 Purpose of Report

In the last two decades, two National Cooperative Highway Research Program (NCHRP) syntheses have been prepared on PCC overlays: *Resurfacing with Portland Cement Concrete* (Hutchinson 1982), and *Portland Cement Concrete Resurfacing* (McGhee 1994). These are both significant reports that effectively document the status of PCC overlays at the time of their publication. However, as the examples cited above illustrate, there has been considerable work in the area of PCC overlays since the most recent NCHRP synthesis. Moreover, much of the information is scattered and not readily accessible in a single source. Consequently, there is an acute need to assemble and synthesize that information on PCC overlays for use by pavement engineers and practitioners throughout the country.

It is the purpose of this report to synthesize that information and present the latest on the design, construction, and performance of PCC overlays. It is intended to supplement and update the two NCHRP projects and serve to provide the current "state of the technology" of PCC overlays of both existing PCC pavements and existing HMA pavements. As the basis for this work, a comprehensive literature search on PCC overlays was conducted so that the latest and most up-to-date information is provided.

1.3 Overview of Report

This report consists of seven chapters in addition to this one. Chapter 2 presents a general overview of the different types of PCC overlays and describes common materials used in their construction. Chapters 3 through 6 present information on specific overlay types, with chapter 3 describing bonded PCC overlays, chapter 4 presenting unbonded PCC overlays, chapter 5 discussing conventional whitetopping overlays, and chapter 6 describing UTW overlays. In each of these chapters, the latest information on the design, construction, and performance of each PCC overlay type is provided. Chapter 7 presents information on selecting PCC overlays as a rehabilitation alternative. Finally, chapter 8 summarizes the report and suggests future research needs.

Two appendixes are presented in support of the technical report. Appendix A presents an annotated bibliography of reference documents that were identified as part of a comprehensive literature search on PCC overlays. The search was limited to documents produced from 1985 to the present. Appendix B presents a glossary of common terms used in describing PCC overlay design, construction, and performance.

CHAPTER 2. PCC OVERLAY TYPES AND CONSTRUCTION MATERIALS

2.1 Introduction

This chapter presents general information on the different types of PCC overlays that are commonly used in pavement rehabilitation. PCC overlays for both existing PCC and existing HMA pavements are described, including a summary of their defining characteristics and a description of their general use and applicability. This chapter serves only as an introduction to the different PCC overlay types, as more detailed information is presented in later chapters.

Also presented in this chapter is a description of the materials used in the construction of PCC overlays. This includes a summary of PCC paving materials and mix designs, as well as interface materials and other construction incidentals.

2.2 Types of PCC Overlays

2.2.1 *PCC Overlays of Existing PCC Pavements*

PCC overlays of existing PCC pavements are generally classified according to the proposed bonding condition between the new PCC overlay and the existing PCC pavement. They may be placed in a bonded, partially bonded, or unbonded condition, the selection of which depends largely upon the condition of the existing pavement and on the future traffic levels. Table 2-1 summarizes some key characteristics of each of these PCC overlay types (Hoerner et al. 2001). Additional information is provided in the following sections.

2.2.1.1. Bonded PCC Overlays

Bonded PCC overlays consist of a thin layer of PCC (typically 76 to 102 mm [3 to 4 in] thick) that is bonded to the existing PCC pavement (see figure 2-1). These are used to increase the structural capacity of an existing PCC pavement or to improve its overall ride quality, and should be used where the underlying pavement is free of structural distress and in relatively good condition (ACPA 1990a; McGhee 1994).

Perhaps the most important construction and performance aspect of bonded PCC overlays is the achievement of an effective bond between the overlay and the existing PCC pavement. This is needed to create a pavement system that behaves monolithically; if such bonding is not achieved, cracking of the overlay will result due to increased curling and loading stresses. Extensive surface preparation of the existing PCC pavement is required, with the intention of producing a clean, roughened surface that will promote bonding between the two layers. This is commonly and effectively accomplished using shotblasting equipment.

Bonded PCC overlays have been constructed in many different states, including California, Illinois, Iowa, Louisiana, New York, Pennsylvania, South Dakota, Texas, and Virginia, as well as in the countries of Belgium, Canada, Japan, and Sweden. By far the most common bonded PCC overlay type is JPCP, and these have been placed on existing JPCP, JRCP, and CRCP designs (McGhee 1994). Some bonded JRCP overlays have been used on existing JPCP and JRCP, although presently they are rarely used. Texas and Virginia have both constructed several bonded overlays on existing CRCP.

Table 2-1. Summary of PCC overlays of existing PCC pavements (adapted from Hoerner et al. 2001).

	Bonded	Partially Bonded	Unbonded
Typical Thicknesses	• 76 to 102 mm (3 to 4 in)	• 152 to 203 mm (6 to 8 in)	• 152 to 305 mm (6 to 12 in)
How Bonding Condition Achieved	• Cleaning and preparing surface (e.g., shotblasting) • Possible application of bonding agent	• No special surface preparation other than sweeping	• Placement of a separation layer to separate overlay from existing pavement
Condition of Existing Pavement	• Relatively good condition • No materials-related distress	• Fair to moderate condition	• Fair to poor condition
Preoverlay Repair	• Most deteriorated cracks, joints, punchouts	• Limited repair	• Limited repair
Special Design and Construction Considerations	• Achieving bond between two PCC layers • Matching joints of overlay with those in existing pavement	• Matching joints of overlay with those in existing pavement	• Achieving separation between two PCC layers • Mismatching joints of overlay with those in existing pavement
PCC Overlay Types	• JPCP • JRCP • CRCP	• JPCP • JRCP • CRCP	• JPCP • JRCP • CRCP

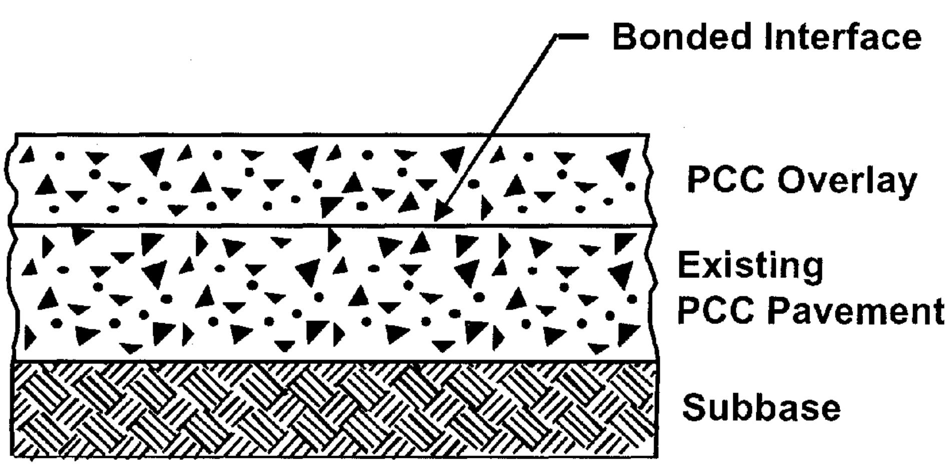

Figure 2-1. Bonded PCC overlay (McGhee 1994).

2.2.1.2. Partially Bonded PCC Overlays

Partially bonded PCC overlays are placed directly on the existing PCC pavement with little, if any, surface preparation. These are used where the question of whether and to what degree bonding takes place is not critical to the performance of the overlay (Lokken 1981). Partially bonded PCC overlays are more commonly used on airfield pavement where required overlay thicknesses are typically much greater (because of heavy aircraft loads) and where the slabs are more fully restrained. On airfield pavements, partially bonded overlay thicknesses are generally greater than 305 mm (12 in), whereas on highway applications thicknesses are generally in the range of 152 to 203 mm (6 to 8 in).

As previously mentioned, no special measures are taken to prepare the existing pavement to receive a partially bonded PCC overlay, other than just sweeping the surface. As a result, varying degrees of bonding will occur, so reflection cracking can potentially be a problem. Consequently, partially bonded overlays should be used only when the existing PCC pavement is in sound, well-seated condition and with no major distresses, distortions, or rocking slabs (Lokken 1981).

According to the comprehensive list of PCC overlay construction projects prepared by McGhee (1994), most partially bonded overlays are either JPCP or JRCP designs, although a few CRCP have been constructed. However, that same list also shows that partially bonded PCC overlays are not widely used for highway applications, so their use is not discussed further in this report.

2.2.1.3. Unbonded PCC Overlays

An unbonded PCC overlay (sometimes called a separated overlay) contains an interlayer between the existing PCC pavement and the new PCC overlay (see figure 2-2). This separation layer is placed to ensure independent behavior between the two slabs, thereby minimizing the potential for reflection cracking. Unbonded PCC overlays are typically constructed between about 152 and 305 mm (6 to 12 in) thick.

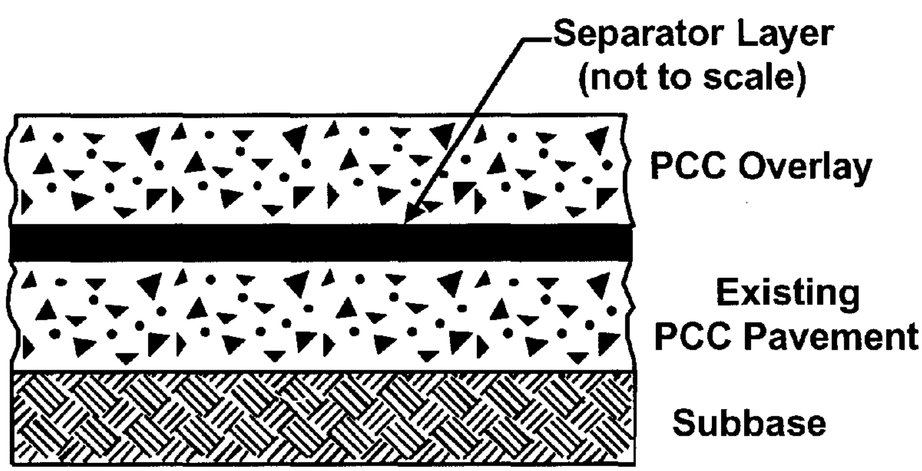

Figure 2-2. Unbonded PCC overlay (McGhee 1994).

Unbonded PCC overlays are used when the existing pavement deterioration is so advanced that it cannot be effectively corrected prior to overlaying (ACPA 1990b). Because the two pavements will be separated, little preoverlay repair is typically required. However, the separator layer must be effective at ensuring independent behavior between the two rigid layers. HMA layers (typically about 25 mm [1 in] thick) are commonly used as an interlayer.

In both highway and airfield applications, unbonded overlays have seen far greater use than either bonded or partially bonded overlays. Most unbonded overlays have been JPCP designs, although a significant number of unbonded CRCP designs have also been constructed. Current pavement practice is away from JRCP designs, and these are rarely constructed any more.

2.2.2 *PCC Overlays of Existing HMA Pavements*

The use of PCC overlays of existing HMA pavements, sometimes referred to as "whitetopping," has increased considerably in the last decade. These overlay types are generally classified as either conventional whitetopping or ultra-thin whitetopping (UTW), according to the thickness of the PCC overlay. Table 2-2 summarizes some of the key characteristics of these PCC overlay types (Grogg et al. 2001). Additional information is provided in the following sections.

Table 2-2. Summary of whitetopping overlays (Grogg et al. 2001).

	Conventional Whitetopping	**Ultra-Thin Whitetopping**
Typical Thicknesses	• 102 to 305 mm (4 to 12 in)	• 50 to 102 mm (2 to 4 in)
Condition of Existing Pavement	• All deteriorated HMA pavements	• Low-volume deteriorated HMA pavements (particularly in areas where rutting is a problem)
Bonding Condition	• Designed as unbonded, but some partial bonding occurs (and may enhance pavement performance)	• Strong bond required between existing HMA pavement and new PCC overlay
Preoverlay Repair	• Limited repair (failed areas only) • Possible milling to correct profile	• Repair of areas unable to contribute to load-carrying capacity • Milling of the HMA surface
Minimum Thickness of HMA	• 50 mm (2 in) (after any milling)	• 76 to 152 mm (3 to 6 in) (after any milling)
Special Design and Construction Considerations	• Adequate support critical to performance • Adequate joint design (including joint spacing and load transfer) • Placement of a whitewash on HMA surface on hot days	• Bonding with HMA pavement • PCC mix design is often high-strength and/or fiber modified • Extremely short joint spacings (typically between 0.6 and 1.8 m [2 to 6 ft])
PCC Overlay Types	• JPCP • JRCP • CRCP	• JPCP

2.2.2.1. Conventional Whitetopping

Conventional whitetopping is the placement of a PCC overlay on an existing HMA pavement. These are generally designed as new PCC pavement structures and can range from 102 to 305 mm (4 to 12 in) thick. A typical cross section of a whitetopping PCC pavement is shown in figure 2-3 (McGhee 1994). The interface shown in that figure may be either a milled surface, an HMA leveling course, or no treatment at all (direct placement).

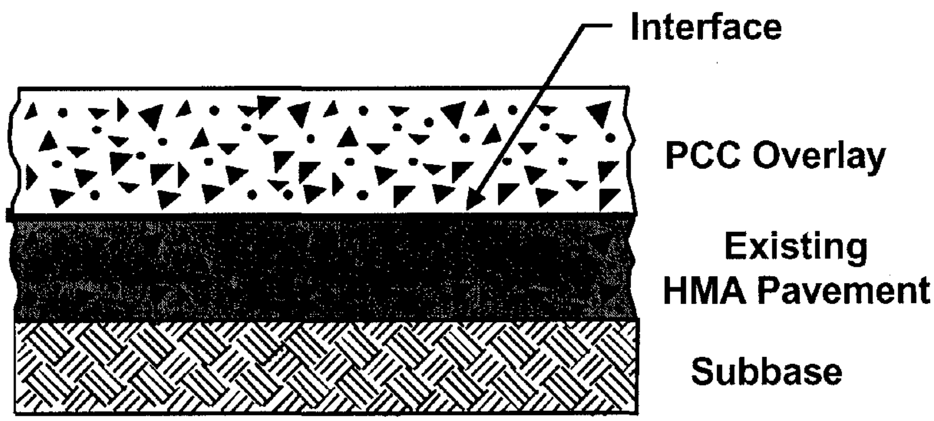

Figure 2-3. PCC overlay of HMA pavement (McGhee 1994).

Conventional whitetopping is an alternative solution to rehabilitating deteriorated flexible pavements that exhibit such distresses as rutting, shoving, and alligator cracking (ACPA 1998). Preoverlay repair of badly distressed or failed areas is required, and many agencies cold mill the existing HMA surface to remove ruts or surface irregularities prior to placement of the PCC overlay (McGhee 1994).

A conventional whitetopping overlay is designed essentially as a new PCC pavement on a treated base course, assuming an unbonded condition between the layers. However, some partial bonding between the PCC overlay and existing HMA pavement can occur, which can contribute to the performance of the pavement. Conventional whitetopping overlays have been constructed with JPCP, JRCP, and CRCP designs, with JPCP designs most commonly used. JRCP designs are rarely used in present-day PCC paving.

2.2.2.2. UTW

UTW is a process in which a thin layer of PCC (between 50 to 102 mm [2 to 4 in] thick) is placed over a distressed HMA pavement (ACPA 1998). In a UTW project, the existing HMA surface is cold milled in order to enhance the bond between the PCC overlay and the existing HMA pavement to create a monolithic structure. UTW is intended for parking lots, residential streets, low volume roads, general aviation airports, and HMA intersections where rutting is a problem but no other significant structural deterioration is present (ACPA 1998).

UTW overlays employ short slabs, typically square and between 0.6 and 1.8 m (2 and 6 ft). This is to help reduce bending and thermal curling stresses. Figure 2-4 shows a schematic of a UTW overlay (Grogg et al. 2001).

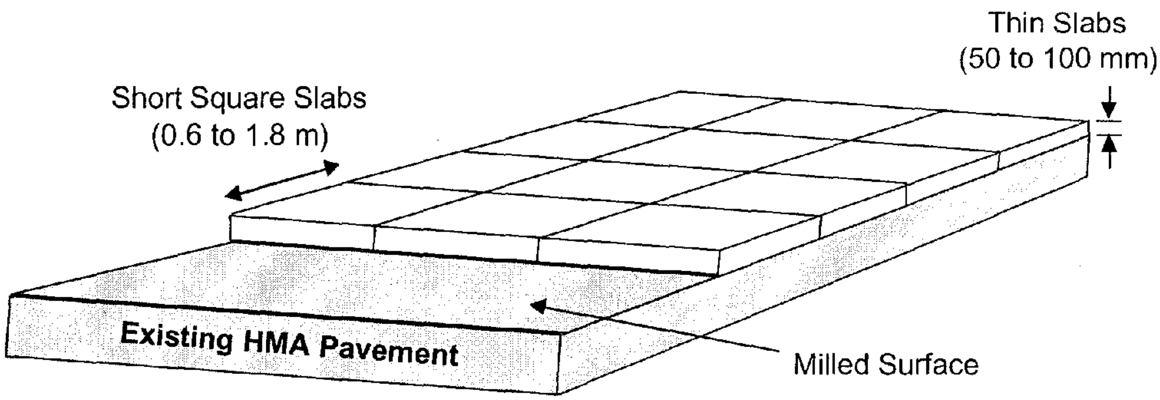

Figure 2-4. Schematic of UTW overlay (Grogg et al. 2001).

The use of UTW has grown rapidly in the last decade, with over 200 projects built in thirty-five states since 1992 (ACPA 2000a). The State of Tennessee has constructed the most UTW projects, followed closely by Kentucky and Kansas. All UTW projects have been JPCP designs, with some incorporating fiber reinforcement in the PCC mix.

A member of the whitetopping family that is closely related to ultra-thin whitetopping is *thin* whitetopping. These are 102- and 203-mm-(4- and 8-in)-thick PCC slabs with shorter joint spacing (typically between 1.8 and 3.6 m [6 and 12 ft]) that are placed on a milled HMA. As with the UTW design, the milling of the HMA surface is intended to promote the bonding between the PCC overlay and the existing HMA pavement. This bonding is accounted for in the design process, and has a positive effect on the performance of the PCC overlay. Thin whitetopping overlays have been used by a few highway agencies, primarily on state highways.

A variation of thin whitetopping overlays is an inlay, in which one or more of the travel lanes of an existing pavement are milled and the PCC overlay placed in that milled area. This technique has the advantages of targeting only the distressed lanes, maintaining existing pavement elevations, and eliminating the need for any unnecessary shoulder work or repair.

2.2.2.3. Summary of Whitetopping Overlay Characteristics

The following design and construction characteristics can be used to distinguish between the various types of whitetopping overlays:

- Conventional whitetopping is designed essentially as a new pavement on a stabilized base, and assumes an unbonded condition between the PCC overlay and the HMA pavement, even if the HMA pavement is milled. Conventional whitetopping can be used on any type of pavement facility.

- Thin whitetopping is a moderately thin PCC overlay (thicknesses between 102 and 203 mm [4 and 8 in]) that is placed on a milled HMA pavement. The bond between the PCC overlay and HMA pavement is relied upon in the design procedure, and short joint spacing (between 1.8 and 3.7 m [6 and 12 ft]) is used. Thin whitetopping overlays have been used most often on state highways and secondary routes.

- UTW is similar in concept to thin whitetopping in that the PCC is bonded to a milled HMA, and that bonding is relied upon in the design process. PCC overlay thicknesses are between 50 and 102 mm (2 and 4 in) and square slabs (between 0.6 and 1.8 m [2 and 6 ft] on a side) are employed. These overlays also commonly use fibers in the PCC mix design. UTW overlays are commonly used on urban streets and intersections.

2.2.3 *Other PCC Overlay Types*

A review of the available literature reveals several other types of PCC overlays that have been used. Primarily, these include fiber-reinforced PCC overlays and prestressed PCC overlays. However, these represent changes to the PCC mix design or pavement design, and do not alter the way of classifying PCC overlays into the existing categories already defined.

Prestressed PCC overlays have seen limited use in the U.S., and their use is not discussed further in this report. Fiber-reinforced PCC overlays have been used since about the 1950s, and have seen a recent increase in use as part of many UTW projects. Some additional information on the use of fiber in PCC overlays is presented later in this chapter.

2.3 Pavement Surfacing Materials

This section describes pavement surface materials used in PCC overlays. Conventional portland cement concrete is by far the most commonly used paving surface material, although fiber-reinforced or polymer-modified portland cement concretes are sometimes used.

2.3.1 *Portland Cement Concrete*

Conventional PCC paving mixes are typically used in the construction of PCC overlays. As with conventional PCC pavements, an effective mix design is essential to the performance of a PCC overlay. Each of the components used in a PCC mix must be carefully selected so that the resulting mixture is dense, relatively impermeable, and resistant to both environmental effects and deleterious chemical reactions over the length of its service life (Van Dam et al. 2002). Additional information on PCC mix design can be found in references by the American Concrete Institute (ACI 1991) and the Portland Cement Association (PCA 1992).

As with conventional PCC pavements, Type I and Type II cements are commonly used in PCC mixtures for PCC overlays. In situations where high-early strength is desired, some agencies use a Type III cement, which is more finely ground to promote the development of high early strength. Depending on the mix design and strength requirements, cement contents are typically in the range of 300 to 415 kg/m^3 (500 to 700 lb/yd^3), although higher contents are sometimes used.

Aggregates used in PCC mixtures range from crushed stones to river gravels and glacial deposits (McGhee 1994). To help ensure the longevity of the pavement, these aggregate should not only possess adequate strength, but should also be physically and chemically stable within the PCC mixture (Van Dam et al. 2002). Agencies generally require that aggregates conform to ASTM C 33, but this alone does not necessarily ensure durability (Van Dam et al. 2002). Consequently, extensive laboratory testing or demonstrated field performance is often required to ensure the selection of a durable aggregate.

The maximum coarse aggregate size used in PCC mixtures is a function of the pavement thickness or the amount of reinforcing steel (if used) (ACI 1991; PCA 1992). It is recommended that the largest practical maximum coarse aggregate size be used to minimize paste requirements, reduce shrinkage, minimize costs, and improve mechanical interlock properties at joints and cracks (Van Dam et al. 2002). Although maximum coarse aggregate sizes of 19 to 25 mm (0.75 to 1 in) have been common in the last two decades, some agencies are examining the use of larger maximum coarse aggregate sizes (38 to 50 mm [1.5 to 2 in]) for conventional PCC paving. However, for thinner PCC overlays (such as bonded PCC or UTW), smaller maximum coarse aggregate sizes are required. For unreinforced pavement structures, the PCA recommends a maximum aggregate size of one-third of the slab thickness (PCA 1992).

Guidance on the selection of the appropriate water-to-cementitious material ratio (*w/c*) is provided in ACI (1991) and PCA (1992). A maximum *w/c* value of 0.45 is common for pavements in a moist environment and subjected to freeze thaw cycles (PCA 1992). However, lower *w/c* values are used on thinner PCC overlays (bonded PCC overlays and UTW) to minimize drying shrinkage (McGhee 1994; ACPA 1998).

Various admixtures and additives are commonly introduced into PCC mixtures. These include the following:

- Air entrainment, intended to protect the hardened PCC from freeze-thaw damage and deicer scaling. However, air entrainment also helps increase the workability of fresh PCC, significantly reducing segregation and bleeding (PCA 1992). Typical entrained air contents of PCC pavement are in the range of 4 to 6 percent.

- Set accelerators, which are intended to increase the rate of PCC strength development. In the pavement field, they are commonly used in full-depth repairs or on "fast-track" paving projects in which early opening times are required. Calcium chloride is commonly used as a set accelerator.

- Water reducers, which are added to PCC mixtures in order to reduce the amount of water required to produce PCC of a given consistency. This allows for a lowering of the *w/c* while maintaining a desired slump, and thus has the beneficial effect of increasing strength and reducing permeability (Van Dam et al. 2002).

- Pozzolanic materials, such as fly ash, ground granulated blast furnace slag, and silica fume, may also be added to PCC mixtures. These materials may be placed in addition to the portland cement, or as a partial substitution for a percentage of the portland cement. Of these, fly ash is the most commonly used. Fly ash is a by-product of coal-fired power plants, and may be classified as either Class C (high calcium fly ash) or Class F (low calcium fly ash). Fly ash helps to improve the workability of the mix and also increase its durability; it also can increase the long-term strength of the PCC, although the short-term strength may be less. In addition, Class F fly ash is effective in reducing alkali-silica reactivity.

Most agencies specify a minimum PCC strength requirement for their pavements. Typical values include a 28-day compressive strength of 28 MPa (4000 lbf/in^2) or a 28-day, third-point flexural strength of 4.5 MPa (650 lbf/in^2). Fast-track paving mixtures, using a low *w/c* (typically less than 0.43), a higher cement content (typically greater than 385 kg/m^3 (650 lb/yd^3]), and perhaps a Type III cement (or a set accelerator), have been used by highway agencies to meet opening times of as little as 4 to 8 hours (Hoerner et al. 2001). However, fast-track paving technology entails not only the development of fast-setting PCC mixtures, but also the planning and coordination of all construction activities needed to minimize down time (ACPA 1994a; FHWA 1994; ACI 2001).

Although the use of fast-track mixes and paving practices has become more common, there has been some concern regarding the potential detrimental effect of quicker setting cements and faster construction times on the long-term durability of PCC mixtures (Van Dam et al. 2002). Thus, it may be that both the speed of construction and the long-term PCC durability need to be considered during the mix design phase of a project, with emphasis given to the use of the least "exotic" material that will still provide the desired opening times for a specific project.

2.3.2 *Fiber-Reinforced Concrete*

Fiber reinforced concrete (FRC) is portland cement concrete containing randomly distributed fibers throughout the PCC mixture. The principal reason for incorporating fibers is to increase the "toughness" of the PCC (which is a measure of its energy-absorbing capacity), as well as to improve its cracking and deformation characteristics; in some cases, PCC flexural strength may also be increased (PCA 1991).

A wide variety of fiber materials have been used to reinforce PCC, with steel, polypropylene, and polyester fibers most commonly used in the U.S. (PCA 1991). More recently, polyolefin fibers have also been used on several paving projects. Characteristics of these fibers are described below:

- Steel fibers are primarily made of carbon steel, although stainless steel fibers are also manufactured (PCA 1991). Steel fibers may be either round or flat, and are typically between 6.4 to 76 mm (0.25 to 3 in) long, with an aspect ratio (ratio of the fiber length to the fiber diameter or equivalent diameter) typically between 20 and 100 (ACI 1997a). The aspect ratio is an important parameter influencing the bond between the PCC and the fiber, with longer fibers providing greater strength and toughness (PCA 1991). Steel fibers may also have certain geometric features to enhance bonding, such as crimped or hooked ends, or surface deformations and irregularities (ACI 1997a).

  Steel fibers are typically added to PCC mixtures at a rate of 0.5 to 2.0 percent by volume, or between about 15 to 60 kg/m^3 (25 to 100 lb/yd^3) for conventional mixtures (PCA 1991; AASHTO 2000). Perhaps the biggest advantage of steel fibers is their effect on increasing the flexural strength of the PCC, typically by 50 to 70 percent (ACI 1997a). Using any PCC pavement design procedure, this would allow a reduction in the required slab thickness. Steel fiber-reinforced PCC pavements also help control plastic and drying shrinkage cracking, and also exhibit excellent toughness and post-cracking behavior (ACI 1997a). However, there are some concerns regarding the corrosion of steel fibers, particularly on pavements exhibiting cracks greater than 0.1 mm (0.004 in) wide (ACI 1997a).

- Polypropylene fibers are produced as continuous cylindrical-shaped filaments that can be chopped to specified lengths (PCA 1991). They can be produced in monofilament, multifilament, or fibrillated form. A monofilament is a single strand of fiber, multifilament consists of several strands of fiber, and fibrillated means that the fiber is slit such that tiny "branches" are formed (PCA 1991; AASHTO 2000). The fibrillation process greatly enhances the bonding between the PCC and the polypropylene fibers. Polypropylene fibers are also hydrophobic, so they do not absorb water nor affect the mixing requirements of the PCC (PCA 1991).

  Polypropylene fibers have been introduced into PCC mixtures at volume percentages ranging from 0.1 to 10 percent, although rates of 0.1 to 2 percent (or about 0.6 to 1.8 kg/m^3 [1 to 3 lb/yd^3]) are more common (ACI 1997a; AASHTO 2000). At the lower dosage rates (which are typical for pavement applications), polypropylene fibers are effective in controlling plastic shrinkage cracking (PCA 1991). Although there is no consensus on the effect of polypropylene fibers on the PCC flexural or compressive strength, studies have shown increases in toughness and improvements in post-cracking behavior in PCC containing the fibers (ACI 1997a).

- Polyester fibers are available only in monofilament form and are typically between 19 and 50 mm (0.75 and 2 in) long (PCA 1991). They are commonly added at relatively low fiber contents (typically 0.1 percent by volume) and are used to control plastic shrinkage-induced cracking (ACI 1997a). Test data regarding the properties of polyester fiber PCC are limited, but in general it does not appear that the addition of polyester fibers at such low contents significantly changes the mechanical properties of hardened PCC (PCA 1991).

- Polyolefin fibers are a relatively new material seeing increased use in various pavement applications, including conventional PCC pavements (Ramakrishnan and Tolmare 1998; Wojakowski 1998), unbonded PCC overlays (MoDOT 2000), whitetopping overlays (Ramakrishnan 1997), and UTW (Armaghani and Tu 1997; Sprinkel et al. 1997). Polyolefin fibers are purported to significantly enhance the physical properties of PCC, including toughness, ductility, and resistance to shrinkage cracking (Ramakrishnan 1997). These are typically added at a rate of about 2 percent by volume.

In recent years, polypropylene fibers have been most commonly used for PCC overlays, with the use of polyolefin fibers steadily increasing. Steel fibers, on the other hand, have seen less use in PCC overlays in the last few decades, although they are used in some airfield pavement applications and by several European countries. Table 2-3 summarizes some recent PCC overlay projects incorporating fibers (AASHTO 2000).

In summary, fibers are most commonly used on thin PCC pavements (such as bonded overlays or UTW) where plastic shrinkage is a concern. Some additional benefits may be obtained in terms of increased toughness and improved post cracking behavior; steel fibers also provide the additional benefit of significantly increasing PCC flexural strength. More detailed information on the characteristics of these fibers, and on the design and use of fibers in PCC mixtures is available from PCA (1991), ACI (1997a), and AASHTO (2000).

Table 2-3. Summary of recent fiber usage in PCC overlays (AASHTO 2000).

Location	Year Built	Type of Overlay	Type of Fiber
I-10, Baton Rouge, LA	1990	Bonded	Steel (85 lb/yd^3)
Cherokee County, OK	1994	Bonded	Polypropylene (3 lb/yd^3)
Beltway 8, Houston, TX	1996	Bonded	Steel
Beltway 8, Houston, TX	1999	Bonded	Steel
Iola, KS	2000	Bonded	Polypropylene (3 lb/yd^3)
I-29, Atchison County, KS	1998	Unbonded	Polyolefin (25 lb/yd^3)
Various projects in Tennessee (Chattanooga, Nashville, Memphis, Knoxville, and others)	1992-present	UTW	Polypropylene (3 lb/yd^3)
Leawood, KS	1995	UTW	Polypropylene (3 lb/yd^3)
US Highway 14, Pierre, SD	1996	UTW	Polyolefin (25 lb/yd^3)
Various street intersections in Springfield, MA	1999	UTW	Polypropylene (3 lb/yd^3)

2.3.3 *Polymer-Modified PCC*

Polymer-modified PCC has been primarily used on bridge decks, but this thin bonded surfacing may also be appropriate for pavements (McGhee 1994). In polymer-modified PCC mixtures, a water soluble or emulsified polymer has been added to the PCC during the mixing process (Mindess and Young 1981; FHWA 1986). As the cement hydrates, the polymer material forms a continuous polymer matrix throughout the PCC material. Primary advantages of polymer-modified PCC are its resistance to chloride penetration, its excellent bonding capabilities, its resistance to freeze-thaw, and its ease of application (FHWA 1986). A primary disadvantage to polymer-modified PCC is its high construction costs.

2.3.4 *Polymer Concrete*

Different from polymer-modified PCC but also used for bridge deck overlays is polymer concrete. Polymer concrete is a composite material formed by physically blending an aggregate with a polymeric binder; portland cement and water are not used (FHWA 1986). The polymer binds the aggregate together and imparts its own characteristics properties to the polymer concrete. Commonly used polymeric binders are epoxies, polyurethane, and methyl methacrylate.

Polymer concrete is resistant to chloride penetration and sets up very quickly, allowing it to be used when lane closure times are limited. Although it has been used extensive on bridge decks, it has seen little use in pavement applications. A recent ACI special report describes the performance of polymer concrete overlays used in bridge deck applications (ACI 1997b).

2.4 Interface Materials

Interface materials serve one of two purposes: they either enhance the bond between the PCC overlay and the existing pavement (to ensure monolithic behavior), or they separate the PCC overlay from the existing pavement (to ensure independent behavior) (McGhee 1994). Materials commonly used for these applications are described below.

2.4.1 *Bonding Agents*

For bonded PCC overlays of existing PCC pavements, achieving bond between the two layers is critical in order to obtain monolithic slab behavior. To help achieve this, many agencies place either a cement grout or an epoxy resin on the existing PCC pavement just ahead of the paver. Cement grouts are generally produced in a mobile mixer from a mixture of portland cement and water; the grout should have a maximum *w/c* of 0.62 (ACPA 1990a). Epoxy bonding agents should be applied in accordance with the manufacturer's instructions. Prior to the placement of either type of bonding agent, the pavement surface should have already been prepared and should be dry (ACPA 1990a).

As discussed in chapter 3, some studies suggest that no bonding agent is necessary if aggressive surface preparation is conducted (Whitney et al. 1992; Wells, Stark, and Polyzois 1999). In addition, there is the possibility that the bonding agent could act as a bondbreaker if allowed to dry before the placement of the PCC overlay (ACPA 1990a; Delatte et al. 1998; Sprinkel 2000). Consequently, the use of a bonding agent should be carefully considered by each highway agency.

2.4.2 *Separator Layers*

The performance of unbonded PCC overlays of existing PCC pavements depends largely upon obtaining effective *separation* between the two pavements. Because unbonded PCC overlays are placed on PCC pavements in a more advanced state of deterioration, distresses in the underlying pavement can reflect through the new overlay and compromise its performance.

To minimize the effect of the distresses in the underlying pavement on the performance of the PCC overlay, a separator layer is placed so that the two pavements act independently of each other. Other functions of the separator layer include to provide a level-up layer for uniform overlay thickness construction, to provide sufficient bonding and friction so that joints can form in JPCP and JRCP overlays, and to provide sufficient bonding and friction so that the proper amount of cracks can form in a JRCP or CRCP overlay (ERES 1999a).

A wide variety of materials has been used as separator layers, including polyethylene sheeting, wax-based curing compounds, liquid asphalts, and HMA materials (McGhee 1994). The most successful interlayer, and the one most commonly used, is a thick (25 mm [1 in] or more) layer of HMA (ERES 1999a). The use of thin asphaltic interlayers, such as chip seals or slurry seals, have worked well in some cases, but it is generally recommended that they not be used because they do not provide sufficient leveling capabilities, they erode near the joints, and they do not effectively separate the two layers (ERES 1999a). Polyethylene sheeting and curing compounds are also not recommended because they do not prevent working cracks from reflecting through the new overlay (ERES 1999a).

The HMA separator layer is typically a dense-graded mixture meeting an agency's standard paving specification. A few agencies have experimented with the use of permeable HMA interlayers, but long-term performance data for these types of interlayers are not available (ERES 1999a).

If the temperature of the HMA interlayer is expected to exceed 43 °C (110 °F), whitewashing of the surface may be required. This will reduce the temperature of the interlayer prior to the placement of the PCC overlay, preventing bonding and shrinkage cracking (ACPA 1990b). Whitewashing may be accomplished using either a lime slurry mixture or a white pigmented curing compound (ACPA 1990b). However, this practice may cause debonding between the PCC overlay and the separator layer, which could adversely affect the performance of the unbonded overlay (ERES 1999a).

2.5 Incidental Materials

Other materials used in the construction of PCC overlays are essentially the same as used in conventional PCC pavement construction, as summarized below:

- Dowel bars are typically billet steel, grade 60 bars that conform to ASTM A615 or AASHTO M31. The dowel bar size, layout, and coatings should be selected for the specific project location and traffic levels.
- Tie bars are typically billet steel, grade 40 bars that meet ASTM A615 or AASHTO M31 specifications. Tie bars are deformed bars and should be at least a number 5 bar (16 mm [0.62 in]) spaced no more than 762 mm (30 in) apart.
- Reinforcement in PCC overlays may be either deformed bars or welded wire fabric (WWF). Deformed reinforcing bars should conform to ASTM A615 or AASHTO M31, and welded wire fabric should conform to ASTM A185 or AASHTO M55. The steel contents should be determined based on the design conditions.
- Joint sealant materials are either hot-poured rubberized materials conforming to ASTM D3405, AASHTO M301, or a governing state specification; silicone materials conforming to a governing state specification; or preformed compression seals conforming to ASTM D2628, AASHTO M220, or a governing state specification.
- Curing of the completed pavement may be accomplished using wet burlap, polyethylene, or liquid membrane forming curing compounds that adhere to ASTM C309 or AASHTO M148.

2.6 Summary

This chapter presents an overview of the different types of PCC overlays, including bonded PCC overlays, partially bonded PCC overlays, and unbonded PCC overlays on existing PCC pavements, and conventional whitetopping and ultra-thin whitetopping of existing HMA pavements. Typical overlay characteristics, general designs features, and typical applications are described for each overlay type, with more detailed information presented in later chapters.

Materials used in PCC overlay construction are also described in this chapter. This includes a summary of PCC and its use in PCC overlays, the characteristics and use of modified PCC mixtures (fiber modified and polymer modified), a summary of overlay interface materials, and a description of incidental materials used in PCC overlay construction.

CHAPTER 3. BONDED PCC OVERLAYS

3.1 Introduction

As described in chapter 2, a bonded PCC overlay (also referred to as a bonded concrete overlay [BCO] or a thin bonded overlay [TBOL]) is the placement of a comparatively thin PCC layer on an existing PCC pavement. These overlays are differentiated from unbonded PCC overlays by the following key factors:

- Thickness. Bonded PCC overlays are typically 50 and 102 mm (2 and 4 in) thick, while unbonded PCC overlays typically range from 152 to 305 mm (7 to 12 in) thick.

- Bonding condition. Some sort of surface preparation is required to promote the required bonding between the PCC overlay and the PCC pavement, whereas with unbonded overlays steps are usually taken to prevent bond.

- Existing pavement condition. The existing pavement must be comparatively free of distress (either from preoverlay repair or because it is not exhibiting significant distress), in contrast with unbonded overlays, which can be placed over deteriorated pavements.

- Joint spacings. Transverse joints in the bonded overlays must match those in the underlying pavement, while in unbonded overlays they should be mismatched.

A bonded overlay increases the overall structural capacity of the pavement, but that structural benefit only occurs when the overlay and the underlying PCC behave monolithically. Thus the development of an effective bond between the PCC overlay and the existing PCC pavement is critical to the performance of these overlays. When a bonded overlay is properly constructed and the application is appropriate, its expected advantages are that it lasts longer than conventional HMA overlays and that it provides a higher level of serviceability over its service life.

Bonded PCC overlays have been used as a pavement rehabilitation technique for almost 90 years (Hutchinson 1982; Delatte and Laird 1999). A number of highway agencies have substantial experience with bonded overlays, both in the United States and internationally; among some of the agencies in the United States that have constructed bonded PCC overlays are Texas, Iowa, Pennsylvania, Louisiana, Virginia, Illinois, and California. These experiences cover a wide range of overlay designs placed over a variety of pavement types and conditions. A recent report by Sprinkel (2000) documents the performance of bonded PCC overlays that were constructed under the 1991 Intermodal Surface Transportation Efficiency Act (ISTEA) legislation.

3.2 Design of Bonded PCC Overlays

3.2.1 *General Design Considerations*

As previously mentioned, bonded PCC overlays are an appropriate rehabilitation strategy when the structural capacity of a pavement needs to be increased. This need is identified by an anticipated increase in traffic rather than by signs of pavement deterioration, which are the more common triggers of pavement rehabilitation. However, even after a bonded overlay is determined to be feasible, it is important to evaluate the pavement to confirm that it is a good candidate for a bonded overlay. Recommended evaluation steps include the following:

- Evaluation of existing pavement condition. A thorough examination of pavement conditions must be made in order to determine whether a bonded overlay is a feasible rehabilitation treatment. The ACPA gives guidelines on the types of distresses, their critical severity level, and the recommended preoverlay repair procedure prior to constructing a bonded overlay (ACPA 1990a).

- Traffic evaluation. Bonded PCC overlays are a good tool for improving the load-carrying capacity of an existing PCC pavement. Because a properly designed and constructed bonded overlay can provide many years of performance, care should be taken to develop meaningful projections of future traffic as an input to the design process. A bonded overlay is an appropriate rehabilitation treatment for any traffic level.

- Surface preparation. The single most important factor that affects the performance of a bonded overlay is achieving and maintaining a strong bond between the underlying PCC pavement and the PCC overlay. This involves determining the most appropriate surface preparation methodology.

An important consideration in the use of bonded PCC overlays is whether this type of overlay is appropriate for a given situation. Bonded PCC overlays are most appropriate when there is a need to restore a suitable riding surface or when there is a need to increase the structural capacity of an existing PCC pavement. The latter need could develop from an originally underdesigned pavement, when an unanticipated increase in heavy truck traffic occurs, or when unexpected growth or development outpaces design projections. In such cases, it may be apparent that the pavement, as designed, is inadequate to carry the (increased) traffic loading and this becomes an ideal application for a bonded PCC overlay.

When considering a bonded PCC overlay, it is imperative that the existing structural capacity of the underlying pavement not already be compromised. Where structural-related distresses are present, such as pumping, faulting, mid-panel cracks, or corner breaks, the load-carrying capabilities of the underlying pavement are already compromised and a bonded PCC overlay is not an appropriate rehabilitation technique. Furthermore, the presence of D-cracking or other materials-related distresses (MRD) in the underlying PCC suggest conditions where the effectiveness of a bonded overlay will be limited.

Bonded PCC overlays are constructed within the same general range of thicknesses as HMA overlays. As such, the same construction considerations regarding overhead clearances, shoulder drop-offs, guardrails, and so on that apply to the use of HMA overlays also apply to the use of bonded PCC overlays.

Most bonded PCC overlays are JPCP designs, with transverse and longitudinal joints matching those in the underlying pavement. Some bonded JRCP overlays have been used on existing JPCP and JRCP, although presently JRCP designs are rarely used. A few states, such as Texas and Virginia, have constructed bonded overlays on existing CRCP. In the case of JRCP and CRCP overlays, embedded steel is not used in the bonded overlay itself, as the overlay simply serves to increase the thickness (and load-carrying capacity) of the existing PCC pavement.

3.2.2 *Pavement Evaluation*

As with the design of other types of overlays, an evaluation of the existing pavement provides important information used to determine whether a bonded overlay is the appropriate method of structural improvement. A comprehensive evaluation typically consists of a visual distress survey, deflection testing using the falling weight deflectometer (FWD), and coring. The visual distress survey is the first step in determining the suitability of the pavement for a bonded overlay. The primary purpose is to determine if structural deterioration is present to the extent that it will impair the performance of the overlay. Examples of distresses that indicate structural deterioration include:

- Deteriorated transverse cracking.
- Corner breaks.
- Pumping.
- Faulting.
- Punchouts (CRCP).

Where any of these are present, their severity and extent should be considered to determine whether a bonded overlay is appropriate. If not widespread, full-depth repairs prior to placement of the overlay will help to minimize their effect on future performance; where these distresses are widespread or too severe, a bonded overlay will not perform well.

If MRD such as D-cracking or reactive aggregate are present, an analysis of the cores from the existing PCC can help to identify the nature and extent of the problem. D-cracking, for example, is typically a "bottom up" distress, so that by the time it is visible on the surface much of the PCC's integrity is compromised. Pavements with MRD are not good candidates for bonded PCC overlays.

The existing pavement's structural capacity is a key consideration in project selection, and the best method to determine structural capacity is by deflection testing with a FWD. The information that can be obtained from FWD testing results includes the following:

- Backcalculated subgrade k-value and PCC modulus.
- Subgrade variability.
- Load transfer efficiency.
- Presence of voids under joints and cracks.

The backcalculation can be accomplished using the procedure provided in the AASHTO guide (AASHTO 1993), the AASHTO Supplemental Design Procedure (AASHTO 1998), or any other established procedures. More detailed information on PCC pavement backcalculation is provided by Hall (1992). Recommended void detection procedures are described by Crovetti and Darter (1985).

3.2.3 *Thickness Design*

The typical thickness of a bonded PCC overlay is between 50 and 102 mm (2 and 4 in). This range is defined at the low end by the minimum thickness that can be placed by slipform paving equipment. At the high end, economic considerations and practical concerns (such as clearances and matching grades) control the maximum overlay thickness.

3.2.3.1. AASHTO Overlay Design Procedure

One widely used bonded overlay design procedure is that contained in the AASHTO *Guide for Design of Pavement Structures* (1993). This methodology is based on the structural deficiency approach, in which the required structural capacity of a new overlay (SC_{OL}) is equal to the difference between the structural capacity of a new pavement (SC_f) needed to carry the projected (future) traffic and the "effective" structural capacity of the existing pavement (SC_{eff}). This concept is illustrated in figure 3-1.

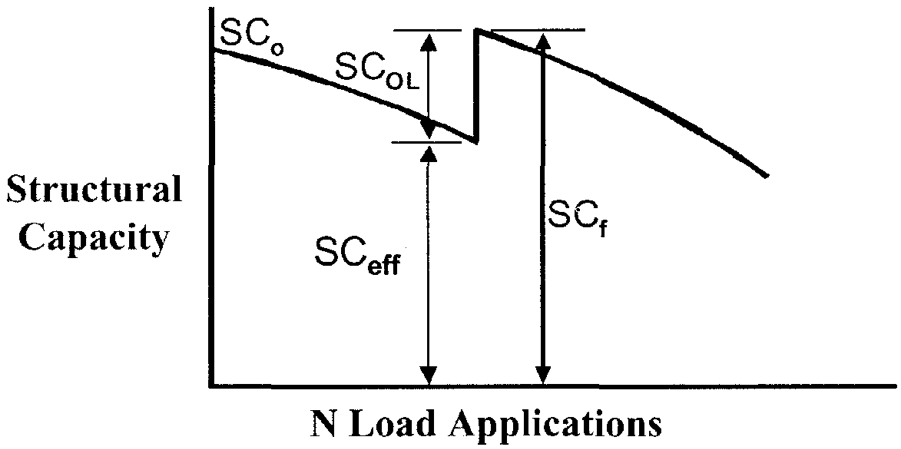

Figure 3-1. Structural deficiency approach used in AASHTO overlay design (AASHTO 1993).

The structural capacity for PCC overlays is represented by the slab thickness. For bonded PCC overlays, the required overlay thickness may be computed as follows:

$$D_{OL} = D_f - D_{eff} \tag{3-1}$$

where:

D_{OL} = Thickness of the bonded PCC overlay, mm (in)
D_f = Required thickness of a new PCC pavement for the future traffic loadings, mm (in)
D_{eff} = Effective thickness of the existing slab, mm (in)

The thickness required to carry future loadings, D_f, can be determined using the AASHTO design procedure for new PCC pavements, or any new PCC thickness design procedure. In the AASHTO procedure, the effective thickness of the existing slab, D_{eff}, can be calculated from one of two methods:

- Condition survey method. In this method, the actual thickness of the existing PCC pavement is reduced based on observed distress conditions; the more distress, the less the effective thickness. The effective thickness is computed as:

$$D_{eff} = D \times F_{jc} \times F_{dur} \times F_{fat} \tag{3-2}$$

where:

D_{eff} = Effective thickness of the existing slab, mm (in)
D = Actual thickness of the existing slab, mm (in)
F_{jc} = Joint condition adjustment factor
F_{dur} = Durability adjustment factor
F_{fat} = Fatigue damage adjustment factor

There are several charts and tables provided in the 1993 AASHTO *Guide for Design of Pavement Structures* to assist in selecting the various adjustment factors.

- Remaining life method. In this method, the amount of traffic that the existing PCC pavement has carried to date is compared to its design traffic, which provides an estimate of its remaining life. By knowing the remaining life, a chart in the AASHTO design guide can be used to determine a condition factor (CF), which is used to estimate the effective slab thickness as follows:

$$D_{eff} = CF \times D \tag{3-3}$$

where:

D_{eff} = Effective thickness of the existing slab, mm (in)
D = Actual thickness of the existing slab, mm (in)
CF = Condition factor estimated based on remaining life

Because bonded PCC overlays are meant to be placed over pavements in good condition, it is recommended that the CF should either be 1.0 or the pavement should be repaired so that it becomes 1.0.

3.2.3.2. Portland Cement Association Overlay Design Procedure

The Portland Cement Association (PCA) also has a design procedure for bonded PCC overlays based on a structural equivalency approach (Tayabji and Okamoto 1985). In this approach, the overlay thickness is determined such that critical stresses in the monolithic structure are equal to or less than the critical stresses in a new PCC pavement. Design charts for three categories of flexural strength of the existing PCC pavement are available, which produce minimum and maximum PCC overlay thicknesses of 50 mm (2 in) and 127 mm (5 in), respectively.

3.2.4 *Joint Design*

Since the performance of a bonded PCC overlay depends on creating a monolithic pavement structure, it is imperative that the joints in the overlay match the joints in the underlying pavement. Matched joints help to ensure that the two layers of the pavement structure are able to move together, helping to maintain bond between them. Matched joints also help to prevent reflection cracking. Because of the importance of matched joints, not only must the location of the joint be matched, but also the joint width and type (i.e., if there is an expansion joint in the underlying pavement it must be recreated in the overlay). Dowels or other load transfer devices are not used in conventional thin bonded PCC overlays.

3.2.5 *PCC Mix Design*

The importance of achieving and maintaining adequate bond between the overlay and the existing pavement is emphasized throughout this chapter. The mix design of the overlay PCC plays a role in this because it affects how the overlay will perform as it undergoes drying shrinkage and thermal movement. Delatte et al. (1998) recommend that the mix be designed for "rapid strength gain, minimum thermal expansion and contraction, and minimum shrinkage." Conventional PCC mixes have been used successfully and are acceptable for most applications. High early strength (HES) PCC mixes have also been used successfully, and should be considered where the pavement must be reopened to traffic on an accelerated schedule. In either case, a water reducer is commonly used to control the water-to-cementitious material (w/cm) ratio (ACPA 1990a). Components of the mix that should be carefully considered include:

- Water-cementitious material ratio. The higher the water content, the greater the potential for shrinkage as the water evaporates.

- Cementitious content. High cementitious contents, and especially high heats of hydration, affect the rate of strength gain. If the strength gain and high temperatures are not anticipated and controlled, they can be problematic.

- Aggregate properties. The coefficient of thermal expansion determines how the PCC will expand and contract when subjected to changes in temperature. These properties should be similar to those of the base PCC. Aggregate absorption affects shrinkage; an absorptive aggregate has a higher moisture demand and can contribute to debonding during curing.

Examples of actual mix designs evaluated by Delatte, Wade, and Fowler (2000) are provided in table 3-1 (it should be emphasized that these are not presented as examples of recommended mix designs). The standard is a mix meeting minimum TxDOT specifications for paving PCC. The State's specifications call for a compressive strength of 32 MPa (4640 lbf/in^2) at 28 days and a flexural strength of 4.4 MPa (640 lbf/in^2) at 7 days.

As described in chapter 2, fibers have also been used in bonded overlay mixes, but they are not a required constituent. Reported advantages include higher flexural strength, greater resistance to cracking, reduced shrinkage, and improved post-cracking behavior.

3.2.6 *Drainage*

As with any rehabilitation project, when constructing a bonded PCC overlay the evaluation and design phases provide an opportunity to evaluate existing pavement drainage characteristics and to upgrade them if necessary. Pavement performance indicators that signal a need to improve drainage capabilities include pumping, faulting, and corner breaks. The pavement cross slope should also be evaluated; current recommendations suggest a minimum cross-slope of 1.5 percent and this should be carried through to the bonded PCC overlay.

If it is determined that the existing pavement's drainage characteristics are deficient, the feasibility of retrofitting edge drains should be considered. Several NHI training courses (ERES 1999b; Hoerner et al. 2001) provide detailed information on the evaluation and design of drainage facilities in pavement rehabilitation.

Table 3-1. Example of mix design proportions for bonded PCC overlay materials tested in Texas (from Delatte, Wade, and Fowler 2000).

Mixture Components and Properties	Standard TxDOT Mix	High Early Strength Mix
Constituents, in kg/m^3		
Water	151	151
Type I/II cement	379	520
Coarse aggregate	1,105	1061
Fine aggregate	673	652
Admixtures, ml/100 kg cement		
Air-entraining admixture	131	78.5
High range water reducer (superplasticizer)		262
Mix Characteristics		
w/c ratio	0.40	0.29
Slump, mm	25 to 75	25 to 75
Air content	6%	4.5%
Estimated material cost, \$/m^3	79	94
PCC Strength		
Compressive strength		
7 days	30 MPa	39 MPa
28 days	39 MPa	41 MPa
Flexural strength		
7 days	4.1 MPa	Note: samples damaged, but results likely to have been acceptable
28 days	4.9 MPa	

3.3 Construction of Bonded PCC Overlays

3.3.1 *Preoverlay Repair*

Although a bonded overlay is most appropriate for pavements in good structural condition, some degree of preoverlay repair is still required. An important consideration is whether movement in the underlying pavement, either due to environmental conditions or to applied loads, will cause movement in the overlay. Any movement in the overlay that does not occur at matched joints (or cracks) will contribute to debonding and the subsequent deterioration of the overlay.

Appropriate preoverlay repairs include the following:

- Full-depth repair of medium- and high-severity transverse and longitudinal cracking, corner breaks, and punchouts (CRCP).
- Partial-depth repair of joint spalling.
- Slab stabilization to fill voids and prevent future pumping and loss of support.
- Load transfer restoration across working cracks or nondoweled joints.

Hoerner et al. (2001) provide more detailed information on these repair activities. Note that if any deterioration, such as joint spalling, is due to a materials-related distress, a bonded overlay is probably not an appropriate rehabilitation strategy. A life-cycle cost analysis is recommended to evaluate whether the cost of extensive repairs, in combination with the cost of the bonded PCC overlay, suggest the need to consider alternatives such as unbonded PCC overlays.

3.3.2 *Surface Preparation*

After proper project selection, surface preparation probably has the greatest impact on the long term performance of bonded PCC overlays. The purpose of surface preparation is to ready the existing PCC pavement for placement of either the bonding agent or the direct placement of the overlay. The objective is to remove contaminants, loose PCC, paint and other materials that could adversely affect the bonding of the overlay, and to provide a coarse macrotexture that promotes the mechanical bond between the old and the new pavement.

Over the years, many different surface preparation procedures have been used, including sandblasting, shotblasting, waterblasting, scabbling, milling, and acid etching. Today, the most commonly used (and most effective) surface preparation procedure is shotblasting, which is often followed by sandblasting and airblasting immediately prior to placement of the overlay. Alternatively, waterblasting is also reported to provide a suitable surface, but the surface must be allowed to dry before the placement of the PCC overlay (Delatte and Laird 1999).

The surface preparation technique should not be so aggressive that it damages the underlying PCC. Warner et al. (1998) note that commonly used preparation techniques such as shotblasting and waterblasting cause little "bruising" of the existing surface, while more aggressive preparation techniques (scabblers and drum-type carbide pick mills, for example) strike the pavement with great force and can create a weak layer in the existing PCC, just below the bond interface.

The ability of whatever surface preparation method is used to create the desired surface texture can be assessed by physical measurement. The sand patch test (ASTM E965) is one widely used surface texture measurement procedure, although there are also recently developed texture measurement devices based on lasers that may prove to be quicker and more accurate. The Texas Department of Transportation requires a 2.0 mm (0.08 in) minimum average texture depth (measured by the Sand Patch Method) following surface preparation (Delatte et al. 1998).

Another surface preparation issue involves whether or not to pre-wet the surface. Some believe the surface should be completely dry, whereas others believe that pre-wetting of the existing PCC (to a saturated surface dry condition) can be extremely beneficial because it can help to control the moisture demand from the bonded PCC overlay. If too much moisture is absorbed by the existing PCC from the plastic PCC, shrinkage will occur and debonding is likely to follow. Regardless of whether the surface is prewetted, Wells, Stark, and Polyzois (1999) recommend that some degree of surface preparation is required in order to achieve bond.

Another debate in the placement of bonded PCC overlays involves whether to use a grout or epoxy bonding agent layer between the substrate PCC and the PCC overlay. While these materials can be used to promote bond, they also have the potential to act as debonding layers if their placement is not carefully controlled. The consensus is that with the proper surface preparation technique, a bonding agent is not required (ACPA 1990a; Whitney et al. 1992; Wells, Stark, and Polyzois 1999; Sprinkel 2000). In fact, in a study of bonded PCC overlays constructed under the 1991 ISTEA legislation, Sprinkel (2000) concludes that shotblasting followed by a saturated surface dry (SSD) surface and no grout provide very good bond strength. In addition, a study by Wells, Stark, and Polyzois (1999) showed that without sufficient surface preparation, the use of a bonding agent alone was not enough to promote sufficient bond; it was also demonstrated that bonding agents with higher strengths than conventional cement slurry were not likely to lead to higher bonding strengths.

If used, a typical mix for a bonding agent consists of 783 kg (1726 lb) of cement to 387 kg (853 lb) of water, with a maximum water-cement ratio of 0.62 (ACPA 1990a). During placement, it is important that the bonding agent be placed immediately in front of the paver with a separation of no more than 3 m (10 ft) to prevent it from drying before the bonded PCC overlay is placed (ACPA 1990a).

The bonding of the PCC pavement overlay is also greatly affected by the prevailing climatic conditions at the time of construction, such as ambient temperature, humidity, and wind speed (McCullough and Rasmussen 1999a). If significant stresses develop during the first 72 hours following PCC placement, debonding of the overlay from the underlying pavement may occur. A computer program, HIPERBOND, part of the HIPERPAV software, has been developed that predicts the development of interface bond stresses and strengths to assess the possibility of early-age failures (cracking or delamination) of the PCC overlay (McCullough and Rasmussen 1999a; McCullough and Rasmussen 1999b).

As with many of the construction steps, the ultimate objective of surface preparation is to achieve the desired bond strength between the two PCC layers of the pavement structure. The literature teems with references to bond strength and the steps used to achieve it and measure it (see Warner et al. 1998, for example, for a summary of procedures to measure bond strength). Common guidance on the required bond has suggested a bond (shear) strength of 1.4 MPa (200 lbf/in^2) between the two PCC layers is sufficient (ACPA 1990a). However, Sprinkel and Ozyildirim (2000) have suggested the following guidelines for evaluating bond strength, based on a pull (tensile) test:

- 2.1 MPa (300 lbf/in^2): *Excellent.*
- 1.7 to 2.1 MPa (250 to 299 lbf/in^2): *Very Good.*
- 1.4 to 1.7 MPa (200 to 249 lbf/in^2): *Good.*
- 0.7 to 1.4 MPa (100 to 199 lbf/in^2): *Fair.*
- 0 to 0.7 MPa (0 to 99 lbf/in^2): *Poor.*

Wells, Stark, and Polyzois (1999) report on a Canadian Standard of 0.90 MPa (130 lbf/in^2).

3.3.3 *PCC Placement and Finishing*

The placement of a bonded PCC overlay is generally no different from that of conventional PCC. Specific recommendations for bonded overlay placement include (ACPA 1990a):

- Grade adjustments must be such as to leave the required thickness of the bonded PCC overlay.

- For vehicles operating on the existing PCC, care must be taken that they do not drip oil or other contaminants that will affect the bond.

The finishing of the bonded PCC overlay surface should follow the same practices used to finish any PCC pavement. Acceptable surface finishes include transverse tining, and dragging of astroturf, burlap, or other appropriate material.

3.3.4 *Texturing*

Texturing of the finished PCC pavement surface is required to ensure adequate surface friction of the roadway. Initial texturing is often done with a burlap drag or turf drag, with the final texturing provided by tining. Tining provides macrotexture, which contributes to surface friction by tire deformation, and also channels surface water out from between the pavement and the tire. Tining should be conducted as soon as the sheen goes off of the PCC. Additional guidance on surface tining is found in reports by Kuemmel et al. (2000) and by ACPA (2000c).

Tining has traditionally been conducted transversely and at uniform intervals, but recent studies suggest that uniformly spaced transverse tining produces irritating pavement noise (Larson and Hibbs 1997; Kuemmel et al. 2000). Consequently, some agencies are experimenting with transverse tining that is randomly spaced and skewed to the centerline of the pavement, the pattern of which must be carefully designed and constructed in order to minimize discrete noise frequencies that are most objectionable to the human ear (Kuemmel et al. 2000). In addition, some agencies are investigating the use of longitudinal tining, which produces lower noise levels than either uniformly or randomly spaced transverse tining (Kuemmel et al. 2000). Current recommendations for tining are as follows:

- The depth of tining should be 3 to 5 mm (0.12 to 0.20 in), and the individual tines should be 3.0 mm (0.12 in) wide (Kuemmel et al. 2000; WisDOT 2001).

- When tining transversely, tines should be spaced randomly at a minimum spacing of 10 mm (0.4 in) and a maximum spacing of 57 mm (2.2 in) apart (WisDOT 2001). Either skewed or nonskewed transverse tining may be conducted, although skewed tining is quieter (WisDOT 2001). A recommended random tining pattern specifically developed to avoid repeating tine patterns over the typical passenger car's wheelbase is available in the Wisconsin DOT Construction and Materials Manual (WisDOT 2001).

- When tining longitudinally, the tining should be done parallel to the centerline of the pavement with tines uniformly spaced at 19 mm (0.75 in) intervals (ACPA 1999b; ACPA 2000c).

However, both AASHTO and FHWA recommend that friction and safety not be compromised to obtain slight, usually short-term, reductions in noise levels (Smith and Hall 2001).

3.3.5 *Curing*

Proper curing is extremely important to the long-term performance of a bonded PCC overlay. While factors that have an effect on the curing of a bonded overlay are generally no different than for conventional PCC, there are several characteristics of the bonded overlay that make curing particularly important. These include the presence of an existing PCC layer with its own thermal properties and the thinncss of the overlay. With conventional PCC, improper curing affects the rate of strength gain and can lead to shrinkage cracking and other surface defects; with bonded PCC overlays, improper curing affects the bond strength and can lead to overlay failure.

Challenges to proper curing occur at both the surface and the base of the overlay. If the temperature of the substrate PCC is either very hot or very cold, it will impact the curing of the overlay. Similarly, if there is low ambient humidity or the ambient temperatures drop substantially after placement of the bonded PCC overlay, situations are created which have an adverse effect on the curing of the pavement.

The application of a curing compound to the surface and exposed edges should be sufficient to control the rate of drying shrinkage in a bonded PCC overlay; ACPA (1990a) recommends a minimum application rate of 2.5 m^2/l (100 ft^2/gal), which is about twice the normal rate. Under harsh climatic conditions, the use of curing blankets, or even temporarily halting paving, may also be advisable. Delatte et al. (1996) provide recommendations for curing under the harsh environmental conditions of southwestern Texas. Under such conditions, it is advisable to use a portable weather station to monitor ambient temperatures, wind speed, and relative humidity (Delatte et al. 1998).

3.3.6 *Joint Construction*

Important factors in sawing joints in the bonded PCC overlays include placement, timing, and depth. As noted previously, the location of transverse and longitudinal joints in the bonded PCC overlay must coincide closely with the joints in the underlying pavement. Experience suggests that a deviation greater than 25 mm (1 in) will contribute to secondary cracking and spalling. Thus it is necessary to lay out the joints in the underlying pavement accurately and carefully. A common technique is to locate the existing transverse joints with guide nails driven off on either side of the pavement in the shoulder (and away from the trackline of the paver). After the PCC overlay is placed, a chalkline is used to connect those guide nails across the new PCC overlay and then "snapped" to establish the transverse joint locations. Longitudinal joints, when uniform and consistent, can be easily located by measuring the horizontal offset from the edge of the existing pavement.

The timing of joint sawing for bonded overlays follows similar guidelines for any PCC pavement in that it must occur as soon as the surface can support the joint sawing equipment and the pavement can be sawed without spalling or raveling the joint. This is especially important for bonded overlays because of their thinness and the fact that if they crack before joints are formed, debonding is likely to occur. Depending on the mix and the prevailing climatic conditions, joint sawing operations generally occur within 4 to 12 hours after paving.

Guidance on the depth of sawing is provided by the ACPA (1990a). Their recommendations are repeated in table 3-2.

Table 3-2. Recommended joint sawing depths for bonded PCC overlays (ACPA 1990a).

Joint Type	Overlay Thickness	
	< 102 mm (4 in)	102 mm (4 in)
Transverse Contraction	Nominal thickness + 12 mm (0.5 in)	nominal thickness
Longitudinal	½ nominal thickness	nominal thickness
Expansion	Nominal thickness + 12 mm (0.5 in)	Nominal thickness + 12 mm (0.5 in)

3.4 Performance of Bonded PCC Overlays

As noted above, bonded PCC overlays have been successfully used as a PCC rehabilitation technique for many years. However, as summarized in NCHRP synthesis documents (Hutchinson 1982; McGhee 1994), the performance of these projects has been mixed. Although many projects have provided good long-term performance, several have failed in the first few years after construction. These failures are often characterized by reflective cracking and corner breaks (in areas where bond has been lost). Most of these failures, however, were attributed to the application of the bonded overlay to a pavement that was too far deteriorated (McGhee 1994). Others are attributed to inadequate or ineffective preoverlay repair (Peshkin and Mueller 1990). The performance of those projects is summarized elsewhere (see Hutchinson 1982 or McGhee 1994 for reviews of earlier projects).

Nevertheless, where used in an appropriate application, bonded PCC overlays have provided good performance. In this section, recent reviews of the performance of several bonded overlay projects are provided.

3.4.1 *Illinois (I-80 and I-88, both east of Moline)*

Volle (2000; 2001) reports on the performance of two bonded PCC overlay projects in Illinois. Both were constructed over 203 mm (8-in) CRCP, and included experimental sections that evaluated the impacts of a number of design features, including thickness (76 and 102 mm [3 and 4 in]), mix design, grout materials, and surface preparation techniques.

The performance of these projects was mixed. The project on I-80, constructed in 1994 and 1995, did not perform well. In particular, the eastbound lanes required repair soon after construction, consisting of both full-depth and partial-depth patches. No possible reasons for this rapid deterioration are given, but it is noted that the traffic volumes on I-80 are rather high (Volle 2000; Volle 2001). The I-88 project, which was constructed in 1996, exhibited better performance, but was also in better condition prior to the construction of the overlay and is subjected to 40 percent of the traffic on I-80. Adequate bond strength was developed on both the sections with and without grout.

Several conclusions can be drawn from the performance of these two projects (Volle 2000; Volle 2001):

- Adequate bond strength can be developed without a bonding agent.

- Shotblasting provides sufficient surface preparation to promote good bond.

- The existing pavement condition should be carefully considered in determining project feasibility (there was a significant difference between the condition of the I-80 and I-88 pavement).

- Vehicle loadings (ESALs), in combination with underlying pavement condition, affect bonded PCC overlay performance.

3.4.2 *Virginia (U.S. 13, Northampton County)*

Virginia's first bonded PCC overlay was constructed in 1990 (Freeman 1996). The 89-mm (3.5-in) overlay was placed on a 203-mm (8-in) thick JPCP that was constructed in 1965. The existing pavement exhibited "joint faulting and, to a lesser extent, joint spalling." Some longitudinal cracking was also noted throughout the project.

Surface preparation included the removal and replacement of damaged PCC and joint resealing. The entire surface was prepared by shotblasting; half of the project included the placement of grout as a bonding agent and half was placed directly on the underlying pavement. This project was constructed as a fast-track overlay, in that the time from commencement of paving until opening to traffic was 58 hours. The PCC used in the overlay was a Virginia Department of Transportation (VDOT) Class A3 mix with the following characteristics:

- Minimum cement content: 444.5 kg/m^3 (750 lb/yd^3).
- Maximum w/c ratio: 0.42.
- Minimum compressive strength at 24 hours: 20.7 MPa (3,000 lbf/in^2).
- Coarse aggregate size: No. 57 or No. 68.

Performance surveys conducted over the following 6 years tracked the development of distresses in the PCC overlay. These distresses included corner breaks, transverse and longitudinal cracking, and joint spalling. Corner breaks and joint spalling increased throughout the monitoring period, while the cracking stayed relatively level. Freeman's conclusions included recommendations regarding taking special care to construct the transverse joints and to keep them sealed to reduce the extent of overlay bond failure and thereby delay the formation of corner breaks and cracking. Other conclusions and recommendations from this project are summarized below:

- A bonded overlay is an effective means for enhancing the structural capacity of jointed PCC pavements when those pavements are not seriously distressed.

- Thin bonded overlays constructed in a fast-track mode should be considered by VDOT to be a viable rehabilitation alternative for certain pavements.

- Chain drags are an effective method for identifying areas of delamination (where the bond interface is not too deep).

3.4.3 *Virginia (I-295 near Richmond and I-85 near Petersburg)*

Sprinkel and Ozyildirim (2000) describe three hydraulic cement concrete (HCC) overlays that were constructed in Virginia in 1995. Two of these were placed over CRCP in order to increase the cover over shallow reinforcement and to "enhance the structural integrity." The overlays included a range of material design variables and overlay thicknesses, and their performance was monitored to evaluate how well they bond to the underlying PCC, how they affect the overall pavement stiffness, how well they reduce chloride infiltration and corrosion, their effect on skid resistance, and their cost effectiveness.

Follow-up evaluations and testing showed that the primary distress in these overlays is transverse cracking, at a spacing similar to cracks in the underlying pavement. Testing with a falling weight deflectometer (FWD) showed that the composite stiffness of the pavement had increased considerably due to the placement of the overlay. From the condition surveys and extensive testing, the authors conclude that these high performance HCC overlays can be successfully placed on CRCP and are effective at decreasing the permeability of the pavement, improving the stiffness of the pavement, and providing satisfactory skid resistance (Sprinkel and Ozyildirim 2000). However, based on the research the authors conclude that the use of bonded PCC overlays is not economical when compared to HMA overlays, although their use may be justified under special circumstances (Sprinkel and Ozyildirim 2000).

3.4.4 *Texas (North Loop I-610, Houston [1985] and I-10 El Paso [1995])*

Many bonded overlays have been constructed in Texas and their performance is summarized both in individual construction reports and in the two NCHRP syntheses (Hutchinson 1982; McGhee 1994). Delatte et al. (1998) briefly summarize the performance of a number of bonded PCC overlays, including projects constructed in Houston in 1985 and in El Paso in 1996. Both of the original pavements in these projects were 203-mm (8-in) thick CRCP.

The Houston project included 10 sections, and tested different overlay reinforcements (welded wire and steel fiber) and overlay aggregates (limestone and siliceous river gravel). All overlay thicknesses were 102 mm (4 in). In the Houston project, debonding was correlated to higher evaporation rates and the use of latex as a bonding agent. Other factors adversely affecting bond included high substrate temperatures (over 52 °C [126 °F]) and ambient temperature drops greater than 14 °C (57 °F). The aggregate type was also believed to affect bonding, due to different coefficients of thermal expansion. It was reported that once bond was achieved during construction, it generally remained present during trafficking. This emphasizes the importance of following proper placement practices, but also suggests the many different components of a bonded PCC overlay that have an effect on performance.

The bonded PCC overlay constructed on I-10 in El Paso was 165 mm (6.5 in) thick and was placed on the inside lanes in both directions. Shortly after construction, delaminations started to occur. An in-depth study of the causes focused on environmental conditions at the time of placement, in conjunction with the moisture content of the mix and delays in applying the curing compound after placement. It was concluded that the bonding problems were "due mainly to inadequate paste adhesion caused by deficiencies in surface preparation, materials, and curing." Recommendations to avoid the same problems in the future focused on:

- Paying attention to the substrate surface condition (clean and saturated surface dry).
- Adjusting w/c to ambient conditions.
- Placing the curing compound as soon as possible.

3.4.5 *Oklahoma (I-40, Oklahoma City)*

In 1998, Oklahoma constructed a 300-m (1000-ft) experimental bonded PCC overlay in the westbound lanes of I-40 near Oklahoma City (Hubbard and Williams 1999). The transverse joints in the existing 229-mm (9-in) JPCP were spaced at 4.6-m (15-ft) intervals and did not contain dowel bars. The existing pavement exhibited moderate faulting and some cracked slabs. Prior to the placement of the bonded PCC overlay, cracked slabs were repaired full depth and dowel bars were retrofitted across the nondoweled transverse joints. A milling machine was used to cut the slots for the dowel bars in both lanes of the existing pavement, at distances of 0.6, 0.9, 1.2, 2.7, 3.0, and 3.4 m (2, 3, 4, 9, 10, and 11 ft) from the centerline (Hubbard and Williams 1999). Epoxy-coated dowel bars (diameter 32 mm [1.25 in]) fitted with expansion caps on one end were placed in the 102-mm (4-in) deep, 0.9-m (3-ft) long slots.

The bonded PCC overlay was placed directly on the exposed dowel bars, with laborers carefully placing PCC around the dowel bars just ahead of the paver (Hubbard and Williams 1999). The bonded PCC overlay contained a polypropylene fiber and was constructed 76-mm (3-in) thick. The transverse joints were sawed to match those of the underlying pavement and later sealed with a self-leveling silicone sealant (Hubbard and Williams 1999).

This project is intriguing in that it combines several innovative rehabilitation methods and materials into a single project: retrofitted dowel bars, bonded PCC overlay, and polypropylene fibers. Preliminary performance of this project is excellent, and Oklahoma will continue monitoring its performance for several more years.

3.4.6 *Other Studies*

There are at least two other studies that have the potential to further promote the general understanding of the construction and performance of bonded PCC overlays. The first is a series of projects that were constructed in conjunction with the 1991 ISTEA legislation, in which up to $15M was set aside to construct and evaluate thin surface laminates, including bonded PCC overlays, to explore the state of the technology (Sprinkel 2000). Ultimately, seven agencies constructed 23 test sites that looked at this technology. Some of Sprinkel's conclusions regarding the performance of bonded PCC overlays are summarized in the following points:

- The performance of PCC overlays on CRCP is very good, and the bond strengths between the two surfaces ranged from good to excellent.

- Considerable transverse reflection cracking occurred in PCC overlays of CRCP, regardless of the thickness of the overlay or the presence of fibers.

- Shotblasting followed by SSD surface and no grout provides very good bond strength for PCC overlays of PCC pavement.

The Long-Term Pavement Performance SPS-7 project is also a potential source of information about the design, construction, and performance of bonded PCC overlays. Under that project, four bonded overlay projects were constructed and their long-term performance is being monitored (Smith and Tayabji 1998).

3.5 Summary

Bonded PCC overlays are an appropriate rehabilitation technique for existing PCC pavements that are either in good condition or that can be restored to good condition. The key to their performance lies in achieving and maintaining a good bond between the overlay and the existing pavement. Factors that are often mentioned as having a significant impact on bond include surface preparation and the use of a bonding agent. However, there are many other factors that affect the bond; these include condition of the existing pavement, various design details (such as joint design), the overlay mix design, and the prevailing environmental conditions.

Many bonded PCC overlays have been constructed over the years. The reported experience has been varied, but in most cases poor performance is related to the failure to achieve long term bond between the two PCC layers. The various causes of these failures are discussed, with the objective of improving future projects.

CHAPTER 4. UNBONDED PCC OVERLAYS

4.1 Introduction

Unbonded PCC overlays are used most commonly to remedy structural deficiencies of existing PCC pavements. Compared to other types of overlays, the performance of unbonded overlays is relatively insensitive to the condition of the existing PCC pavement and therefore minimal preoverlay repairs are required. Consequently, candidate pavements for unbonded overlays are typically those with extensive deterioration, including those with material-related distresses (MRD) such as D-cracking or reactive aggregate. However, adequate consideration of the structural capacity of the existing pavement is still necessary for the structural design of unbonded overlays, and preoverlay repairs of certain types of distresses are still needed to avoid localized failures. Other critical factors that affect the performance of unbonded overlays include the separator layer design, the joint spacing layout, and the load transfer design.

An unbonded overlay must be adequately isolated from the underlying deterioration and allowed unrestricted horizontal movement; however, a certain amount of bonding or friction between the overlay and the separator layer and between the separator layer and the underlying pavement is also important to achieve good performance. Consequently, a more technically correct description of an unbonded overlay is a "separated overlay."

4.2 Design of Unbonded PCC Overlays

4.2.1 *General Design Considerations*

The design of unbonded overlays requires consideration of several factors. These are briefly described below.

- Existing pavement type and condition. The type of existing pavement and its overall condition go a long way in determining the engineering and economic feasibility of unbonded overlays. In general, unbonded overlays are feasible when the existing pavement is extensively deteriorated, and this includes pavements affected by severe MRD.

- Overlay pavement type. The selection of the overlay pavement type depends largely on agency preference. By far the most common type of unbonded overlay is JPCP, although a significant number of CRCP overlays have been constructed. Current pavement practice is away from JRCP designs, and these are rarely constructed any more. In general, the condition of the underlying pavement is more critical to the performance of unbonded CRCP overlays; CRCP overlays require more uniform support and an effective separator layer (e.g., 25-mm [1-in] HMA) to ensure good performance.

- Preoverlay repair. Unbonded overlays do not require extensive preoverlay repairs, but repair of certain types of distresses (e.g., shattered slabs in jointed concrete pavements [JCP] and punchouts and deteriorated cracks in CRCP) are important to avoid localized failures. Additional discussion of preoverlay repairs is presented in section 4.2.3.

- Separator layer design. The design of the separator layer is critical to the performance of unbonded overlays. The separator layer must adequately isolate the overlay from underlying deterioration and provide sufficient friction to ensure proper formation of joints in JCP and desirable crack spacing in CRCP. Additional information on separator layer design is presented in section 4.2.5.

There are various site factors can affect the feasibility of unbonded PCC overlays, including the following:

- Traffic control.
- Shoulders.
- Overhead clearance.

In urban areas, where traffic congestion is already a problem, management of detour traffic during construction can be a critical issue (TRB 1998). At some point, pavement reconstruction is unavoidable; however, while rehabilitation alternatives with less severe lane closure requirements are still viable (e.g., HMA overlay), the lane closure requirement can be a key factor that determines the feasibility of unbonded overlays. For projects in congested areas, the use of fast-track paving techniques with unbonded overlays may be appropriate to minimize lane closure time.

In addition, the construction of an unbonded overlay requires the construction of new shoulders because of the increase in the elevation of the mainline pavement. The elevation change also means that interchange ramps have to be adjusted and guardrails may have to be raised, both of which affect the economic feasibility of unbonded overlays.

Overhead clearances are another factor that affects the feasibility of unbonded overlays. Because unbonded overlays add significant thickness to a pavement's overall cross section, short sections of reconstruction may be required at overhead structures (such as bridge overpasses) to ensure that adequate vertical clearance is provided. Raising the structure is another, albeit more expensive, alternative. However, both of these options add complexity, time, and cost to pavement rehabilitation projects, which makes unbonded overlays less favorable on projects that include many overhead structures.

4.2.2 *Pavement Evaluation*

As with other types of overlays, a thorough evaluation of the existing pavement is also important in the unbonded overlay design process. Pavement evaluation techniques typically include a visual distress survey, deflection testing using a falling weight deflectometer (FWD), and coring. If MRD such as D-cracking or reactive aggregate problems are present, laboratory testing of the cores may also be needed to verify the nature of MRD and to assist in avoiding similar problems in the new PCC overlay.

The existing pavement condition is a key input to thickness design in all design procedures. In both the AASHTO (1993) and PCA (Tayabji and Okamoto 1985) overlay design procedures, the structural value of the existing pavement is determined based on visual distress survey results. The condition survey is also important for identifying the areas that should be repaired prior to overlaying and for identifying MRD or drainage problems that require special consideration.

The preferred method of characterizing the structural condition of the existing pavement for rehabilitation design is through FWD testing. Information that can be obtained from FWD testing includes:

- Backcalculated subgrade k-value and PCC modulus.
- Subgrade variability.
- Load transfer efficiency.
- Presence of voids under joints and cracks.

Backcalculation can be accomplished using the procedures provided in the AASHTO guide (AASHTO 1993) or the AASHTO Supplemental Design Procedure (AASHTO 1998), or using any other established procedures. More detailed information on PCC pavement backcalculation is provided by Hall (1992). Recommended void detection procedures are described by Crovetti and Darter (1985).

4.2.3 *Preoverlay Repair*

Unbonded PCC overlays generally require minimal preoverlay repairs. Only the distresses that cause a major loss of structural integrity require repair. Those distresses, and their recommended method of repair, are listed below (Hutchinson 1982; ACPA 1990b; ERES 1999a):

- Shattered slabs should require full-slab replacements.

- Punchouts in CRCP require full-depth repairs.

- High-severity transverse cracks with ruptured steel on CRCP should be full-depth repaired.

- Unstable slabs or pieces of slabs with large deflections or pumping should be full-depth repaired or undersealed.

- Faulting is generally not a problem when a thick separator layer (25 mm [1 in] or more of HMA) is used. However, diamond grinding or milling is recommended for faulting greater than 6 mm (0.25 in). Alternatively, a thicker separator layer (50 mm [2 in] HMA) may be used when faulting exceeds 6 mm (0.25 in).

- High-severity spalling at existing pavement joints or cracks should be filled and compacted with HMA.

- Settlements, if significant, should be leveled up with HMA.

If construction of an unbonded CRCP overlay is contemplated, more attention must be paid to preoverlay repair activities to ensure that the existing distresses do not reflect through the overlay (ERES 1999a). Depending on the condition of the existing pavement a thicker separator layer, a higher steel content, or a thicker overlay may be used to address concerns for reflection cracking.

As an alternative to preoverlay repairs, the existing pavement may be fractured to provide a more uniform support under the overlay. Some European countries have used this technique with excellent results (FHWA 1993). In Germany, the standard practice for unbonded overlay construction is to crack and seat the existing PCC pavement, place a 102-mm (4-in) lean-concrete separator layer, and place the PCC overlay (FHWA 1993). Notches are cut into the lean-concrete separator layer (matched to the joints in the overlay) to prevent random cracking. For pavements with severe MRD, slab fracturing may be particularly applicable because the continued progression of MRD in the original pavement can cause premature deterioration of thinner (178 mm [7 in]) unbonded overlays (ERES 1999a).

4.2.4 *Thickness Design*

The thickness of unbonded overlays on major highways has ranged from about 178 mm (7 in) to more than 254 mm (10 in). Thicknesses can be determined using several design methods, two of which are described in the following sections. However, it is important to recognize that thickness alone does not ensure adequate performance, and other key factors such as separator layer design and joint design must be adequately considered.

4.2.4.1 AASHTO Overlay Design Procedure

The most common procedure for the design of unbonded PCC overlays is the methodology used in the AASHTO *Guide for Design of Pavement Structures* (1993). This methodology is based on the structural deficiency approach, in which the required structural capacity of a new overlay (SC_{OL}) is equal to the difference between the structural capacity of a new pavement (SC_f) needed to carry the projected (future) traffic and the "effective" structural capacity of the existing pavement (SC_{eff}). This concept is illustrated in figure 3-1 of chapter 3.

As described in chapter 3, the structural capacity for PCC overlays is represented by the slab thickness. For unbonded PCC overlays, the required overlay thickness may be computed as follows:

$$D_{OL} = \sqrt{D_f^2 + D_{eff}^2} \qquad (4\text{-}1)$$

where:

D_{OL} = Thickness of the unbonded PCC overlay, mm (in)
D_f = Required thickness of a new PCC pavement for the future traffic loadings, mm (in)
D_{eff} = Effective thickness of the existing slab, mm (in)

The origin of the above "square root" equation is believed to date back to the analysis of the performance data at the Bates Road Test (Older 1924), and it remains the most widely used procedure for the structural design of unbonded overlays (Hall, Darter, and Seiler 1993).

The thickness required to carry future loadings, D_f, can be determined using the AASHTO design procedure for new PCC pavements, or any new PCC thickness design procedure. In the AASHTO procedure, the effective thickness of the existing slab, D_{eff}, can be calculated from one of two methods:

- Condition survey method. In this method, the actual thickness of the existing PCC pavement is reduced based on observed distress conditions; the more distress, the less the effective thickness. The effective thickness is computed as:

$$D_{eff} = D \times F_{jcu} \tag{4-2}$$

where:

D_{eff} = Effective thickness of the existing slab, mm (in)
D = Actual thickness of the existing slab, mm (in) (maximum of 254 mm [10 in])
F_{jcu} = Joint condition adjustment factor for unbonded overlays

Unlike the bonded overlay design equation, there are not any adjustment factors for durability or fatigue damage because of their minimal effect on the performance of the unbonded PCC overlay (AASHTO 1993). A chart for determining F_{jcu} is provided in the 1993 AASHTO *Guide for Design of Pavement Structures*.

- Remaining life method. In this method, the amount of traffic that the existing PCC pavement has carried to date is compared to its design traffic, which provides an estimate of its remaining life. By knowing the remaining life, a chart in the 1993 AASHTO design guide can be used to determine a condition factor (CF), which is used to estimate the effective slab thickness as follows:

$$D_{eff} = CF \times D \tag{4-3}$$

where:

D_{eff} = Effective thickness of the existing slab, mm (in)
D = Actual thickness of the existing slab, mm (in) (maximum of 254 mm [10 in])
CF = Condition factor estimated based on remaining life

The designer should recognize that the effective thickness determined using this method does not reflect the benefit of any preoverlay repair.

4.2.4.2 Portland Cement Association Overlay Design Procedure

The Portland Cement Association has a design procedure for unbonded overlays that uses a structural equivalence approach (Tayabji and Okamoto 1985). In this approach, the overlay thickness is selected to achieve equivalency in the structural response between the overlay and new full-depth pavement required for the design conditions; the J-SLAB finite element program was used to develop this procedure. Three different design charts are available to determine the required unbonded overlay thickness for existing pavements in good, fair and poor condition. The PCA design procedure is consistent with mechanistic design principles, and accurate results can be obtained if the stress equivalencies are established considering all stress components (i.e., both load and curling/warping stresses). However, curling and warping stresses are not considered in the procedure.

4.2.4.3 Discussion on Current Unbonded Overlay Design Procedures

Current unbonded overlay design procedures have significant limitations. Some of the major flaws in the existing thickness design procedures include the following:

- Lack of consideration of layer interaction. The structural contribution of the separator layer and the effects of friction or bonding between the overlay and the separator layer and between the separator layer and the underlying PCC pavement are ignored.

- Excessive credit given to existing pavement in some cases. Design procedures that are based on structural deficiency (e.g., AASHTO 1993) tend to produce unconservative results when the existing pavement is relatively thick.

- Lack of consideration of curling and warping stresses. The effects of curling and warping are even more critical for unbonded overlays than for new pavements because of the very stiff support provided by the underlying pavement. However, existing design procedures do not consider the effects of curling and warping. The main result of this deficiency is that the effects of joint spacing on critical stresses are not reflected in the thickness design. For unbonded JPCP overlays with long joint spacings (e.g., greater than 4.6 m [15 ft]), the lack of consideration of curling and warping stresses is a critical deficiency that often leads to unconservative overlay thicknesses.

These deficiencies offset each other to a certain extent for overlays that have a relatively thick (25 mm [1 in] or more) HMA separator layer, as long as the joint spacing is not excessive (greater than 4.6 m [15 ft]). For example, ignoring the structural contribution of the HMA separator layer and the layer interactions leads to conservative overlay thicknesses; however, excessive credit given to the existing pavement and lack of consideration of curling and warping stresses tend to produce unconservative overlay thicknesses. The net result is reduced overall design error due to the opposing effects of the errors from different sources. However, if the joint spacing is excessive, any beneficial effects of layer interaction can be overshadowed by excessive curling stresses.

A thin separator layer (such as a curing compound or polyethylene sheeting) does not promote layer interaction. For those designs, all errors in thickness design are additive and lead to less conservative designs.

Current practice in mechanistic analysis also has many limitations, including the inability to consider interlayer friction in all but the most sophisticated general purpose 3-D finite element analysis programs. The current development in ISLAB 2000 (a finite element program for PCC pavement analysis) to add a friction model may resolve the limitations in the available analysis tools, but further research is needed to determine the appropriate values of the friction parameter.

The best measure of the adequacy of the designs obtained using existing thickness design procedures is field performance. The field performance of unbonded overlays has been generally very good; however, inadequate overlay thickness can be blamed for premature failure of some unbonded overlay projects. The development of a robust mechanistic thickness design procedure would greatly improve design reliability and help optimize unbonded overlay designs.

4.2.5 *Separator Layer Design*

The separator layer design is one of the primary factors influencing the performance of unbonded PCC overlays. The separator layer performs the following important functions (Voigt, Carpenter, and Darter 1989; ERES 1999a):

- Isolate the overlay from underlying irregularities to allow uninhibited horizontal movement. Without adequate isolation, the underlying deterioration can cause localized locking of the overlay, which can result in reflection cracking.
- Provide adequate friction to ensure proper formation of joints in JCP and cracks in CRCP. The use of materials that offer minimal frictional resistance (e.g., lime slurry or polyethylene sheeting) can cause problems with joints not forming at the intended intervals. The friction between pavement layers also contributes to the composite action that is beneficial to overlay performance.
- Provide a level surface for the overlay construction.

The recognition that debonding of the pavement layers is not essential or desirable for proper functioning of unbonded overlays is an important recent development in unbonded overlay design. However, the modeling of the structural effects of the separator layer is a subject requiring further research.

Many different separator designs have been used in unbonded overlay construction. The design that has given the best results are thick (minimum 25-mm [1-in]) HMA layers (Voigt, Carpenter, and Darter 1989; ACPA 1990b; Hall, Darter, and Seiler 1993; ERES 1999a), which is also the design recommended in the 1993 AASHTO guide. In some applications, chip seals, slurry seals, and sand-asphalt mixtures have also worked well; polyethylene sheeting, roofing paper, and curing compounds, on the other hand, generally have not worked well (Voigt, Carpenter, and Darter 1989; ACPA 1990b). However, the use of the thin-layer materials is not recommended because they erode easily near joints, and they do not provide adequate isolation of the overlay PCC from underlying deterioration if the existing pavement has significant roughness from faulted joints and cracks (ACPA 1990b; ERES 1999a). A summary of general information for selecting separator layers is provided in table 4-1.

4.2.6 *Joint Spacing*

Joint spacing directly affects critical stresses in unbonded JPCP overlays. Depending on the pavement design, the climate, season, and time of the day, curling stresses in JPCP can equal or exceed the load stresses. Because of the very stiff support provided by the underlying pavement, curling stresses are more critical in unbonded overlays than in new PCC pavements constructed on a granular base. Mechanistically, joint spacing is an essential input to thickness design for JPCP overlays, and the design thickness is only valid for the joint spacing assumed in the design analysis. However, joint spacing is not directly considered in either the AASHTO or the PCA overlay design procedures and is another source of variability for the performance of unbonded overlays.

Table 4-1. General recommendations for selecting separator layers (ACPA 1990b; ERES 1999a)

General Pavement Condition	Repair Work Performed?	Minimum Recommended Interlayer*	Other Factors to Consider
ALL CONCRETE PAVEMENTS			
Badly Shattered Slabs	Yes—Replaced Full-Depth	Thin	Subgrade repair/drainage
High Deflection/Pumping	Yes—Replaced Full-Depth Yes—Seated	Thin Thick	Subgrade repair/drainage Drainage/Dowels in Overlay**
Unstable Slabs	Yes—Undersealed Yes—Seated	Thin Thick	Drainage/Faulting/Dowels in Overlay** Drainage/Dowels in Overlay**
Faulting <0.25 in	None	Thin	Repair Voids? – Drainage
Faulting > 0.25 in	Yes—Cold Milled None	Thin Thick	Repair Voids? – Drainage Repair Voids? – Drainage
Surface Spalled/Extensive D-Cracking	Yes—Filled with Cold Patch None	Thin Thick	Mismatch joints/Dowels in Overlay** Mismatch joints/Dowels in Overlay**
Reactive Aggregate	None	Thin	Drainage
JOINTED CONCRETE PAVEMENTS			
Spalled and Deteriorated Joints	Yes—Filled with Cold Patch None	Thin Thick	Mismatch joints Mismatch joints
CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS			
Punchouts	Yes—Replaced Full-Depth	Thick	Subgrade Repair/Drainage
COMPOSITE PAVEMENTS			
Rutting < 2 in	None	Thin/None	Joint Sawing Depth
Rutting > 2 in	Yes—Cold Milled	Thin	Drainage
Medium- to High-severity Reflective Cracks	Yes—Repair Existing PCC None	Thin Thin	Mismatch Joints from Reflective Cracks Mismatch Joints from Reflective Cracks
Remove Asphalt Surface	Yes—Repair Existing PCC	Thin	Drainage

* Thick Interlayer > 0.5 in, Thin Interlayer < 0.5 in

** Particularly for heavy traffic routes

Note: If poor load transfer exists (< 50 percent deflection load transfer), a minimum HMA interlayer thickness of 38 mm (1.5 in) is recommended.

Because of the concerns about high curling stresses, a shorter joint spacing is typically recommended for unbonded JPCP overlays than for new JPCP designs. The current AASHTO guide recommends limiting the maximum joint spacing to 21 times the slab thickness. For example, for a 203-mm (8-in) overlay, the maximum recommended joint spacing is 4.2 m (14 ft). In general, this recommendation is reasonable for slab thicknesses up to about 229 mm (9 in), except that joint spacing less than 3.7 m (12 ft) is not warranted because that would make the slabs shorter than the lane width. For thicker slabs (e.g., 241 mm [9.5 in] or greater), the joint spacing based on 21 times the slab thickness is excessive, which increases the risk of premature slab cracking (especially top-down cracking). Indeed, there are documented cases demonstrating that 6.1-m (20-ft) joint spacing can be excessive even for new PCC pavements constructed 330 mm (13 in) thick (Yu and Khazanovich 2001). In general, the risk of premature cracking on unbonded PCC overlays can be greatly minimized by limiting the maximum joint spacing to 4.5 m (15 ft), even for very thick overlays.

Guidelines for joint spacing are sometimes given in terms of the L/ ratio, where L is the joint spacing and is the radius of relative stiffness:

$$l = \sqrt[0.25]{\frac{E \cdot h^3}{12 \cdot (1 - ì^2) \cdot k}} \tag{4-4}$$

where:

	=	Radius of relative stiffness, mm (in)
E	=	PCC elastic modulus, MPa (lbf/in^2)
h	=	Slab thickness, mm (in)
ì	=	Poisson's ratio
k	=	Modulus of subgrade reaction, MPa/mm, (lbf/in^2/in)

The maximum recommended L/ ratio for unbonded overlays is 4.5 to 5.5 (ERES 1999a). This recommendation is comparable to the guidelines given in the AASHTO guide for typical design conditions. Again, however, the results may be unconservative for thick slabs (e.g., greater than 241 mm [9.5 in]) and overly conservative if the k-value is very high.

Current pavement design practices are away from the use of JRCP designs, and they are rarely constructed any more. This is true for both new construction and for overlay construction. If used, the recommended maximum joint spacing for JRCP is 9.1 m (30 ft) (FHWA 1990).

4.2.7 *Load Transfer Design*

Joint performance in unbonded PCC overlays is significantly better than in new JPCP construction because of the load transfer provided by the underlying pavement serving as "sleeper" slabs (Hall, Darter, Seiler 1993; ERES 1999a). To maximize the benefits of load transfer from the underlying pavement, mismatching joints is recommended for unbonded overlays (see figure 4-1). However, the use of doweled joints is still highly recommended for unbonded PCC overlays that will be subjected to heavy truck traffic to avoid corner breaks and to minimize faulting. Without dowels, the risk of corner breaks is high in unbonded overlays because of the very stiff support conditions.

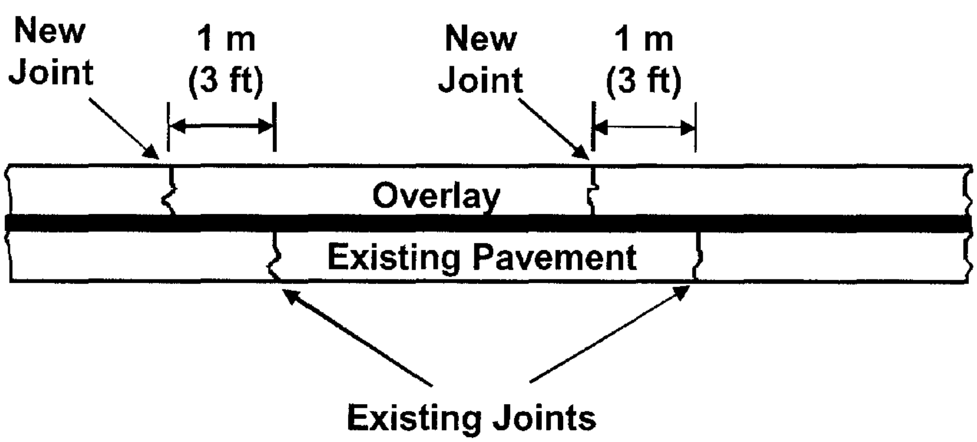

Figure 4-1. Illustration of mismatched joints (ACPA 1990b).

Studies have shown that adequately sized dowels must be provided to obtain good faulting performance (Snyder et al. 1989; Smith et al. 1997). Dowel diameter is often selected based on slab thickness, but traffic may be a more important factor for consideration. Recommended load transfer designs are summarized in table 4-2.

Table 4-2. Recommended load transfer designs (Smith and Hall 2001).

Design Feature	**Recommendation**
Dowel Diameter	<u>Design Catalog (Darter et al. 1997)</u> < 30 million ESALs 32 mm (1.25 in) bar 30 – 60 million ESALs 38 mm (1.50 in) bar > 90 million ESALs 41 mm (1.625 in) bar <u>Industry (ACPA 1991a)</u> < 10 in slab 32 mm (1.25 in) bar ≥ 10 in slab 38 mm (1.50 in) bar
Dowel Length	460 mm (18 in)
Dowel Spacing	305-mm (12-in) center-to-center across the joint *Alternative*: Cluster dowels in wheelpath (see figure 4-2).
Dowel Coating	Epoxy

The recommended number and spacing of dowels is the same as those for new pavements. In general, uniform 305-mm (12-in) spacing is recommended, but nonuniform spacing has also been used successfully. In the nonuniform dowel spacing design, the dowels are concentrated in the wheelpaths (Darter et al. 1997). One recommended design for variable dowel bar spacing is illustrated in figure 4-2.

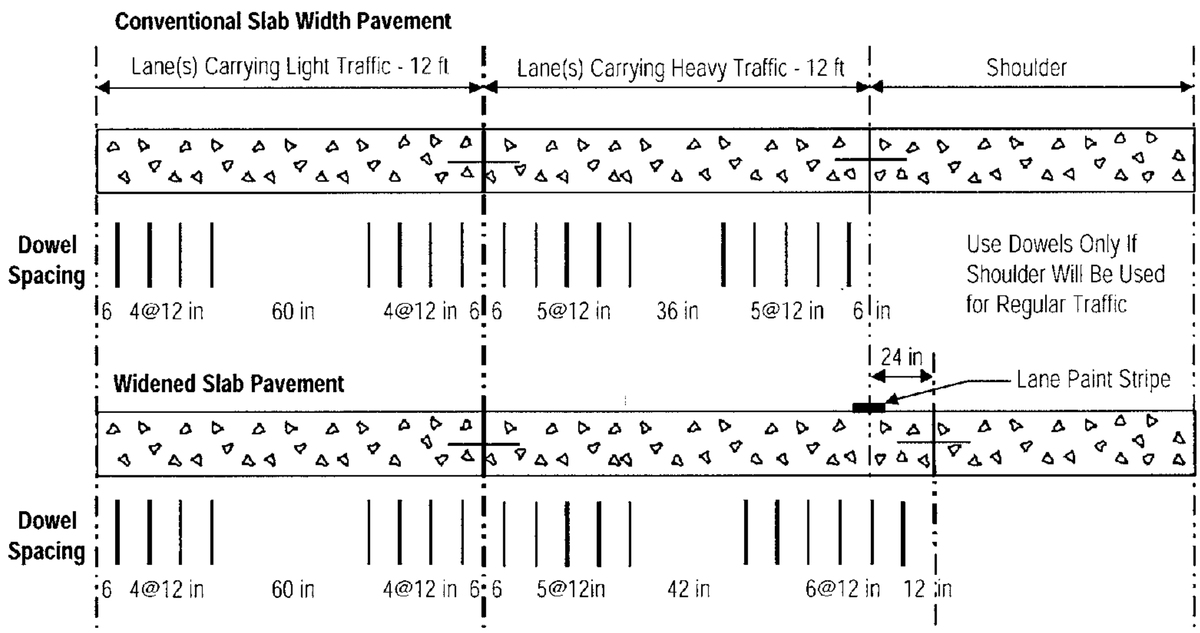

Figure 4-2. Recommended design for variable dowel bar spacing (Darter et al. 1997).

4.2.8 *Joint Orientation*

In general, perpendicular joints are recommended for unbonded overlays. Only limited data are available on the effects of joint orientation on performance of unbonded overlays. On new JPCP, studies have shown that skewed joints can be effective in reducing faulting on nondoweled pavements but have no effect when used on properly doweled pavements (Yu et al. 1998b; Khazanovich et al. 1998). Furthermore, JPCP designs with skewed joints constructed on a stiff base (cement treated or lean concrete) are prone to corner breaks. Therefore, perpendicular joints are recommended for unbonded overlays.

4.2.9 *Reinforcement Design*

Reinforcement design for unbonded CRCP overlays is similar to that for new design. The recommended minimum steel content is 0.60 percent, and the use of deformed bars is strongly recommended (Darter et al. 1997). The depth of reinforcing steel has a significant effect on crack opening, and studies have shown that a higher steel placement leads to tighter cracks and better long-term performance (Dhamrait and Taylor 1979; Roman and Darter 1988). However, a minimum steel cover of 64 mm (2.5 in) is still recommended for corrosion protection.

Unbonded JRCP overlays are no longer routinely constructed. If used, deformed bars or deformed welded wire fabric (WWF) is recommended at a minimum steel content of 0.19 percent (Darter et al. 1997). A minimum steel cover of 64 mm (2.5 in) is again recommended.

4.2.10 *PCC Mix Design*

As described in chapter 2, the same PCC mixes used for new construction are generally used for unbonded overlays. For projects in congested urban areas, however, extended lane closures due to pavement rehabilitation may be highly undesirable. For those projects, the use of fast-track paving may be considered to minimize traffic disruptions. Numerous fast-track PCC mixes are available that can provide the strength required for opening to traffic in 12 hours or less, and the techniques for fast-track paving are well established (ACPA 1994a; FHWA 1994; ACI 2001). Fast-track paving has been used successfully on a number of projects where traffic congestion or accessibility is a critical issue (ACPA 1994a; ACI 2001). However, little information is available on the long-term performance and durability of pavements constructed using high-early-strength PCC mixtures.

4.2.11 *Edge Support*

Edge support refers to design features provided to either reduce critical edge stresses or reduce the occurrences of edge-loading conditions that cause the highest stresses in PCC pavements. These features include tied PCC shoulders and widened slabs. Studies have shown that, in general, widened slabs are more effective than tied PCC shoulders in contributing to the performance of the pavement (Smith et al. 1995). The effectiveness of tied PCC shoulders depends on the effectiveness of the tie system, and shoulders constructed monolithically with the mainline pavement provide significantly better support than those paved separately. Even with an effective tie system, however, tied PCC shoulders have not significantly improved faulting performance, although they have been shown to be effective in reducing critical edge stresses (Smith et al. 1995; Yu, Smith, and Darter 1995).

Widened slabs are effective in improving both faulting and cracking performance (Smith et al. 1995). One study showed that the structural benefit of widened slabs is roughly equivalent to about 25 mm (1 in) of additional slab thickness (Yu, Smith, and Darter 1995). However, widened slabs should be used with care on unbonded overlays because of the increased risk of longitudinal cracking. This is because unbonded overlays are constructed on very stiff foundations and are generally constructed thinner than new PCC pavements, both of which increase the potential for longitudinal cracking.

Although widened slabs are a better edge support design feature for new pavements, tied PCC shoulders may be preferable for unbonded PCC overlays. This is because of the increased risk of cracking due to high curling stresses (resulting from stiff support conditions) of the widened slab. Furthermore, if the unbonded PCC overlay is doweled, the effect of the widened slab on the faulting performance is minimal. Finally, since shoulder work is required for unbonded overlays, an opportunity exists for including tied PCC shoulders as part of the overlay project.

4.2.12 *Lane Widening*

Construction of unbonded overlays may involve the widening of an old pavement with narrow traffic lanes or the addition of new travel lanes or the extension of ramps. On such projects, the potential for longitudinal cracking is again a concern. In general, prior to placing the separator layer for the overlay, the widened portion should be provided with a cross section that closely matches that of the underlying pavement (AASHTO 1993). The recommended construction is illustrated in figure 4-3, and can use either a PCC or HMA fill beneath the widening. A study conducted by Minnesota Department of Transportation (MnDOT) showed no significant difference in performance between the widening accomplished using either HMA and PCC materials (Engstrom 1993). The main concern is to ensure that the widened portion provides adequate support without settlements or loss of support (ERES 1999a).

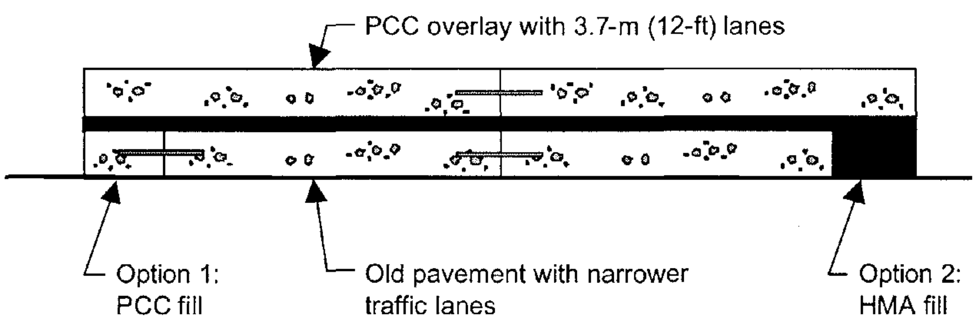

Figure 4-3. Lane widening options for unbonded overlay construction.

4.2.13 *Drainage*

The effectiveness of subsurface drainage on the performance of unbonded overlays is not well known because very little performance data are available. Since unbonded overlays are structurally similar to new PCC pavements constructed on a lean concrete base, the effects of edgedrains may be similar. The possible benefits of properly designed, constructed, and maintained edgedrains on new PCC pavements include the following (Smith et al. 1995; ERES 1999b):

- Reduced pumping and faulting.
- Lower rate of crack deterioration on CRCP.
- Improved material performance (e.g., D-cracking, HMA separator layer stripping).

Some states (e.g., Minnesota and Pennsylvania) have experimented with permeable asphalt-treated separator layer; however, extensive stripping of this layer was observed on several projects investigated in Minnesota (ERES 1999a). Further research is needed to determine the effectiveness of permeable separator layers; however, conceptually, the appropriateness of using a permeable separator layer is questionable. This is because on an unbonded overlay, the permeable separator layer transfers the infiltrated water to joints and cracks in the underlying pavement. While the water in the permeable separator layer can be drained quickly, the permeable layer offers no advantage in draining the water in the underlying pavement. Without the permeable separator layer, less water is allowed into the underlying pavement. Therefore, while a permeable separator may improve the exposure condition for the overlay PCC, it may actually worsen the drainage condition for the underlying pavement.

Unfortunately, no information is currently available to substantiate any of the above-listed performance benefits, so further research is needed to determine the effectiveness of edgedrains on the structural performance of unbonded overlays. However, based on the evaluation of edge drains in new PCC pavements, it might be that edgedrains could improve material performance in whitetopping overlays (Smith et al. 1995; ERES 1999b).

4.2.14 *Job-Site Considerations*

Several job-site factors require special considerations, including bridge approaches, overhead clearances, and shoulders. At bridge approaches or underpasses, reconstruction of a short section may be necessary to satisfy vertical clearance requirements. Reconstruction requires sections at both ends to provide a smooth transition between the overlay and the reconstructed section. The recommended taper length for the transition is 90 to 150 m (300 to 500 ft) (ACPA 1990b). A similar transition section is also needed at bridge approaches. An example transition section is shown in figure 4-4.

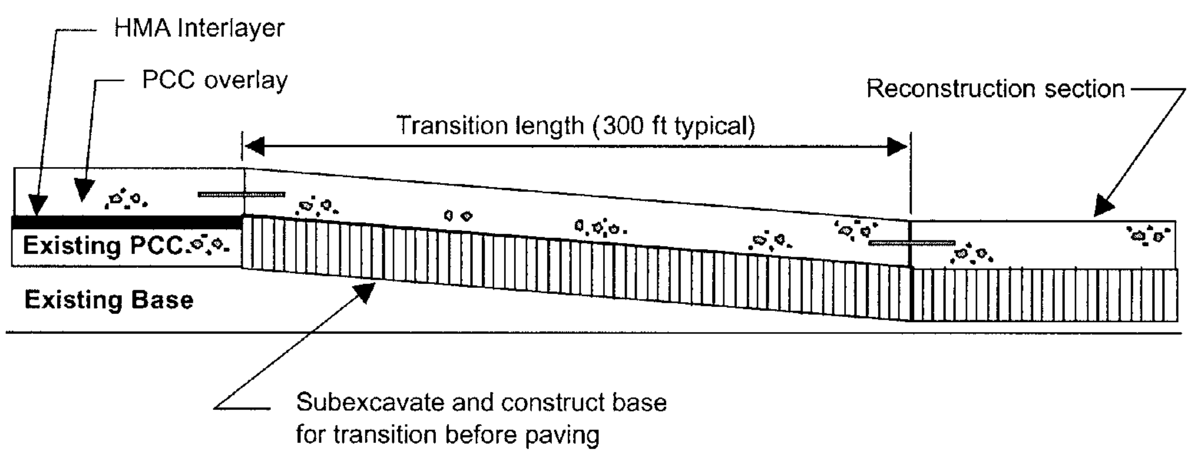

Figure 4-4. Example transition detail for bridge approaches and overpasses (ACPA 1990b).

The construction of unbonded overlays requires shoulder work. Either PCC or HMA shoulders could be provided, but as discussed above, tied PCC shoulders may be advantageous for unbonded overlays, particularly in urban areas or in areas where the shoulder may at some point in the future be used as a travel lane. When used, it is important that the transverse joints in the PCC shoulder match those in the mainline pavement.

4.3 Construction of Unbonded PCC Overlays

The construction of unbonded overlays involves limited preoverlay repairs, placement of the separator layer, and the placement of the PCC overlay. In lieu of preoverlay repairs, the existing pavement may be fractured to provide uniform support. These construction activities are briefly described in the following sections.

4.3.1 *Preoverlay Repair*

The techniques for preoverlay repairs are the same as those for PCC pavement restoration. Recommended practices for these techniques are summarized in several references by the American Concrete Pavement Association (ACPA 1990b; ACPA 1994a; ACPA 1994b, and ACPA 1995) and in an National Highway Institute (NHI) training course (Hoerner et al. 2001).

As emphasized previously, unbonded overlays do not require significant preoverlay repairs. Typical repairs consist of full-depth repair of badly shattered slabs for JCP and punchouts for CRCP. Section 4.2.3 provides additional discussion on preoverlay repair.

4.3.2 *Slab Fracturing*

Slab fracturing may be appropriate if the existing pavement is extensively deteriorated or if it has severe MRD. Depending on the type of the pavement, the existing pavement may be cracked and seated (JPCP), break and seated (JRCP or CRCP), or rubblized (all pavement types), as described below:

- Crack and seat. This technique involves fracturing the existing JPCP into pieces 0.3 to 1.2 m (1 to 4 ft) on a side by inducing full-depth cracks using a modified pile driver, guillotine hammer, whip hammer, or other equipment. After fracturing, the broken PCC is firmly seated using a pneumatic roller prior to placing the overlay to prevent rocking.

- Break and seat. This technique is similar to crack and seating, except conducted on JRCP and using greater impact energy. The greater impact energy is needed to rupture the steel or break its bond with the PCC in order to ensure independent movement.

- Rubblization. This technique involves breaking the existing pavement into pieces no larger than 152 mm (6 in) on a side using a vibratory beam breaker or resonant frequency pavement breaker. After rubblization, the broken pieces of PCC are compacted using a vibratory roller before placing the overlay.

The above techniques are used more commonly prior to placing an HMA overlay, but there has been some use for PCC overlays as well. The techniques for slab fracturing are documented in numerous references, including Hoerner et al. (2001); Thompson (1999); Ksaibati, Miley, and Armaghani (1999); and Dykins and Epps (1987).

Weak subgrades can cause looseness and shifting of the fractured PCC, especially under saturated conditions, resulting in premature surface failures. For HMA overlays, rubblization is not recommended if the underlying subgrade is very weak (Ksaibati, Miley, and Armaghani 1999). Similar concerns may be applicable to unbonded overlays.

4.3.3 *Separator Layer Placement*

The placement of a separator layer does not involve any special or unusual construction techniques. The procedure depends on the interlayer material, but standard application procedures apply. Prior to placing the separator layer, the existing pavement surface needs to be swept clean of any loose materials. Either a mechanical sweeper or an air blower may be used (ACPA 1990b).

With bituminous separator layers, precautionary steps may be needed to prevent the development of excessively high surface temperatures prior to PCC placement. If the surface of the separator layer is uncomfortable to touch with an open palm, water fogging is recommended to cool the surface (ACPA 1991b; McGhee 1994). An alternative to this is to construct the PCC overlay at night.

As described in chapter 2, whitewashing of the bituminous surface using a lime slurry may also be performed in order to cool the surface (ACPA 1990b). However, this practice may lead to debonding between the overlay PCC and the separator layer, and some degree of friction between the overlay PCC and the separator layer is believed to be beneficial to the performance of unbonded overlays (ERES 1999a).

4.3.4 *PCC Placement and Finishing*

The procedures for placing and finishing PCC for unbonded overlays are the same as those for new pavements. Standard practices apply, and the NHI training course on *Construction of Portland Cement Concrete Pavements* (ACPA 2000b) provides a good summary of recommended practices for PCC placement and finishing. The only reported problem with PCC placement and finishing for unbonded overlays is the difficulty of anchoring dowel baskets in relatively thin separator layers (ERES 1999a). While agencies need to be aware of this potential problem, it is manageable. For example, some contractors have used nail guns to secure the baskets to the underlying PCC pavement. Alternatively, pavers equipped with dowel bar inserters could be used to avoid this problem altogether.

4.3.5 *Texturing*

Texturing of the finished PCC pavement surface is required to ensure adequate surface friction of the roadway. Initial texturing is often done with a burlap drag or turf drag, with the final texturing provided by tining. Tining provides macrotexture, which contributes to surface friction by tire deformation, and also channels surface water out from between the pavement and the tire. Tining should be conducted as soon as the sheen goes off of the PCC. Additional guidance on surface tining is found in reports by Kuemmel et al. (2000) and by ACPA (2000c).

Tining has traditionally been conducted transversely and at uniform intervals, but recent studies suggest that uniformly spaced transverse tining produces irritating pavement noise (Larson and Hibbs 1997; Kuemmel et al. 2000). Consequently, some agencies are experimenting with transverse tining that is randomly spaced and skewed to the centerline of the pavement, the pattern of which must be carefully designed and constructed in order to minimize discrete noise frequencies that are most objectionable to the human ear (Kuemmel et al. 2000). In addition, some agencies are investigating the use of longitudinal tining, which produces lower noise levels than either uniformly or randomly spaced transverse tining (Kuemmel et al. 2000). Current recommendations for tining are as follows:

- The depth of tining should be 3 to 5 mm (0.12 to 0.20 in), and the individual tines should be 3.0 mm (0.12 in) wide (Kuemmel et al. 2000; WisDOT 2001).
- When tining transversely, tines should be spaced randomly at a minimum spacing of 10 mm (0.4 in) and a maximum spacing of 57 mm (2.2 in) apart (WisDOT 2001). Either skewed or nonskewed transverse tining may be conducted, although skewed tining is quieter (WisDOT 2001). A recommended random tining pattern specifically developed to avoid repeating tine patterns over the typical passenger car's wheelbase is available in the Wisconsin DOT Construction and Materials Manual (WisDOT 2001).
- When tining longitudinally, the tining should be done parallel to the centerline of the pavement with tines uniformly spaced at 19 mm (0.75 in) intervals (ACPA 1999b; ACPA 2000c).

However, both AASHTO and FHWA recommend that friction and safety not be compromised to obtain slight, usually short-term, reductions in noise levels (Smith and Hall 2001).

4.3.6 *Curing*

The curing of unbonded overlays generally follows conventional practices. The most common practice is to spray liquid, white-pigmented, membrane curing compound, and this is also the recommended method for unbonded overlays (ACPA 1990b). For conventional PCC mixtures under normal placement conditions, application rates between 3.7 and 4.9 m^2/l (150 to 200 ft^2/gal) are common.

For unbonded overlays, particular attention should be paid to the environmental conditions during construction to avoid excessively high temperature gradients through the PCC during curing. Studies have shown that an excessive temperature gradient through the PCC slab at the time of PCC hardening can cause locking-in of a significant amount of curling into PCC slabs, which can be deleterious to fatigue performance (Yu, Smith, and Darter 1995; Yu et al. 1998a; Byrum 2000). The locked-in construction curling is a special concern for unbonded overlays because of the very stiff support conditions. The HIPERPAV computer program may be used to predict the potential for uncontrolled early-age cracking of PCC pavements (including unbonded overlays) for given environmental and mix design inputs (McCullough and Rasmussen 1999a; McCullough and Rasmussen 1999b).

4.3.7 *Joint Sawing and Sealing*

As with new PCC pavements, timely sawing is critical on unbonded overlays to avoid random cracking. The same procedures and recommendations given for new pavements are applicable to unbonded overlays. Joint sawing recommendations are given in numerous references, including Okamoto et al. (1994) and the NHI training course on PCC pavement construction (ACPA 2000b). For fast-track projects, green sawing may be advantageous because that procedure allows sawing at PCC strengths as low as 1.0 MPa (150 lbf/in^2) using ultra-light saws (Hoerner et al. 2001).

4.4 Performance of Unbonded Overlays

The performance of unbonded overlays has generally been very good. Unbonded overlays are usually considered a long-term rehabilitation solution, and they are expected to provide the level of service and performance life comparable to those of new PCC pavements. A review of available performance data shows that for the most part unbonded overlays have been effective in meeting those expectations. The following presents a brief discussion of the factors affecting the performance of unbonded overlays, followed by an overview of field performance.

4.4.1 *Factors Affecting Performance*

The performance of unbonded overlays is relatively insensitive to the condition of the underlying pavement and the overlays can be placed with minimal preoverlay repair. However, various factors can significantly influence the performance of unbonded overlays, including the following:

- Condition of the existing pavement (type, severity, and extent of distresses).
- Preoverlay repair.
- Separator design (type of material and layer thickness).
- Overlay design features (overlay thickness, joint spacing, load transfer design, reinforcement design, and drainage design).
- Traffic (axle weights and number).
- Climate (temperature and moisture conditions).
- Construction quality and curing.

Further research is needed to quantify the effects of many of the above factors on the performance of unbonded overlays. While there is significant experience in the design and construction of unbonded overlays, conclusive data documenting the basis of many design and construction practices that have become standard today are not available in many cases. For example, the effects of the existing pavement condition or the effects of preoverlay repairs on the performance of unbonded overlays are not well documented.

4.4.2 *Current State Practices*

The use of unbonded overlays dates back as early as 1916 (McGhee 1994). NCHRP Synthesis 204 (McGhee 1994) catalogs 392 unbonded and partially bonded PCC overlay projects, of which over half were constructed since 1970. An extensive survey of state practice conducted in 1996 under NCHRP Project 10-41 (ERES 1999a) showed that twenty-three states have constructed unbonded PCC overlays since 1970. Of those, eleven states constructed more than five projects during that period, with Colorado, Iowa, Minnesota, Ohio, Pennsylvania, and Wisconsin each constructing ten or more projects.

Historically, unbonded JPCP has been the most popular type of PCC resurfacing. NCHRP Synthesis 204 showed that nearly two-thirds of PCC overlays are JPCP. The survey conducted under NCHRP Project 10-41 showed similar results, with unbonded JPCP overlays making up 65 percent of the projects reported (ERES 1999a). Recent trends are toward even greater use of JPCP designs for unbonded PCC overlays.

Unbonded PCC overlay thicknesses range from 76 mm (3 in) to 330 mm (13 in). However, very few projects have overlay thicknesses less than 152 mm (6 in). The distribution of unbonded overlay thickness is shown in figure 4-5. This figure includes JPCP, JRCP, and CRCP overlays. In general, CRCP overlays are thinner than JCP overlays.

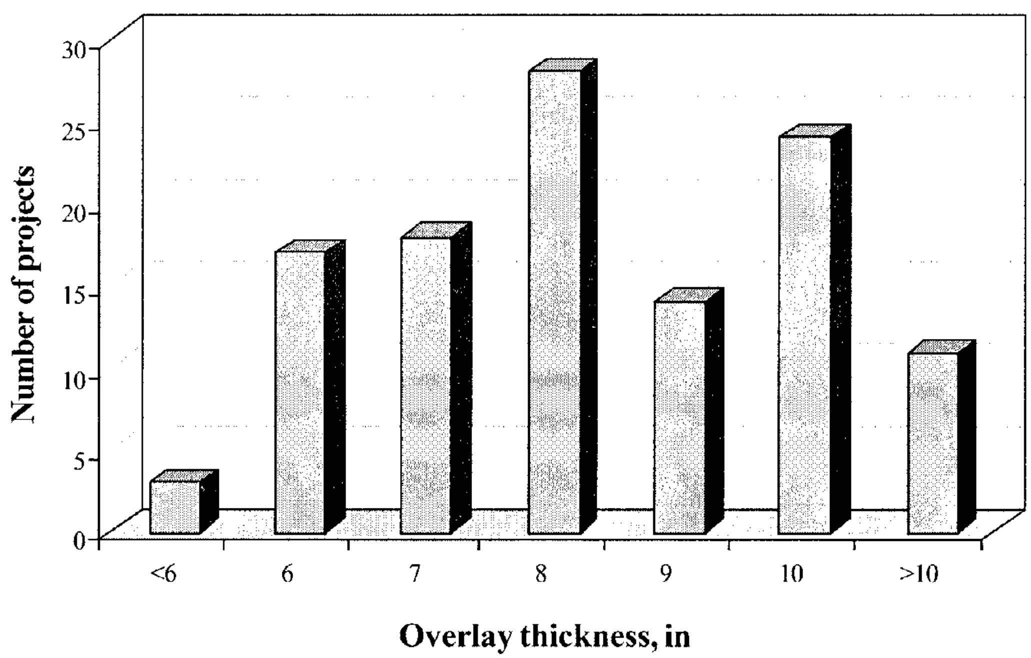

Figure 4-5. Distribution of unbonded overlay thickness (ERES 1999a).

The design features of the in-service unbonded overlays are consistent with recommended design practice. The distribution of separator layer types commonly in use is shown in figure 4-6. By far the most common type of separator layer is HMA (69 percent or responding agencies). Although other types of separator layers are also used, bituminous materials make up 91 percent of all separator layer types reported in use.

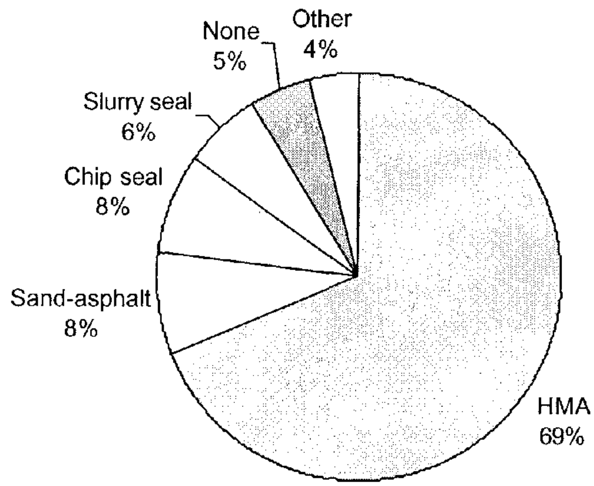

Figure 4-6. Distribution of separator layer type (ERES 1999a).

Figure 4-7 shows the distribution of the thicknesses of the HMA separator layers used by highway agencies. The predominant thickness of the HMA separator layer is 25 mm (1 in), which is the typical recommended design. Nearly all projects incorporating an HMA separator layer in the database have a layer thickness 19 mm (0.75 in) or greater.

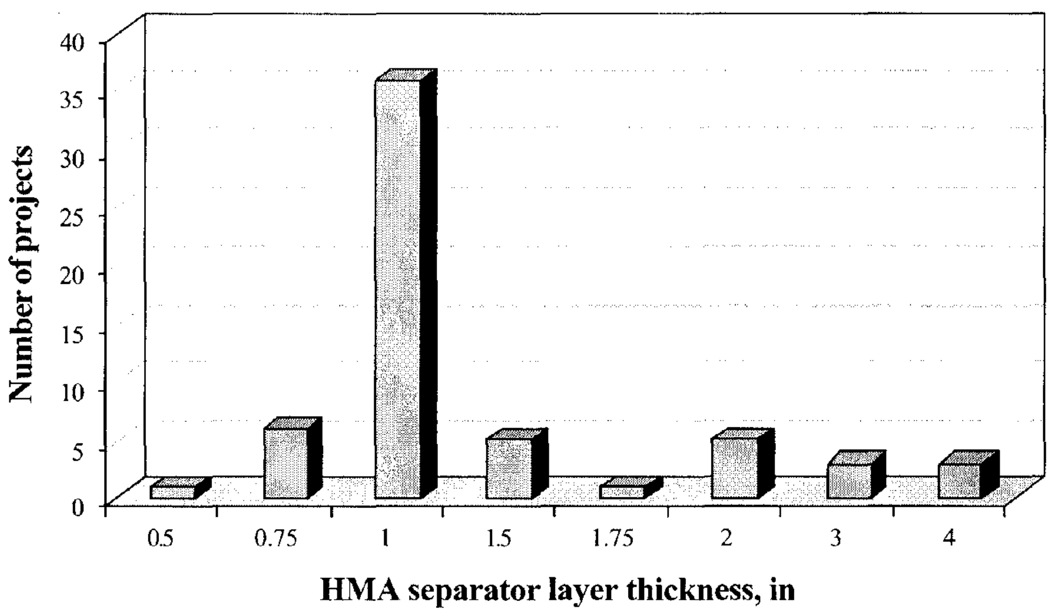

Figure 4-7. Distribution of HMA separator layer thickness (ERES 1999a).

The joint spacings for the unbonded JCP overlays are similar to those for new pavements. For example, table 4-3, which summarizes design and construction information for selected unbonded overlay projects, shows that the maximum joint spacing for JPCP is 6.1 m (20 ft) and for JRCP it is 18.7 m (61.5 ft). This table also shows that many of the JPCP sections incorporate random joint spacings.

Table 4-3. Summary table of unbonded overlay projects with available performance data and design information (ERES 1999a).

ID	Hwy	Overlay Type	Thick. in	Const. Year	Insp. Year	Age yrs	Traffic MESAL	Rating	Exist. Pvt.	Preoverlay Repair	Interlayer Type	Interlayer h	Joint Spacing, ft	Dowel
GA-4	I-75	CRCP	7 & 8	1972	1993	21.0	30.0	Good	JPCP	None	None	n/a	n/a	n/a
IL-1	I-55	CRCP	9.0	1974	1986	12.0	9.0	Good	JRCP	Limited patching	AT	4.0	n/a	n/a
IL-2	I-55	CRCP	8.0	1970	1986	16.0	9.0	Good	JRCP	Limited patching	AT	4.0	n/a	n/a
IL-3	I-70	CRCP	8.0	1967	1986	19.0	17.0	Good	JRCP	Limited patching	AC	4.0	n/a	n/a
GA-1	I-85	CRCP	6.0	1975	1993	18.0	16.0	Fair	JPCP	CPR as needed	None	n/a	n/a	n/a
IL-3	I-70	CRCP	7.0	1967	1986	19.0	17.0	Fair	JRCP	Limited patching	AC	4.0	n/a	n/a
ND-1	I-29	CRCP	8.0	1974	1993	19.0	2.0	Fair	JPCP		AC	2.0	n/a	n/a
ND-2	I-29	CRCP	6.0	1972	1993	21.0	3.0	Fair	JPCP		AC	2.0	n/a	n/a
AR-1	I-30	CRCP	6.0	1992	1995	3.0	6.0	Poor	JRCP	Limited patching	AC	1.0	n/a	n/a
GA3	I-85	CRCP	3.0	1975	1985	10.0	7.0	Poor	JPCP	CPR where required	None	n/a	n/a	n/a
GA5	I-85	CRCP	4.5	1975	1985	10.0	7.0	Poor	JPCP	CPR where required	None	n/a	n/a	n/a
IL-3	I-70	CRCP	6.0	1967	1986	19.0	17.0	Poor	JRCP	Limited patching	AC	4.0	n/a	n/a
MD-1	I-70	CRCP	6.0	1974	1985	11.0	8.0	Poor	JRCP		AC	1.0	n/a	n/a
PA-5	I-90	CRCP	7.0	1976	1993	17.0	20.0	Poor	JRCP	5% patching	SA	1.0	n/a	n/a
WI-1	I-94	CRCP	8.0	1980	1993	13.0	12.0	Poor	JRCP	Patching	ST	1.8	n/a	n/a
CA-1	I-80	JPCP	10.2	1993	1994	1.0	2.0	Good	JPCP	Shattered slab replacement	AC	1.0	12-15-13-14	None
CA-2	I-80	JPCP	10.2	1992	1994	2.0	3.0	Good	JPCP	Shattered slab replacement	AC	1.0	12-15-13-14	None
CA-5	I-80	JPCP	10.2	1991	1992	1.0	1.0	Good	JPCP	Shattered slab replacement	AC	1.0	12-15-13-14	None
CA-6	I-8	JPCP	6.0	1970	1991	21.0	6.0	Good	JPCP		AC	0.5	13-19-18-12	
CO-10	I-76	JPCP	8.0	1990	1996	6.0	2.0	Good	JPCP	Some slab replacement	AC	1.0		None
CO-3	I-25	JPCP	8.0	1987	1996	9.0	12.0	Good	JPCP	Some slab replacement	ST			None
CO-5	I-25	JPCP	8.0	1985	1996	11.0	14.0	Good	JPCP	Some slab replacement	ST			None
CO-6	I-25	JPCP	7.8	1984	1996	12.0	20.0	Good	JPCP	Some slab replacement	ST			None
CO-7	I-76	JPCP	8.5	1992	1996	4.0	1.0	Good	JPCP	Some slab replacement	AC	0.8		None
DE-1	I-495	JPCP	12.0	1992	1996	4.0	4.0	Good	CRCP	Spall repair	AC	2.0	20	
IA-7	W47	JPCP	7.0	1990	1992	2.0	1.0	Good	JPCP		AC	1.0	15	
MN-10	I-90	JPCP	8.0	1992	1994	2.0	1.0	Good	CRCP		AC	1.0	15-random	1
MN-2	TH 212	JPCP	7.0	1985	1993	8.0	10.0	Good	JPCP		SA	1.5	15-random	1
MN-4	I-35	JPCP	8.0	1987	1994	7.0	10.0	Good	JRCP		AC	1.0	15-random	1
MN-5	I-90	JPCP	9.0	1988	1993	5.0	6.0	Good	JRCP		AC	1.0	15-random	1.25
MN-6	I-90	JPCP	8.0	1990	1994	4.0	3.0	Good	JRCP		AC	1.0	15-random	1
MN-7	I-90	JPCP	8.0	1991	1994	3.0	2.0	Good	CRCP		AC	1.0	15-random	1
MN-8	TH 52	JPCP	9.5	1992	1994	2.0	1.0	Good	JPCP		AC	2.0	15-random	1.25
MN-9	I-35	JPCP	7.5	1992	1994	2.0	1.0	Good	JRCP		AC	1.0	15-random	1
NB-1	SU-281	JPCP	7.0	1988	1993	5.0	2.0	Good	JRCP	FDR	CS	0.5	16.5	None
OH-1	US-33	JPCP	7.0	1982	1986	4.0	3.0	Good	JRCP	Pothole patching	AC	0.8	12-15-13-14	None
OH-3	US-33	JPCP	8.0	1985	1994	9.0	3.0	Good	CRCP	Level slags with AC	AC	0.8	12-15-13-14	None
PA-15	I-78	JPCP	12.0	1985	1994	9.0	5.0	Good	CRCP	Patching	SA	1.0	20	1.5
PA-16	I-78	JPCP	12 & 13	1991	1995	4.0	8.0	Good	JRCP	AC patching	PAC	4.0	20	1.5
PA-17	I-80	JPCP	13.0	1993	1994	1.0	4.0	Good	JRCP	AC patching	PAC	2.5	20	1.5
PA-18	I-1	JPCP	10.0	1988	1995	7.0	10.0	Good	JRCP		AC	1.0		
TX-6	I-45	JPCP	10.0	1968	1990	22.0	5.0	Good	CRCP		AC	3.9		
GA-2	I-85	JPCP	6.0	1975	1993	18.0	16.0	Fair	JPCP	Underseal, FDR, PDR	None	n/a	15	1.125
IA-10		JPCP	7.0	1987	1992	5.0	1.0	Fair	JPCP		SS	0.3	15	None
IA-8	I-0	JPCP	7.0	1987	1992	5.0	1.0	Fair	JPCP		AC	1.0	15	None
IA-9	I-0	JPCP	7.0	1991	1992	1.0	0.0	Fair	JPCP		AC	1.0	15	None
CA-4	I-80	JPCP	8.0	1989	1992	3.0	8.0	Poor	JPCP	Shattered slab replacement	AC		12-15-13-14	None
IL-4	I-88	JPCP	8.0	1981	1989	8.0	10.0	Poor	JRCP		SA	0.5	14.5-random	None
KS-1	US-24	JPCP	6.0	1978	1988	10.0	4.0	Poor	JRCP		AC	1.0	15	None
MN-1	TH 71	JPCP	5.5	1977	1995	18.0	1.0	Poor	JPCP		SA	1.0	13-16-14-19	0.75
MI-2	I-96	JRCP	7.0	1984	1993	9.0	5.0	Good	CRCP	AC patching	AC	0.8	41 ave.	1.25
MI-3	US-23	JRCP	7.0	1984	1993	9.0	6.0	Good	JRCP	AC patching	AC	0.8	41 ave.	1.25
MN-3	I-90	JRCP	8.5	1986	1994	8.0	10.0	Good	CRCP		AC	1.0	27	0.75
MO-1	I-70	JRCP	11.0	1992	1995	3.0	5.0	Good	JRCP		AC	Varies	61.5	1.5
MS-2	I-20	JRCP	10.0	1990	1993	3.0	5.0	Good	JRCP		AC	3.0	21	1.25
MS-3	I-20	JRCP	10.0	1990	1993	3.0	5.0	Good	JRCP		AC	3.0	21	1.25
MS-4	I-20	JRCP	10.0	1990	1993	3.0	4.0	Good	JRCP		AC	3.0	21	1.25
OH-2	Rte 70	JRCP	10.0	1984	1986	2.0	4.0	Good	JRCP	FDR, underseal	AC	1.0	60	1.375
PA-14	I-80	JRCP	10.0	1988	1994	6.0	5.0	Good	JRCP	Slab stabl., jt & crack rpr, ED	AC	1-4	20	1.5
MS-5	I-20	JRCP	10.0	1987	1993	6.0	7.0	Fair	JRCP		None	n/a	21	1
AR-3	I-40	JRCP	10.0	1985	1995	10.0	15.0	Poor	JRCP		AC	1.0		1.25
MS-1	I-59	JRCP	6.0	1982	1993	11.0	10.0	Poor	JRCP		AC	1.5	20	1

AC = Asphalt concrete, AT = Asphalt treated, SA = Sand asphalt, ST = Surface treatment, CS = Chip seal, SS = Slurry seal, PAC = Permeable AC

Poor = < 10 years old with M-H distresses; Fair = 10-20 years old with M-H distress; Good = >20 years old or no significant distress.

4.4.3 *Overall Field Performance*

In general, the performance of unbonded overlays has been very good (McGhee 1994; ERES 1999a). Where premature failures have occurred, the failures are often attributed to the following causes (Voigt, Carpenter, and Darter 1989; McGhee 1994; ERES 1999a):

- Poor separator layer design.
- Excessive joint spacing.
- Inadequate slab thickness.

Table 4-3 summarizes what is believed to be the best available information on the performance of unbonded overlays, and includes data collected under NCHRP 10-41, data provided by SHAs, and data obtained from the LTPP database. While table 4-3 only provides a general rating of overlay condition, that information is sufficient to draw some useful conclusions. The more critical limitation is that the database does not include a sufficient number of sections to make certain key comparisons possible. For example, the effects of preoverlay repair on overlay performance cannot be determined based on projects included in table 4-3.

The effects of separator design on overlay performance also cannot be determined conclusively based on the data shown in table 4-3. The following observations can be made from table 4-3 regarding the effects of separator layer on unbonded overlay performance:

- In some cases, overlays constructed without any separator layer (e.g., GA-4 and GA-1) performed as well as, or better than, those provided with the recommended separator layer design (25 mm [1 in] or more of HMA).

- The projects that were provided with 25-mm (1-in) HMA separator layer did not universally provide good performance (e.g., AR-1, MD-1, KS-1, and AR-3).

- Several other separator layer designs also gave good performance, including sand-asphalt, surface treatment, chip seal, asphalt-treated aggregate, and permeable HMA.

Numerous factors affect performance of unbonded overlays, and the above observations do not mean that the 25-mm (1-in) HMA is not a good design for a separator layer. Further research is needed to understand how the underlying pavement condition and the levels of preoverlay repair affect the requirements for separator layer and unbonded overlay design.

Table 4-3 shows condition ratings for the unbonded overlays, which are based on the overlay age and condition (distresses). However, the ratings by themselves do not indicate how the overlays are performing. For example, a 254-mm (10-in) overlay that is less than 5 years old and has carried less than 5 million ESALs is expected to be in good condition with virtually no distresses. Consequently, a "fair" rating for such a section is not an indication of acceptable performance. However, a 152-mm (6-in) overlay that has carried more than 10 million ESALs and is in "fair" condition shows excellent performance. Thus, the condition rating needs to be weighed against the expected performance to determine the merits of each design.

The unbonded overlays listed in table 4-3 was evaluated using a "relative performance rating," which rates the condition rating given in table 4-3 against the expected performance determined based on overlay thickness and traffic. The "relative performance rating" is determined as follows:

1. Convert the condition rating for each section to a numerical value using the system shown in figure 4-8. The numerical value of the condition rating is the middle value of the scale that corresponds to "Poor," "Fair," or "Good" rating. Thus the "Poor" rating corresponds to a numerical value of 0.167, "Fair" to 0.5, and "Good" to 0.833.

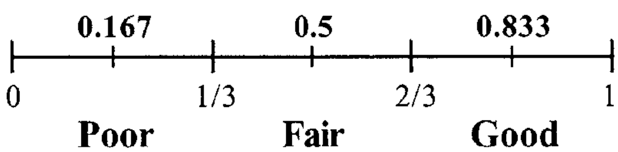

Figure 4-8. Numerical scale for condition rating.

2. Determine the numerical rating of the expected pavement condition based on remaining life, assuming that the pavement condition is roughly correlated to remaining life. The assumed design traffic shown in table 4-4 was used to estimate the remaining life in terms of the relative remaining traffic capacity. For example, table 4-4 shows that a 152-mm (6-in) JPCP overlay is expected to carry 10 million ESALs. The remaining life for that design is 0.5 after carrying 5 million ESALs.

Table 4-4. Assumed design traffic for estimating approximate remaining life.

Overlay Thickness	Design Traffic, million ESALs		
	JPCP	JRCP	CRCP
< 152 mm (6 in)	n/a	n/a	10
152 mm (6 in)	10	10	15
178 mm (7 in)	15	20	20
203-229 mm (8-9 in)	20	25	30
254 mm (10 in)	25	30	n/a
> 254 mm (10 in)	30	35	n/a

3. The difference between the numerical values of the rated condition and expected condition is the "relative performance rating."

A positive "relative performance rating" indicates performance that exceeds expectations, while a negative value indicates performance that falls short of expectations.

The results of this evaluation are shown in figures 4-9, 4-10, and 4-11 for the performance of unbonded JPCP, JRCP, and CRCP overlays, respectively. Since the numerical values for the condition ratings were taken as the middle value for each (figure 4-8), the results shown in figures 4-9 through 4-11 have an accuracy of ±0.17. Thus, relative performance ratings greater than about +0.2 can be taken as good performance and those less than about –0.2 can be taken as poor performance. The following conclusions are drawn from figures 4-9 through 4-11:

- For JPCP, the risk of poor performance is very high for overlays thinner than 152 mm (6 in). Figure 4-12 shows slab thickness vs. the probability of poor performance for unbonded JPCP overlays based on information shown in figure 4-9. The risk of poor performance is moderately high for 152- and 178-mm (6- and 7-in) overlays, but significantly lower for overlays 203 mm (8 in) or thicker.

- For JRCP, slab thicknesses in the range 152 to 279 mm (6 to 11 in) all gave good performance. Although figure 4-10 shows poor performance for two 254-mm (10 in) overlays, the main cause of poor performance is likely factors other than slab thickness. One of the two poorly performing sections is known to have been constructed without a separator layer.

- For CRCP, the risk of poor performance is very high for overlays 152 mm (6 in) or thinner.

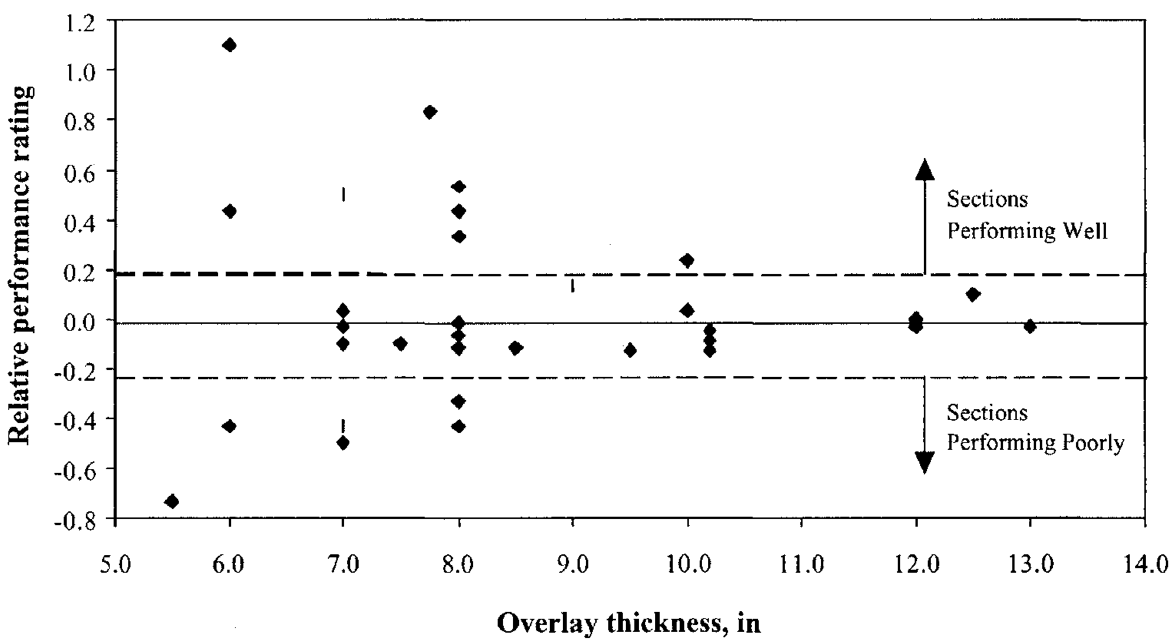

Figure 4-9. Effects of overlay thickness on unbonded JPCP overlay performance.

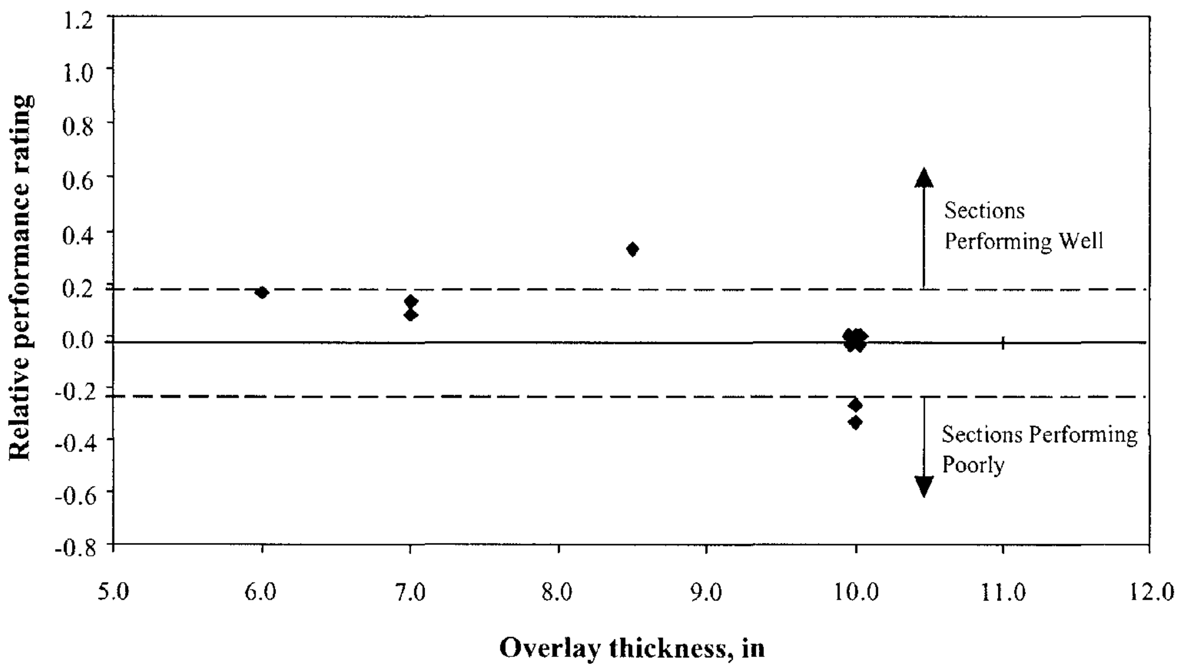

Figure 4-10. Effects of overlay thickness on unbonded JRCP overlay performance.

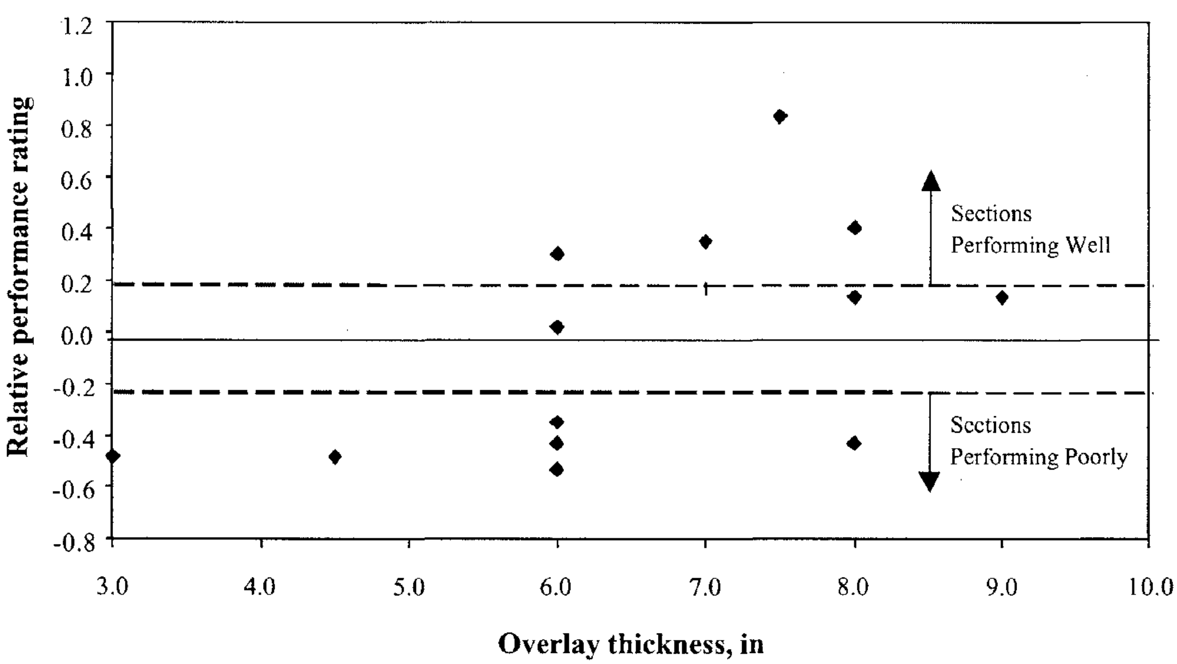

Figure 4-11. Effects of overlay thickness on unbonded CRCP overlay performance.

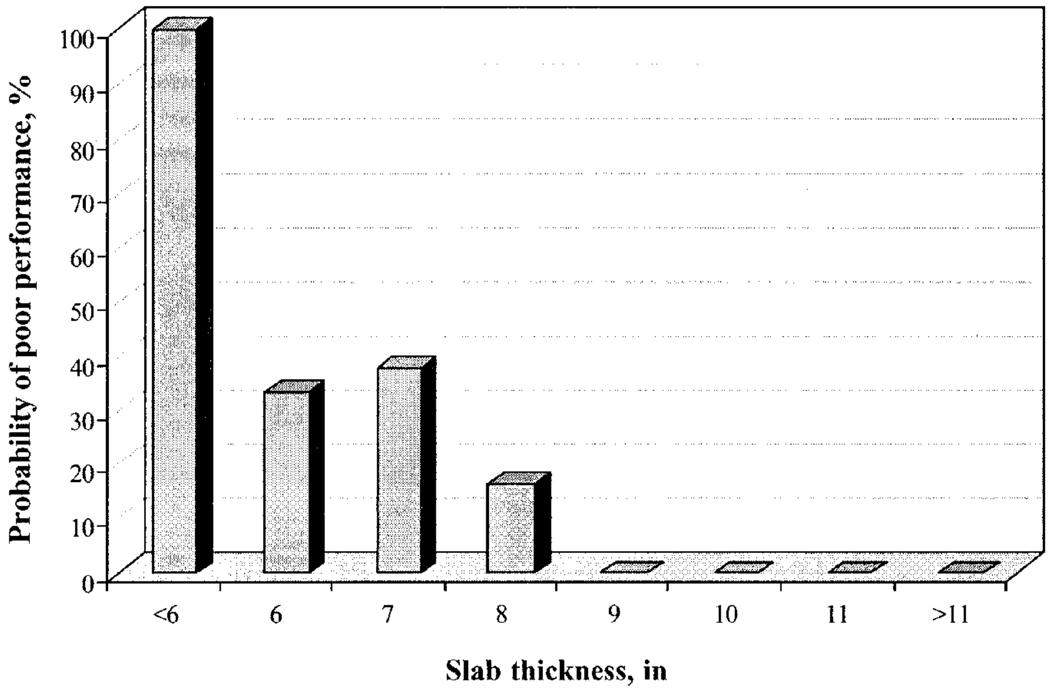

Figure 4-12. Slab thickness vs. probability of poor performance for unbonded JPCP overlays.

In general, figures 4-9 through 4-11 do not indicate gross errors in overlay design thicknesses; however, establishing lower limits on unbonded overlay thickness may be desirable to avoid high-risk designs. These figures show that JCP overlays 152 mm (6 in) or thicker and CRCP overlays 178 mm (7 in) or thicker are all workable, but improved design procedures are needed in order to improve overall design reliability. The poor performance of two 254-mm (10-in) JRCP and one 203-mm (8-in) CRCP overlays shows that not all design issues can be addressed through overlay thickness. Proper consideration of the condition of the underlying pavement, preoverlay repair needs, separator layer design, joint design, and reinforcement design are needed to obtain good performance.

4.4.4 *Specific Unbonded Overlay Projects*

As the above discussion indicates, unbonded PCC overlays have generally performed very well. Reviews of the performance of specific unbonded overlay projects have been summarized in previous NCHRP synthesis documents (Hutchinson 1982; McGhee 1994), and this section provides a review of the performance of several recent unbonded PCC overlay projects.

4.4.4.1 Highway 126 (Highbury Avenue), London, Ontario, Canada

As part of project evaluating pavement rehabilitation alternatives, the Ontario Ministry of Transportation in 1989 constructed an unbonded PCC overlay on a 5-km (3-mi) section of Highway 126 (Highbury Avenue) near London, approximately 200 km (124 mi) west of Toronto (Kazmierowski and Sturm 1994). The existing PCC pavement, built in 1963, was a 225-mm (8.8-in) JRCP with 21.3-m (70-ft) transverse joints that exhibited significant distress in terms of deteriorated transverse cracking, widespread joint spalling, and severe D-cracking (Kazmierowski and Sturm 1994).

The unbonded JPCP overlay was constructed 178-mm (7-in) thick with nondoweled, skewed, random transverse joints spaced between 3.7 and 5.8 m (12 and 19 ft) apart. No effort was made to match or avoid the transverse joints in the overlay with those in the underlying pavement. Prior to the placement of the unbonded overlay, spalled joints and cracks were patched with HMA and a 19-mm (0.75-in) sand-asphalt separation layer was placed (Kazmierowski and Sturm 1994).

An evaluation conducted in 1993 indicated that the unbonded PCC overlay was performing well, with only low-severity cracking observed in eleven driving lane slabs (Kazmierowski and Sturm 1994). Roughness, surface friction, and load transfer efficiencies were all noted to be acceptable. A 10-year evaluation conducted in 1999 again noted the existence of the longitudinal cracking, but the cracks were still confined to eleven slabs and were still of low severity (Kazmierowski and Bradbury 1999). This cracking was attributed to delayed sawcutting of the longitudinal joints in that area. No other distresses were observed in the 10-year evaluation, and ride quality and surface friction remained satisfactory (Kazmierowski and Bradbury 1999).

4.4.4.2 Indiana (I-69, Marion and I-65, Parr)

Indiana has constructed several unbonded overlays, including projects on I-69 near Marion and on I-65 near Parr. The I-69 project was constructed in 1986-1987. The existing JRCP exhibited severe D-cracking, and had previously received an HMA overlay. The entire HMA overlay was removed prior to the placement of a 25-mm (1-in) sand-asphalt interlayer and the 254-mm (10-in) JPCP unbonded overlay (Jiang 1998). The transverse joints in the unbonded overlay were doweled, skewed, and randomly spaced.

The performance of the I-69 project has been the subject of ongoing monitoring. An 8-year evaluation report concluded that the pavement was performing very well (Jiang 1998). Some minor joint spalling had occurred, and transverse cracking was observed on thirty of the slabs over the 9-km (5.6-mi)-long project; however all but five of those cracks were low severity (Jiang 1998). Pavement deflection, roughness, and friction tests indicated that the pavement had sufficient strength, good rideability, and adequate surface friction (Jiang 1998).

The I-65 project was built in 1994 over an existing 254-mm (10-in) JRCP with 12.2-m (40-ft) joint spacing (Gulen and Noureldin 2000). The unbonded JPCP overlay was constructed 305-mm (12-in) thick on top of a 30-mm (1.2-in) HMA separator layer, with doweled, skewed transverse joints spaced at intervals between 4 and 5 m (13 and 16.4 ft) (Gulen and Noureldin 2000). After 5 years of service, the unbonded PCC overlay was exhibiting no signs of distress and had superior ride quality and surface friction compared to adjacent HMA overlays that were constructed at the same time for comparison purposes (Gulen and Noureldin 2000).

4.4.4.3 Missouri (I-29, Rock Port)

As part of the FHWA's High Performance Concrete Pavement (HPCP) program, the Missouri Department of Transportation constructed an experimental unbonded PCC overlay project in 1998. Located in the southbound lanes of I-29, the project includes test sections of conventional unbonded overlays, steel fiber-reinforced unbonded overlays, and polyolefin fiber-reinforced unbonded overlays, all with varying thicknesses (MoDOT 2000; Smith 2001).

This project consists of eight test sections, each 762-m (2500-ft) long (MoDOT 2000; Smith 2001). Of the eight test sections, three sections incorporate steel fibers in the PCC mix, three sections incorporate polyolefin fibers in the mix, and two sections use a conventional PCC mixture. The design and layout of these test sections is illustrated in figure 4-13 (MoDOT 2000; Smith 2001).

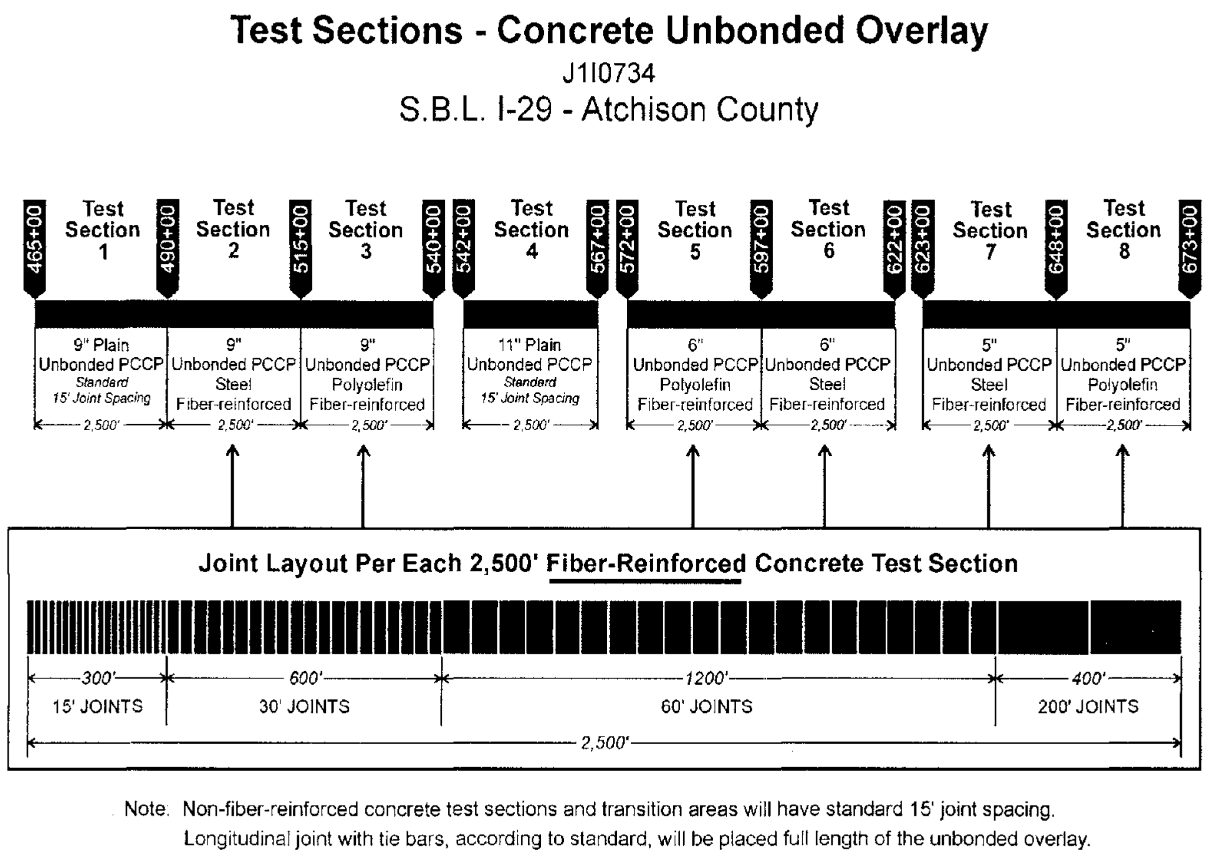

Figure 4-13. Layout of I-29 test sections (MoDOT 2000; Smith 2001).

The underlying pavement is a 229-mm (9-in) JRCP with 18.7-m (61.5-ft) transverse joint spacing (MoDOT 2000). A 25-mm (1-in) asphaltic interlayer treated with white curing compound was used to isolate the unbonded overlays from the underlying pavement (MoDOT 2000; Smith 2001).

After nearly 2 years of service, the performance of most of the sections has been good, although a few of the sections are performing poorly (MoDOT 2000; Smith 2001). In particular, the thin 127-mm (5-in) sections, both steel and polyolefin reinforced, are exhibiting a large amount of transverse cracking. In addition, the 152-mm (6-in) steel fiber-reinforced section is also showing significant transverse cracking (MoDOT 2000; Smith 2001). Because of the problems of the cracking and subsequent spalling, the thin 127-mm (5-in) sections have been removed and reconstructed. The Missouri Department of Transportation continues to monitor the performance of these test sections.

4.5 Summary

An unbonded overlay is a feasible rehabilitation alternative for PCC pavements in practically any condition, including those with MRD. The candidate pavements for unbonded overlays are typically those with extensive deterioration because unbonded overlays can be placed with minimal preoverlay repairs. In lieu of preoverlay repairs, slab fracturing is also an option for unbonded overlays.

Critical design factors for unbonded overlays include the condition of the existing pavement, preoverlay repairs, separator layer design, joint spacing, load transfer design, and reinforcement design. The following is a summary of unbonded overlay designs in use today:

- The most common pavement type for unbonded overlays is JPCP. CRCP overlays are also used, but JRCP is seldom used.

- The thickness of in-service unbonded overlays ranges from 76 mm (3 in) to 330 mm (13 in), but overlays thinner than 152 mm (6 in) or thicker than 254 mm (10 in) are not common.

- The most common type of separator layer is 25 mm (1 in) of HMA. This is currently the recommended design.

- The joint spacing of unbonded JPCP overlays is similar to that of new pavements. The maximum recommended joint spacing is 21 times the slab thickness for JPCP. In general, this recommendation is reasonable, but for thicker slabs (e.g., 241 mm [9.5 in] or thicker), this guideline produces excessive joint spacings.

- The faulting performance of unbonded overlays is significantly better than that of new PCC pavements; however, doweled joints are still recommended for heavy traffic (more than 1 million ESALs per year) to ensure good load transfer across transverse joints and prevent corner breaks.

The performance of unbonded overlays has been generally very good; however, questions remain on many aspects of the design of unbonded overlays. For example, the effects of both preoverlay repair and the separator layer design on the performance of unbonded overlays cannot be shown conclusively with currently available performance data, analysis tools, or performance models. Furthermore, in the past, any bonding between the pavement layers in unbonded overlays was thought to be a cause of poor performance, but current thinking is that a certain amount of bonding (or friction) between the separator layer and the overlay is essential for good performance (ERES 1999a).

Field performance data show that for unbonded JCP overlays, the risk of poor performance is high for overlays thinner than 152 mm (6 in). For CRCP, the risk of poor performance is very high for overlays thinner than 178 mm (7 in). While workable designs can be obtained by following available best-practice guidelines, further research is needed to optimize unbonded overlay design and improve design reliability.

CHAPTER 5. WHITETOPPING OVERLAYS

5.1 Introduction

Whitetopping is the construction of a new PCC pavement over an existing HMA pavement. It is considered an advantageous rehabilitation alternative for badly deteriorated HMA pavements, especially those that exhibit such distresses as rutting, shoving, and alligator cracking (ACPA 1998). Because a relatively thick PCC surface is capable of bridging a significant amount of deterioration in the underlying HMA pavement, minimal preoverlay repairs are needed for whitetopping. The whitetopping described in this chapter is contrasted with ultra-thin whitetopping, which is described in detail in Chapter 6.

Conventional whitetopping overlays are typically designed assuming an unbonded condition between the overlay and the underlying HMA pavement, although in actuality some bonding does occur. Thicknesses for whitetopping overlays are similar to those of new PCC pavements. As described in chapter 2, an emerging subset of conventional whitetopping is *thin* whitetopping, in which 102- to 203-mm-(4- and 8-in)-thick PCC slabs are placed on a milled HMA. The bonding between the layers is accounted for in the design process, and is expected to have a positive effect on the performance of the PCC overlay.

The focus of this chapter is on the current design and construction practices of *conventional* whitetopping overlays. Thin whitetopping overlays are not covered in this chapter, although their general design considerations and construction practices are similar to those for ultra-thin whitetopping overlays.

5.2 Design of Whitetopping Overlays

5.2.1 *General Design Considerations*

Structurally, conventional whitetopping overlays are similar to new PCC pavements, and they are designed as such. Nevertheless, the design of whitetopping overlays involves the consideration of factors that are common to pavement rehabilitation projects, including the following:

- Existing pavement condition. The condition of the existing pavement affects the thickness design of whitetopping overlays as well as their economic feasibility. In general, whitetopping overlays are most appropriate for HMA pavements that are extensively deteriorated. Pavements with excessive rutting, shoving, or alligator cracking are considered good candidates for conventional whitetopping overlays because these problems are not easily corrected with an HMA overlay.

- Overlay pavement type. The selection of the pavement type is usually a reflection of agency preference, but the condition of the existing pavement may be a factor that influences that decision. By far the most common type of whitetopping overlay is JPCP, but CRCP is also used. JRCP designs have also been used, but only a few have been built and most of those were prior to 1960 (McGhee 1994). Current practices are away from the use of JRCP designs.

- Preoverlay repair. Conventional whitetopping overlays do not require extensive preoverlay repairs, but the repair of certain types of distresses may be important to avoid localized failures. In general, the condition of the underlying pavement is more critical to the performance of CRCP whitetopping overlays. Additional information on preoverlay repairs is presented in section 5.2.3.

In terms of assessing the feasibility of whitetopping as a rehabilitation alternative, the following site factors should be considered:

- Traffic control.
- Shoulders.
- Overhead clearance.

In urban areas where traffic congestion is already a daily problem, management of detour traffic during construction can be a critical issue (TRB 1998). At some point, pavement reconstruction is unavoidable; however, while rehabilitation alternatives with less severe lane closure requirements are still viable (e.g., an HMA overlay) the lane closure requirement can be a key factor that determines the feasibility of whitetopping overlays. For projects in congested areas, the use of "fast-track" paving techniques may be appropriate to minimize lane closure times. Fast-track paving can be used to accomplish PCC pavement reconstruction and PCC overlays with weekend lane closures (ACPA 1994a).

The construction of a whitetopping overlay requires the construction of new shoulders because of the increase in the elevation of the mainline pavement. The elevation change also means that interchange ramps have to be adjusted, and guardrails may have to be raised, both of which affect the economic feasibility of whitetopping overlays.

Overhead clearances are another site factor that could affect the feasibility of whitetopping overlays. Because whitetopping overlays add significant thickness, short sections of reconstruction may be required at overhead structures (such as bridge overpasses) to ensure that adequate vertical clearance is provided. Raising the structure is another alternative, although a more costly one. Both of these alternatives add complexity, time, and cost to pavement rehabilitation projects, which makes whitetopping overlays less feasible on projects that involve many overhead structures.

5.2.2 *Pavement Evaluation*

The evaluation of the existing pavement is an essential part of any overlay design. Field evaluation typically consists of a visual distress survey, deflection testing using a falling weight deflectometer (FWD), and coring. For thickness design, the main information that must be obtained is the foundation support value. One problem in determining this value for whitetopping is that for existing HMA pavements the subgrade support value that is normally backcalculated is the resilient modulus (M_R), whereas the input needed for PCC pavement design is the modulus of subgrade reaction (k-value). Approximate correlations are available that can be used to estimate the k-value from backcalculated M_R values or from other soil properties (AASHTO 1998).

The AASHTO *Guide for Design of Pavement Structures* (1993) provides a procedure for determining the *composite* k-value that reflects the structural contribution of the existing HMA pavement from a backcalculated subgrade resilient modulus; the PCA design procedure uses a similar approach (PCA 1984). The composite k-value is meant to represent the effective k-value at the top of the existing HMA pavement and is determined considering the thickness and stiffness of the HMA layer. However, research has shown that this is not a realistic representation of the behavior of PCC pavements, and the concept of the composite k-value is inconsistent with the AASHTO design procedure (Darter, Hall, and Kuo 1994; Hall, Darter, and Kuo 1995). The more appropriate approach is to consider the contribution of the HMA layer in improving the bending stiffness of the PCC surface. This approach is adopted in the *Supplement to the AASHTO Guide for the Design of Pavement Structures* (1998) for the consideration of stabilized bases for new PCC pavement design.

As with any pavement rehabilitation project, variability in the existing pavement condition and in the subgrade are important considerations. The recommended practice is to break out any portion of the project with significantly different conditions as a separate section and design accordingly (AASHTO 1993).

5.2.3 *Preoverlay Repair*

The most critical issue in considering repairs to the existing HMA pavement is to ensure that uniform support is provided for the PCC surface. To obtain the desired performance, areas of subgrade or base failure must be removed and replaced with a stable material (McGhee 1994). In addition, the repair of badly deteriorated areas is recommended; these include severe rutting, shoving, and potholes (ACPA 1998). Guidelines for preoverlay repairs of conventional whitetopping are given in table 5-1.

Table 5-1. Guidelines for whitetopping preoverlay repair (ACPA 1998).

General Pavement Condition	Recommended Repair*
Rutting (< 50 mm [2 in])	None or milling**
Rutting (≥ 50 mm [2 in])	Milling or leveling
Shoving	Milling
Potholes	Fill with crushed stone cold mix or hot mix
Subgrade failure	Remove and replace or repair
Alligator cracking	None
Block cracking	None
Transverse cracking	None
Longitudinal cracking	None
Raveling	None
Bleeding	None

*Other factors to consider: adding edgedrains; costs of direct placement vs. milling or leveling .

** Consider increasing the joint sawing depth.

5.2.4 *Surface Preparation*

For conventional whitetopping, no special efforts are made to encourage bonding between the overlay and the underlying HMA surface; however, a surface preparation step may be required to address distortions in the existing HMA pavement surface or to correct surface profile. Three common methods of surface preparation are used for whitetopping:

- Direct placement. In this approach, the PCC overlay is placed directly on the existing HMA surface after sweeping. Any ruts in the existing pavement are filled with PCC, resulting in a thicker PCC pavement in the rutted areas. In Iowa, country roads are often resurfaced with thickened-edge whitetopping by directly placing PCC on existing HMA sections (McGhee 1994). The direct placement method is recommended when the rutting on the existing HMA pavement does not exceed about 25 mm (1 in). Consideration must also be given to the distorted surface profile in estimating the required material quantities. Additional survey of surface profiles may be required to allow this calculation; however, the additional cost of surveying is generally not as high as the cost of leveling the existing pavement surface (ACPA 1991b).

- Milling. In this approach, the existing HMA surface is milled to obtain a uniform surface. Milling can be used to remove all surface distortions and adjust cross slopes, with removal thicknesses typically ranging from 25 to 76 mm (1 to 3 in) (ACPA 1991b). This approach requires less surveying time and cost than direct placement, but results in additional costs for milling (and disposal, in some cases). Milling can be used in combination with direct placement, as only the parts of the project where the distortion is excessive need to be milled and direct placement can be used for the rest of the project (ACPA 1991b).

- Placement of leveling course. In this approach, a leveling course of HMA is used to produce a uniform surface for paving (ACPA 1991b). The leveling course typically consists of 25 to 50 mm (1 to 2 in) of HMA. Because this method involves the additional expense of HMA work, this option is usually not considered when the distortion depths exceed about 50 mm (2 in) (McGhee 1994). In such cases, milling is typically less expensive (ACPA 1991b).

A minimum HMA thickness of 50 mm (2 in) (after any milling) is recommended for conventional whitetopping overlays (Grogg et al. 2001).

5.2.5 *Thickness Design*

5.2.5.1 Design Procedures

Conventional whitetopping is designed as a new PCC pavement, treating the existing HMA pavement as a stabilized base. The required overlay thickness is determined using any established PCC design procedure for new pavements, such as the 1993 AASHTO methodology (AASHTO 1993), the Portland Cement Association procedure (PCA 1984), or the 1998 AASHTO Supplement procedure (AASHTO 1998). In all of these procedures, the overlay thickness is taken as the new PCC slab thickness required for the future traffic projections and for the given design conditions.

In addition to the design procedures described above, simple design charts have also been developed for selecting PCC thicknesses for whitetopping overlays (ACPA 1998). In these charts, the slab thickness is selected based on the number of trucks per day, the design PCC flexural strength, and the subgrade k-value. A minimum overlay thickness of 152 mm (6 in) is recommended for conventional whitetopping of primary and interstate roads (ACPA 1998).

5.2.5.2 Bonding Condition

In the design of conventional whitetopping overlays, the effects of any bonding between the PCC overlay and the underlying HMA layer is typically ignored, and the 1993 AASHTO and the 1984 PCA design procedures do not have any provisions for handling the effects of bonding. As described previously, both of these procedures account for the effects of the existing HMA pavement by using a composite k-value meant to represent the effective k-value at the top of the HMA pavement, although recent research has shown this to not a realistic representation.

The tendency for the PCC overlay to bond to the underlying HMA surface has long been recognized, and studies have shown that some degree of bonding nearly always occurs between the PCC overlay and the underlying HMA surface (Mack, Cole, and Moshen 1992; Grove, Harris, and Skinner 1993; Cole 1997; Tarr, Sheehan, and Ardani 2000). The findings from these studies gave rise to the development of UTW overlays and to the concept of thin whitetopping overlays, both of which depend on achieving and maintaining a good bond between the PCC overlay and the underlying HMA. Design procedures that take into consideration the bonding between the PCC overlay and the underlying HMA pavement have been developed for these types of overlays (Tarr, Sheehan, and Okamoto 1998; Tarr, Sheehan, and Ardani 2000; Wu and Sheehan 2002).

Incorporating the effects of bonding in the thickness design produces a thinner overlay; however, it is important to note that the long-term durability of the bond between the PCC overlay and the underlying HMA surface is not known. One study showed that PCC pavements constructed on a stabilized base nearly always exhibit bonded behavior (as indicated in FWD basin testing results), but long-term pavement performance (JPCP cracking) indicates unbonded behavior (Yu et al. 1998b). The debonding along the slab edges and corners (where the critical stresses occur in PCC pavements) could explain the apparently conflicting findings. A Colorado DOT study of the load response of instrumented slabs in the field also showed that whitetopping overlays can exhibit both bonded and unbonded behavior depending on the temperature and loading conditions, and that actual bonding between the pavement layers is not necessary for the pavement to show bonded response (Yu et al. 1998a). Therefore, further study is needed to ensure that the long-term benefits of partial bonding can be counted on to ensure the satisfactory performance of whitetopping overlays of reduced thickness.

The effects of layer interaction are included in the *Supplement to the AASHTO Guide for the Design of Pavement Structures* (AASHTO 1998). The interaction between the PCC surface and stabilized base is handled through the use of a friction factor. The guide provides a range of recommended friction factors to account for a range of bonding conditions for different types of bases. Therefore, at least in principle, partial bonding can be considered by assigning the appropriate friction factor. However, the design thickness is insensitive to the friction factors in the range that is recommended for HMA base. Further validation of the design procedure may be needed to ensure the reliability of the design thicknesses obtained using this procedure.

The main advantages of conventional whitetopping are that it is an effective rehabilitation alternative for badly deteriorated HMA pavement and it can be placed with minimal preoverlay repairs. Note that in table 5-1 no preoverlay is required for alligator cracking and other structural distresses. It is important to note, however, that a badly deteriorated HMA pavement does not have a significant structural value to a PCC overlay, even if it is bonded to the overlay. Thus, if partial bonding were considered in the thickness design, consideration must also be given to the condition of the existing HMA pavement and the appropriate repairs made to ensure that the condition assumed in the design is valid.

5.2.6 *Joint Spacing*

Transverse joint spacing directly affects the magnitude of critical stresses in JPCP whitetopping. Depending on the pavement design, the climate, season, and time of the day, curling stresses in JPCP can equal or exceed the load stresses. Mechanistically, joint spacing is an essential input to thickness design for JPCP, and the design thickness is only valid for the joint spacing assumed in the design analysis. However, joint spacing is not directly considered in either the 1993 AASHTO or the 1984 PCA design procedures, and is a source of variability for the performance of PCC pavements. Joint spacing is, however, directly considered as an input in the *Supplement to the AASHTO Guide for Design of Pavement Structures* (AASHTO 1998).

The maximum joint spacing recommended for conventional whitetopping constructed as JPCP is 21 times the slab thickness (ACPA 1998). For example, the recommended maximum joint spacing for a 203-mm (8-in) whitetopping is 4.2 m (14 ft). As a basic rule of thumb, this guideline is generally considered adequate, although it may be excessive for slabs greater than 241 mm (9.5 in). For reliable performance, it is recommended that the joint spacing be directly considered in the thickness design.

Current pavement design practices are away from the use of JRCP designs, and they are rarely constructed any more. This is true for both new construction and for overlay construction. If used, the recommended maximum joint spacing for JRCP is 9.1 m (30 ft) (FHWA 1990).

5.2.7 *Load Transfer Design*

Load transfer designs for conventional whitetopping are identical to those for new PCC pavements. In general, doweled joints are recommended for all pavements that will be subjected to significant truck traffic. Experience in Wyoming and Utah showed that whitetopping projects built without dowels develop significant faulting in a few years under interstate traffic (McGhee 1994). Load transfer recommendations are summarized in table 5-2.

Guidelines for selecting dowel diameter are often based on slab thickness, but traffic may be a more important consideration. Studies have shown that dowel diameter is very important to joint performance, and that adequately sized dowels must be provided to obtain good faulting performance (Snyder et al. 1989; Smith et al. 1997).

The recommended number and spacing of dowels is the same as those for new pavements. In general, uniform 305-mm (12-in) spacing is recommended, but nonuniform spacing has also been used successfully. In the nonuniform dowel spacing design, the dowels are concentrated in the wheelpaths (Darter et al. 1997). One recommended design for variable dowel bar spacing is illustrated in figure 5-1.

Table 5-2. Recommended load transfer designs (Smith and Hall 2001).

Design Feature	Recommendation			
	Design Catalog (Darter et al. 1997)		Industry (ACPA 1991a)	
Dowel Diameter	< 30 million ESALs	32 mm (1.25 in) bar	< 10 in slab	32 mm (1.25 in) bar
	30 – 60 million ESALs	38 mm (1.50 in) bar	≥ 10 in slab	38 mm (1.50 in) bar
	> 90 million ESALs	41 mm (1.625 in) bar		
Dowel Length	460 mm (18 in)			
Dowel Spacing	305-mm (12-in) center-to-center across the joint *Alternative*: Cluster dowels in wheelpath (see figure 5-1).			
Dowel Coating	Epoxy			

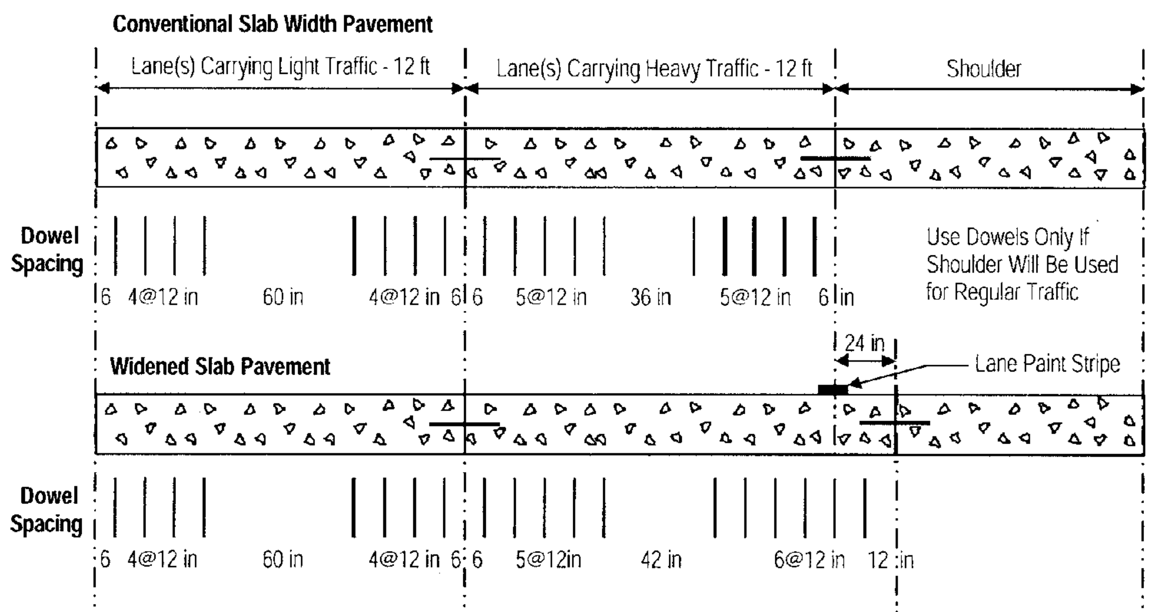

Figure 5-1. Recommended design for variable dowel bar spacing (Darter et al. 1997).

5.2.8 *Joint Orientation*

In general, perpendicular joints are recommended for whitetopping overlays. On new JPCP, studies have shown that skewed joints can be effective in reducing faulting in nondoweled pavements, but have no effect when used on properly doweled pavements (Yu et al. 1997; Khazanovich et al. 1998). Furthermore, JPCP designs with skewed joints constructed on a stiff base (especially cement-treated or lean concrete bases) are prone to corner breaks. Therefore, perpendicular joints are recommended for all doweled JCP. If skewed joints are used on nondoweled JPCP, it is recommended that the maximum skew be no more than 1:10.

5.2.9 *Reinforcement Design*

Reinforcement design for CRCP whitetopping is similar to that for new design. The recommended minimum steel content is 0.60 percent, and the use of deformed bars is strongly recommended (Darter et al. 1997). The depth of reinforcing steel has a significant effect on crack opening, and studies have shown that a higher steel placement leads to tighter cracks and better long-term performance (Dhamrait and Taylor 1979; Roman and Darter 1988). However, a minimum steel cover of 64 mm (2.5 in) is still recommended for corrosion protection.

JRCP designs are no longer routinely constructed. If used, deformed bars or deformed welded wire fabric (WWF) is recommended at a minimum steel content of 0.19 percent (Darter et al. 1997). A minimum steel cover of 64 mm (2.5 in) is again recommended.

5.2.10 *PCC Mix Design*

As described in chapter 2, the same PCC mixes used for new construction are generally used for whitetopping overlays. For projects in congested urban areas, however, extended lane closures due to pavement rehabilitation may be highly undesirable. For those projects, the use of fast-track paving may be considered to minimize traffic disruptions. Numerous fast-track PCC mixes are available that can provide the strength required for opening to traffic in 12 hours or less, and the techniques for fast-track paving are well established (ACPA 1994a; FHWA 1994; ACI 2001). Fast-track paving has been used successfully on a number of projects where traffic congestion or accessibility is a critical issue (ACPA 1994a; ACI 2001). However, little information is available on the long-term performance and durability of pavements constructed using high-early-strength PCC mixtures.

5.2.11 *Edge Support*

Edge support refers to design features provided to either reduce critical edge stresses or reduce the occurrences of edge-loading conditions that cause the highest stresses in PCC pavements. These features include tied PCC shoulders and widened slabs. Studies have shown that, in general, widened slabs are more effective than tied PCC shoulders in contributing to the performance of the pavement (Smith et al. 1995). The effectiveness of tied PCC shoulders depends on the effectiveness of the tie system, and shoulders constructed monolithically with the mainline pavement provide significantly better support than those paved separately. Even with an effective tie system, however, tied PCC shoulders have not significantly affected faulting performance, although they have been shown to be effective in reducing critical edge stresses (Smith et al. 1995; Yu et al. 1998a).

For conventional whitetopping, the widened slab design is preferable over tied PCC shoulders because widened slabs improve both cracking and faulting performance (Smith et al. 1995). One study showed that the structural benefit of widened slabs is roughly equivalent to about 25 mm (1 in) of additional slab thickness (Yu, Smith, and Darter 1995). However, because the placement of an overlay requires shoulder work, the use of PCC shoulders can be easily incorporated into the design. In some cases, there may be nonstructural reasons for providing a tied PCC shoulder on a whitetopping project.

5.2.12 *Drainage*

The effectiveness of subsurface drainage on the performance of whitetopping is not well known, but the effects may be expected to be similar to those on new PCC pavements. The possible benefits of properly designed, constructed, and maintained edgedrains on new PCC pavements include the following (Smith et al. 1995; ERES 1999b):

- Reduced pumping and faulting.
- Lower rate of crack deterioration on CRCP.
- Improved material performance (e.g., D-cracking, HMA stripping).

Unfortunately, no information is currently available to substantiate any of these benefits, so further research is needed to determine the effectiveness of edgedrains on the structural performance of whitetopping overlays. However, based on the evaluation of edge drains in new PCC pavements, it might be that edgedrains could improve material performance in whitetopping overlays (Smith et al. 1995; ERES 1999b).

5.2.13 *Job-Site Considerations*

Several job-site factors require special consideration, including bridge approaches, overhead clearances, and shoulders. At bridge underpasses, reconstruction of a short section may be necessary to satisfy the vertical clearance requirement. Reconstruction requires sections at both ends to provide a smooth transition between the overlay and the reconstructed section. The recommended taper length for the transition is 90 to 150 m (300 to 500 ft) (ACPA 1991b). A similar transition section is also needed at bridge approaches. An example transition is shown in figure 5-2.

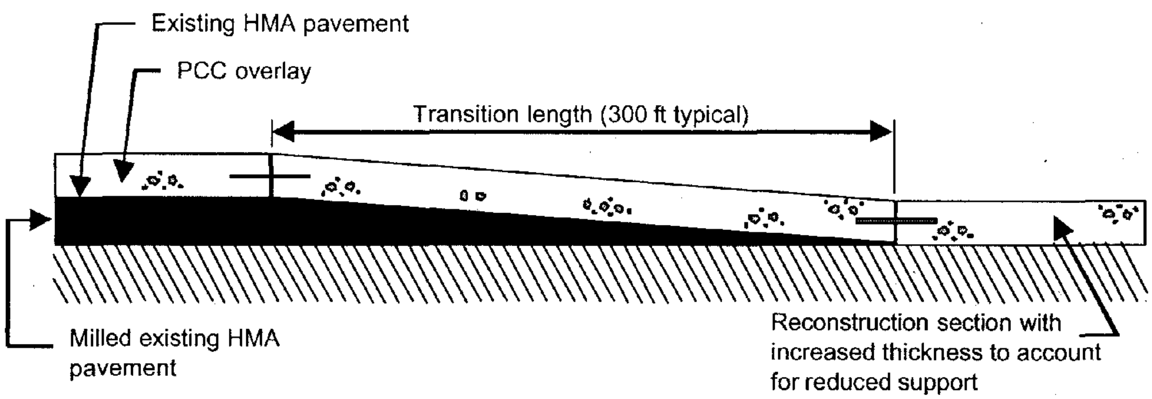

Figure 5-2. Example transition detail for bridge approaches and overpasses (ACPA 1991b).

The construction of whitetopping overlays requires shoulder work. Either PCC or HMA shoulders could be provided, with the use of tied PCC shoulders generally recommended for urban areas or in areas where the shoulder may at some point in the future be used as a travel lane. When used, it is important that the transverse joints in the PCC shoulder match those in the mainline pavement.

5.3 Construction of Whitetopping Overlays

The construction of whitetopping overlays involves limited preoverlay repairs of the existing HMA pavement and then the placement of the PCC overlay. The PCC placement for whitetopping is no different than that for new construction, as briefly described in the following sections.

5.3.1 *Preoverlay Repair*

As discussed in section 5.2.3, conventional whitetopping requires minimal preoverlay repairs. Recommended preoverlay repairs for conventional whitetopping include the following:

- Removal and replacement of areas of subgrade or base failure.
- Repair or removal of severe rutting, shoving, or other distortions.
- Repair of potholes.

The ACPA guidelines for preoverlay repairs for conventional whitetopping are given in table 5-1 (ACPA 1998). The techniques for preoverlay repairs are the same as the repair techniques for HMA pavements. Recommended practices for these techniques are summarized in several references by the American Concrete Pavement Association (ACPA 1991b; ACPA 1998) and in an National Highway Institute training course manual (Grogg et al. 2001).

5.3.2 *Surface Preparation*

As discussed in section 5.2.4, if surface distortions on the existing HMA pavement are excessive (say, greater than 25 mm [1 in]), either milling or the placement of a leveling course may be necessary. Milling and placement of a leveling course are a part of standard HMA paving techniques, and the procedures are no different for whitetopping overlays. The reference manual for the NHI training course *Hot-Mix Asphalt Construction* (NHI 1998) provides details of HMA construction techniques.

5.3.3 *PCC Placement and Finishing*

The procedures for placing and finishing PCC for whitetopping overlays are the same as those for new pavements. Standard practices apply, and the NHI training course on *Construction of Portland Cement Concrete Pavements* (ACPA 2000b) provides a good summary of recommended practices for PCC placement and finishing.

When the air temperature is greater than about 32 °C (90 °F), the surface temperature of the existing HMA pavement can become excessive (ACPA 1998). Placing PCC on a hot HMA surface can lead to cracking due to shrinkage, as well as excessive thermal restraint stresses resulting from the large temperature difference between the PCC at hardening and overnight low temperatures. When the HMA pavement surface becomes uncomfortable to touch with an open palm, water fogging or whitewashing is recommended to reduce the surface temperature (ACPA 1998). Although the use of water fogging has worked well in reducing the surface temperature, the use of whitewashing should be used cautiously as it can reduce the friction between the PCC overlay and the HMA, which is believed to be beneficial to the performance of the whitetopping overlay.

5.3.4 *Texturing*

Texturing of the finished PCC pavement surface is required to ensure adequate surface friction of the roadway. Initial texturing is often done with a burlap drag or turf drag, with the final texturing provided by tining. Tining provides macrotexture, which contributes to surface friction by tire deformation, and also channels surface water out from between the pavement and the tire. Tining should be conducted as soon as the sheen goes off of the PCC. Additional guidance on surface tining is found in reports by Kuemmel et al. (2000) and by ACPA (2000c).

Tining has traditionally been conducted transversely and at uniform intervals, but recent studies suggest that uniformly spaced transverse tining produces irritating pavement noise (Larson and Hibbs 1997; Kuemmel et al. 2000). Consequently, some agencies are experimenting with transverse tining that is randomly spaced and skewed to the centerline of the pavement, the pattern of which must be carefully designed and constructed in order to minimize discrete noise frequencies that are most objectionable to the human ear (Kuemmel et al. 2000). In addition, some agencies are investigating the use of longitudinal tining, which produces lower noise levels than either uniformly or randomly spaced transverse tining (Kuemmel et al. 2000). Current recommendations for tining are as follows:

- The depth of tining should be 3 to 5 mm (0.12 to 0.20 in), and the individual tines should be 3.0 mm (0.12 in) wide (Kuemmel et al. 2000; WisDOT 2001).

- When tining transversely, tines should be spaced randomly at a minimum spacing of 10 mm (0.4 in) and a maximum spacing of 57 mm (2.2 in) apart (WisDOT 2001). Either skewed or nonskewed transverse tining may be conducted, although skewed tining is quieter (WisDOT 2001). A recommended random tining pattern specifically developed to avoid repeating tine patterns over the typical passenger car's wheelbase is available in the Wisconsin DOT Construction and Materials Manual (WisDOT 2001).

- When tining longitudinally, the tining should be done parallel to the centerline of the pavement with tines uniformly spaced at 19 mm (0.75 in) intervals (ACPA 1999b; ACPA 2000c).

However, both AASHTO and FHWA recommend that friction and safety not be compromised to obtain slight, usually short-term, reductions in noise levels (Smith and Hall 2001).

5.3.5 *Curing*

The curing of whitetopping overlays is no different than for new PCC pavement construction. The most common practice is to spray liquid, white-pigmented, membrane curing compound, and this is the recommended method of curing for whitetopping overlays (ACPA 1998). Special attention to the temperature conditions during PCC placement may be warranted to avoid excessively high temperature gradients through the PCC during curing. Studies have shown that an excessive temperature gradient through PCC at the time of PCC hardening can cause locking-in of a significant amount of curling into PCC slabs, which can be deleterious to fatigue performance (Yu, Smith, and Darter 1995; Yu et al. 1998a; Byrum 2000).

5.3.6 *Joint Sawing and Sealing*

As with new PCC pavements, timely sawing is critical to avoid random cracking for whitetopping overlays. The same procedures and recommendations given for new pavements are applicable to whitetopping overlays. Joint sawing recommendations are given in numerous references, including Okamoto et al. (1994) and the NHI training course on PCC pavement construction (ACPA 2000b). For fast-track projects, ultra-light saws can be used on PCC pavements with strengths as low as 1.0 MPa (150 lbf/in^2) (Hoerner et al. 2001).

The sawcut depth is of particular concern for conventional whitetopping overlays because the distortions in the underlying HMA pavement can effectively increase the slab thickness. This is illustrated in figure 5-3. A minimum sawcut depth of one-third the PCC thickness is recommended (ACPA 1998). A deeper cut should be made where the overlay thickness varies more than 25 mm (1 in) over the nominal thickness. For green sawing, a shallower sawcut may be allowable (ACPA 1998).

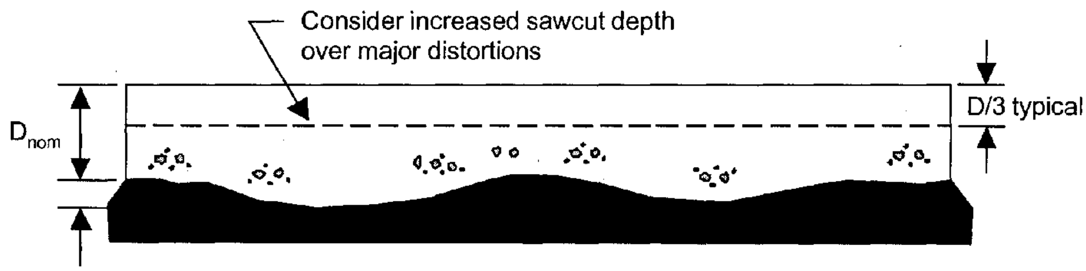

Figure 5-3. Consideration of rut depth in the HMA pavement in determining the appropriate sawcut depth (ACPA 1991b).

5.4 Performance of Whitetopping Overlays

5.4.1 *Factors Affecting Performance*

The performance of conventional whitetopping overlays is insensitive to the condition of the underlying HMA pavement and the overlays can be placed with minimal preoverlay repair. The same factors that affect the performance of new PCC pavements affect the performance of conventional whitetopping overlays, and these include the following:

- Condition of the existing pavement (type, severity, and extent of distress).
- Preoverlay repair (all areas of subgrade or base failures).
- Overlay design features (overlay thickness, joint spacing, load transfer design, reinforcement design, and drainage design).
- Traffic (axle weights and number).
- Climate (temperature and moisture conditions).
- Construction quality and curing.

An effective design procedure should account for the condition of the existing pavement and preoverlay repairs and result in a design that will provide the desired performance. For the most part, existing design procedures have been effective in producing workable design thicknesses for whitetopping. However, there is a possibility that conventional design practices may be overly conservative in some cases because the bond effects have been ignored.

5.4.2 *State Practice*

Whitetopping has been used by many highway agencies, including California, Iowa, Nebraska, Nevada, Texas, and Utah, and has become an increasingly popular technique for rehabilitating deteriorated HMA pavements (McGhee 1994). For example, of the 189 whitetopping projects listed in NCHRP Synthesis 204 (McGhee 1994), 125 of those were built since 1980. Between 1970 and 1980, whitetopping was used in numerous projects to upgrade the existing pavement to the interstate standards (Webb and Delatte 2000).

Whitetopping thicknesses typically range from 203 to 305 mm (8 to 12 in) when placed on primary interstate highways, and from 127 to 178 mm (5 to 7 in) when placed on secondary roads (ACPA 1998). JPCP designs are by far the most common type of whitetopping, although some states have constructed CRCP whitetopping overlays.

The effects of bond between the PCC overlay and the underlying HMA has been gaining more attention since the early 1990s in the design of whitetopping overlays. In 1991, the Iowa Department of Transportation (DOT) conducted a study to determine the bond contribution to whitetopping overlays (Grove, Harris, and Skinner 1993). Their report concludes that the existing HMA pavement does contribute significantly to the structural capacity of the PCC overlay, and that the bond effects should be considered in the thickness design, at least in cases where the traffic loads are not expected to cause stresses that exceed the bond strength. More recently, the Colorado DOT developed a mechanistic design procedure for thin (127 to 178 mm [5 to 7 in]) whitetopping that takes into consideration the effects of partial bonding between the pavement layers (Tarr, Sheehan, and Okamoto 1998; Tarr, Sheehan, and Ardani 2000). Additional research to validate that design procedure is currently underway (Wu and Sheehan 2002).

5.4.3 *Field Performance*

The field performance of whitetopping overlays has been generally very good. The majority of whitetopping overlays have provided good to excellent performance (Lokken 1981; Hutchinson 1982; McGhee 1994). The success of this design is often attributed to the uniform support and bond provided by the underlying HMA pavement (Hutchinson 1982; Grove, Harris, and Skinner 1993; McGhee 1994).

California has used whitetopping extensively since the 1960s, and has enjoyed considerable success with the treatment. For example, numerous highway pavement sections were whitetopped with 178- to 229-mm (7- to 9-in)-thick nondoweled JPCP in the 1960s and 1970s, and these were reported to be performing well after up to 20 years of service (Lokken 1981; Hutchinson 1982).

Iowa also has had outstanding performance from whitetopping overlays, many of which have been placed on their county highway system. For example, beginning in the late 1970s, three Iowa counties (Dallas, Boone, and Washington) began paving with whitetopping overlays, and since that time an average of 31 km (19 mi) of county pavement are whitetopped each year (ACPA 2000d). In most cases, the PCC overlays are placed directly on the existing HMA with little preparation other than sweeping (ACPA 2000d). In addition, the PCC overlays are constructed with 4.6-m (15-ft) joint spacings and a thickened-edge slab designs, in which the center of the pavement is constructed 127 or 152 mm (5 or 6 in) thick and the outer edges are constructed 152 or 178 mm (6 or 7 in) thick (ACPA 2000d). After 22 years of service, the oldest whitetopping overlays have required minimal maintenance and are performing well (ACPA 2000d).

Where the performance of whitetopping overlays has not been satisfactory, the problem can often be traced to a design flaw or inadequacy. For example, the Wyoming DOT reported that nondoweled whitetopping overlays developed significant faulting after a few years of Interstate highway traffic (McGhee 1994). Similar findings were made on nondoweled whitetopping overlays in Utah (McGhee 1994).

5.5 Summary

Conventional whitetopping is considered an advantageous rehabilitation alternative for badly deteriorated HMA pavements, including those that exhibit such distresses as rutting, shoving, and alligator cracking. Because the PCC surface is capable of bridging significant deterioration in the underlying HMA pavement, minimal preoverlay repairs are needed. Conventional whitetopping overlays are similar to new PCC pavements, and they are designed as such. Historically, the effects of any bonding between the overlay and the underlying HMA pavement have been ignored in the thickness design of whitetopping. However, since the early 1990s, the effects of bonding between the PCC surface and the underlying HMA pavement have been gaining more attention, and design procedures have been developed that takes into consideration the effects of bonding.

All recommended design and construction practices for new PCC pavement are directly applicable to conventional whitetopping. Perhaps the only significant difference is that the recommended sawcut depth for transverse joints must consider the rut depth on the underlying HMA surface. Construction of conventional whitetopping overlays does not involve any special equipment or construction techniques. Conventional whitetopping can be placed directly over the existing HMA pavement without any surface preparation (direct placement), but milling or the placement of leveling course may be needed to address excessive distortions (say, greater than 25 mm [1 in]) on the HMA surface.

Whitetopping is an increasingly popular rehabilitation technique for deteriorated HMA pavements, and the field performance of whitetopping overlays has been very good. However, as with new PCC pavements, careful consideration of other critical design elements (e.g., joint design) is required to ensure adequate performance.

CHAPTER 6. ULTRA-THIN WHITETOPPING

6.1 Introduction

As described in chapter 2, ultra-thin whitetopping (UTW) is the placement of a thin PCC pavement over an existing HMA pavement. The development of an effective bond between the PCC overlay and the existing HMA pavement is critical to the performance of this rehabilitation technique because the existing HMA pavement is being relied upon to carry part of the traffic load (Mack, Hawbaker, and Cole 1998). Factors differentiating UTW from conventional whitetopping include (ACPA 1998):

- The use of thin PCC surfacings (between 50 and 102 mm [2 and 4 in]).
- The need for extensive surface preparation to promote significant bonding between the PCC overlay and the HMA pavement.
- The use of short joint spacings (generally between 0.6 and 1.8 m [2 and 6 ft]).
- In many cases, the use of high-strength PCC mixtures to provide early opening times and the inclusion of synthetic fibers (commonly polypropylene or polyolefin) to help control plastic shrinkage cracking and enhance post-cracking behavior.

Although not described in detail here, it is important to recognize that *thin* whitetopping overlays are closely related to UTW because of their similarities in design, construction, and behavior. These are often placed on state highways and secondary routes with typical thicknesses ranging from 102 to 203 mm (4 to 8 in).

UTW overlays are intended for parking lots, residential streets, low volume roads, general aviation airports, and HMA intersections where rutting is a problem (ACPA 1998). UTW usually are applied where a substantial thickness of HMA exists (greater than 76 to 152 mm [3 to 6 in] after any surface preparation) because of the reliance on the HMA to carry a significant part of the load (ACPA 1998).

The project that served to give rise to the consideration of UTW overlays as a viable rehabilitation alternative was constructed in 1991 on a landfill access road in Louisville, Kentucky (Risser et al. 1993). This experimental project featured two different slab thicknesses and varying slab dimensions. The UTW pavements constructed in that project performed very well, carrying far more traffic than originally anticipated. The outstanding performance of that project created a widespread interest in the continued use and development of the technology, and since 1992 over 200 UTW projects have been constructed in at least 35 States (ACPA 2000a).

This chapter presents a summary of the current structural design and pavement construction practices for UTW overlays. This is followed by a review of the field performance of selected UTW overlay projects.

6.2 Design of UTW Overlays

6.2.1 *General Design Considerations*

When contemplating the use of UTW overlays, several general considerations regarding their applicability must first be evaluated. These considerations include:

- Detailed evaluation of existing pavement. A thorough examination of pavement deficiencies and the causes of deterioration must be made prior to selecting UTW as a treatment alternative. Generally speaking, ultra-thin PCC overlays are intended to be placed on existing HMA pavements that are exhibiting rutting, shoving, and other surface distresses; severely deteriorated HMA pavements with significant structural deterioration, inadequate base/subbase support, poor drainage conditions, or stripping of the HMA layers are not candidates for UTW overlays if these conditions are extensive throughout a project. Furthermore, a minimum HMA thickness (after milling) is required to provide the necessary support to the UTW overlay; ACPA (1998) suggests a minimum HMA thickness of 76 mm (3 in) whereas Silfwerbrand (1997) suggests a minimum HMA thickness of 152 mm (6 in) unless stiffer subgrades or lighter loads are present.

- Traffic evaluation. UTW overlays are intended for pavements subjected to lower traffic levels. Consequently, traffic studies should be performed to ensure that the truck traffic levels are such that a UTW overlay is still appropriate. The design life of the UTW overlay will vary depending on the traffic to which it is exposed.

- Surface preparation. The success of UTW overlays depends largely on achieving an effective bond between the new overlay and the existing HMA pavement. This is accomplished by milling the HMA surface to not only enhance the bond, but also to remove any surface distress or distortions.

- Lane closures. Certain locations, such as urban intersections, may present specific constraints on available lane closure times and the management of detoured traffic. In these cases, the use of fast-track paving may be considered for the UTW to minimize lane closure times. Additional information on fast-track paving is found in references by FHWA (1994), ACPA (1994a), and ACI (2001).

Because milling is performed prior to the placement of UTW overlays, vertical overhead clearances, the matching of adjacent shoulder and traffic lane elevations, and the maintenance of curb reveals are generally not a problem. Often the UTW can be placed as an inlay, using adjacent (unmilled) HMA as side formwork, thereby lowering costs and facilitating the construction of a smooth pavement.

6.2.2 *Pavement Evaluation*

As with any rehabilitation technique, the existing pavement must be evaluated as part of the design process. This is to ensure that the existing HMA pavement is structurally adequate to help carry the anticipated traffic loads. If the load-carrying capacity of the existing pavement is too low, there is a risk that the overlaid pavement will crack prematurely from traffic loading (Silfwerbrand 1997).

The structural adequacy of the existing HMA pavement can be assessed through an examination of the type, severity, and extent of the existing distresses, as well as through deflection testing using a falling weight deflectometer (FWD). FWD results can provide information on the stiffness of the HMA pavement and on the subgrade support conditions, which are needed in the design process. Furthermore, FWD results can help to reveal the variability of these properties over the length of the project, and can help identify localized weak areas requiring strengthening. However, as discussed in chapter 5, the backcalculation process on an existing HMA pavement produces a subgrade resilient modulus (MR) value, and this must be converted to a modulus of subgrade reaction (k-value) for whitetopping design. Approximate correlations are available that can be used to estimate the k-value from backcalculated MR values or from other soil properties (AASHTO 1998).

6.2.3 *Preoverlay Repair*

UTW overlays require some preoverlay repair of the existing HMA pavement in order to achieve the desired level of performance. Because the existing HMA pavement is being relied upon to carry part of the traffic loading, any deteriorated areas that would otherwise detract from its load-carrying capacity must be repaired (Grogg et al. 2001). This includes potholes, areas with moderate to severe alligator cracking, and other areas exhibiting structural inadequacies. In addition, milling of the existing HMA surface is required to remove rutting, restore profile, and provide a roughened surface to enhance bonding between the new PCC overlay and the existing HMA pavement (ACPA 1998). The thickness of the existing HMA pavement should also be verified with cores to help determine appropriate milling depths in order to ensure that sufficient HMA thickness remains after milling to contribute structurally to the UTW overlay.

6.2.4 *Thickness Design*

Because it is a relatively new technology, it has been only in the last several years that research activities have focused on the development of a formal design procedure for UTW overlays. The design of UTW is more complex than conventional overlay design because of the following factors (ACPA 1998; ACPA 1999a):

- The bond between the PCC and the HMA pavement.
- The short joint spacings associated with the design.
- The unique PCC mix designs (high strength and fiber reinforced) often used in UTW projects.

As previously described, the bond between the PCC and the HMA pavement is particularly critical to the performance of UTW overlays. A strong bond between these layers enables the pavement to act monolithically, which greatly reduces the magnitude of the stresses that develop in the pavement structure (ACPA 1998; ACPA 1999a).

An interim design procedure for estimating the load-carrying capacity and service life of UTW projects has been developed based on field performance results, instrumented slabs, and three-dimensional finite element modeling (Wu et al. 1997). The design procedure considers the development of critical stresses at both the corner and joint locations of a slab, and also incorporates the effects of temperature-induced stresses (Wu et al. 1997). Based on that procedure, two sets of simplified design charts were developed, each for a specific truck category (ACPA 1998; ACPA 1999a):

- Axle load category A (for low-truck-volume facilities) uses an assumed axle load distribution with a maximum single axle load of 80 kN (18,000 lb) and a maximum tandem axle load of 160 kN (36,000 lb).

- Axle load category B (for medium-truck-volume facilities) uses an assumed axle load distribution with a maximum single axle load of 116 kN (26,000 lb) and a maximum tandem axle load of 196 kN (44,000 lb).

The assumed axle load distributions for each axle load category are defined in a document by ACPA (1992).

One set of design tables (tables 6-1 and 6-2) has been developed for low-truck-volume facilities and one set of design tables (tables 6-3 and 6-4) for medium-truck-volume facilities. The output of the tables is the allowable number of trucks per lane (in thousands) for an assumed axle load distribution and a given UTW design, which is defined in terms of subgrade support, PCC flexural strength, HMA thickness, PCC thickness, and joint spacing. Thus, in using the tables, the adequacy of a selected design is evaluated, rather than the determination of a specific slab thickness design.

As an example in using the tables, assume that a UTW overlay is being contemplated for a city street subjected to truck traffic in axle load category B. The existing subgrade has a k-value of 54 kPa/mm (200 lbf/in^2/in) and the existing HMA pavement will be 102 mm (4 in) thick after milling. If the UTW will be 76 mm (3 in) thick with a flexural strength of (700 lbf/in^2) and a joint spacing of 0.9 m (3 ft), then it should be able to withstand 284,000 trucks in axle load category B (from table 6-4). If the street is subjected to 75 trucks per day, then the UTW is expected to provide a service life of 10.4 years (284,000/[75 x 365.25]).

6.2.5 *Joint Design*

Joint spacings for UTW overlays are very short, typically in the range of 0.6 to 1.8 m (2 to 6 ft). Such short joint spacings are needed in order to reduce the magnitude of curling and bending stresses. The ACPA recommends that the joint spacing of UTW projects should be limited to 12 to 15 times the slab thickness (ACPA 1998); that is:

$$S_{max} = (12 \text{ to } 15) \, x \, t \tag{6-1}$$

where:

S_{max} = Maximum joint spacing, mm (in).
t = Slab thickness, mm (in).

For example, a 76-mm (3-in) thick slab should have a joint spacing between 0.9 and 1.12 m (3 and 3.7 ft). In addition, it is recommended that the slabs for UTW overlays should generally be square (e.g., 0.6 m by 0.6 m [2 ft by 2 ft], 0.9 m by 0.9 m [3 ft by 3 ft], and so on).

Dowel bars, tiebars, and other embedded steel items are not used in UTW overlays. This is because the thin UTW slabs make their installation impractical, and effective load transfer at joints is expected to be provided by the aggregate interlock across the abutting joint faces and by the stiff support of the underlying HMA pavement.

Table 6-1. Allowable number of trucks (in thousands) per UTW traffic lane (Axle Load A, k = 100 lbf/in^2/in) (ACPA 1998; ACPA 1999a).

Average Flexural Strength, psi	h_2, Asphalt Thickness, in	h_1, UTW Thickness					
		2 in		3 in		4 in	
		Joint Spacing					
		3 ft	2 ft	4 ft	3 ft	6 ft	4 ft
700	3	6	60	40	104	137	303
700	4	56	156	125	234	294	546
700	5	169	375	314	507	593	996
700	6 or more	462	839	709	1070	1188	1862
800	3	24	77	90	158	273	458
800	4	81	183	201	311	478	748
800	5	213	422	428	625	858	1290
800	6 or more	507	935	880	1249	1572	2301

Table 6-2. Allowable number of trucks (in thousands) per UTW traffic lane (Axle Load A, k = 200 lbf/in^2/in) (ACPA 1998; ACPA 1999a).

Average Flexural Strength, psi	h_2, Asphalt Thickness, in	h_1, UTW Thickness					
		2 in		3 in		4 in	
		Joint Spacing					
		3 ft	2 ft	4 ft	3 ft	6 ft	4 ft
700	3	30	163	117	258	331	640
700	4	140	385	310	519	606	1045
700	5	384	842	664	1008	1099	1748
700	6 or more	765	1709	1092	1663	1591	2499
800	3	70	209	221	374	577	915
800	4	201	450	436	667	912	1396
800	5	480	938	840	1222	1487	2190
800	6 or more	882	1877	1334	2227	2039	3574

Table 6-3. Allowable number of trucks (in thousands) per UTW traffic lane (Axle Load B, k = 100 lbf/in^2/in) (ACPA 1998; ACPA 1999a).

Average Flexural Strength, psi	h_2, Asphalt Thickness, in	h_1, UTW Thickness					
		2 in		3 in		4 in	
		Joint Spacing					
		3 ft	2 ft	4 ft	3 ft	6 ft	4 ft
700	3	NR	29	1	38	8	136
700	4	15	90	43	122	98	299
700	5	90	228	168	301	273	593
700	6 or more	259	529	428	672	639	1181
800	3	2	43	31	84	106	268
800	4	39	110	98	188	238	471
800	5	129	263	252	406	501	845
800	6 or more	328	596	576	840	1007	1581

NR = Not Recommended

Table 6-4. Allowable number of trucks (in thousands) per UTW traffic lane (Axle Load B, k = 200 lbf/in^2/in) (ACPA 1998; ACPA 1999a).

Average Flexural Strength, psi	h_2, Asphalt Thickness, in	h_1, UTW Thickness					
		2 in		3 in		4 in	
		Joint Spacing					
		3 ft	2 ft	4 ft	3 ft	6 ft	4 ft
700	3	NR	75	6	102	56	298
700	4	55	216	110	284	230	578
700	5	197	497	331	620	553	1076
700	6 or more	511	1053	771	1221	1148	1915
800	3	9	111	79	197	266	551
800	4	101	261	221	398	502	875
800	5	277	622	495	778	922	1460
800	6 or more	639	1183	1002	1493	1583	2438

NR = Not Recommended

Thickened slabs are required for UTW overlays at transition areas between the PCC overlay and an adjacent HMA pavement (ACPA 1998). This is because impact loadings from traffic may induce high stresses in the UTW and cause cracking. A suggested transition detail is shown in figure 6-1 (ACPA 1998).

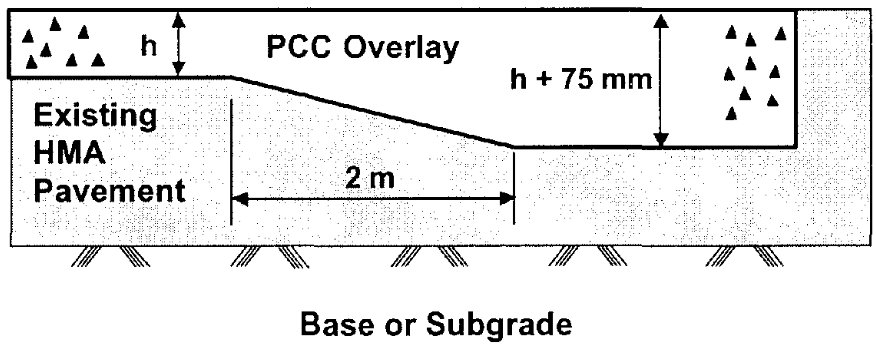

Figure 6-1. Suggested transition from UTW to adjacent HMA pavement (ACPA 1998).

6.2.6 *PCC Mix Design*

PCC mixtures used in UTW overlays are often high-strength mixtures and generally contain fibers (ACPA 1998). Depending upon the lane closure requirements of the particular UTW project, fast-track paving mixtures may also be employed in order to achieve compressive strengths over 20.7 MPa (3000 lbf/in^2) within 24 h (Mack, Hawbaker, and Cole 1998). Such mixtures often employ a Type III cement, higher cement contents, a low w/c ratio, and an accelerator in order to achieve the required strength for the specified opening time.

Table 6-5 summarizes some typical PCC mix designs that have been used on recent UTW overlay projects. As seen in this table, common characteristics of many of these mixtures is a high cementitious material (cement and fly ash) content, low water-to-cementitious material ratio, smaller top size aggregate (typically 19 mm [0.75 in]), and the use of synthetic fibers. As described in chapter 2, the introduction of the fibers is expected to improve the toughness and post-cracking behavior of the PCC, as well as to help control plastic shrinkage cracking. Polypropylene and polyolefin fibers are the two most commonly used synthetic fibers in UTW overlays.

As with any PCC mix design, PCC mixtures for UTW overlays should be carefully designed and proportioned. The production of several trial mixtures using project materials is strongly recommended in order to ensure that the specified strength, durability, and workability requirements of the paving project are met; this is particularly important when fibers are incorporated into the PCC mixtures. More detailed information on PCC mix design procedures can be found in documents by the American Concrete Institute (ACI 1991) and the Portland Cement Association (PCA 1992).

Table 6-5. Sample PCC mix designs for UTW projects.

Mix Component	Woodland and North First Streets, Nashville, TN (1992)	Rt. 21, Belle Plaine, IA (1994)	119th Street, Leawood, KS (1995)	Ramp D, U.S. 22/I-83 Interchange, Harrisburg, PA (1995)	U.S. 169, Elk River, MN Mix 1 (1997)	U.S. 169, Elk River, MN Mix 2 (1997)	I-10 Weigh Station, Ellaville, FL (1997)
Cement, lb/yd^3	800	573	610	520	450	550	610
Fly ash, lb/yd^3	—	—	—	92	120 (Class C)	100 (Class C)	—
Coarse Agg, lb/yd^3	1710	1661	1693	1835	1552 ($^3/_4$ minus) 277 ($^3/_8$ minus)	1552 ($^3/_4$ minus) 277 ($^3/_8$ minus)	1658
Fine Agg, lb/yd^3	1098	1363	1319	1126	1287	1287	1334
Water, lb/yd^3	280	244	224	266	245	240	239
w/c (including fly ash)	0.35	0.43	0.37	0.43	0.43	0.37	0.39
Air Content	5%	6%	6.5%	5–8%	6%	6%	4%
Fiber Type(s)/ Content	Polypropylene 3 lb/yd^3	None/ Polypropylene 3 lb/yd^3	None/ Polypropylene 3 lb/yd^3	Polypropylene 3 lb/yd^3	Polypropylene 3 lb/yd^3	Polyolefin 25 lb/yd^3	Polypropylene 3 lb/yd^3
Other Admixtures	High range water reducer	—	—	Water reducer 3.7 oz/yd^3	Water reducer	Water reducer	Water reducer
Strength Test Results, lbf/in^2	Compressive: 4795 at 1 d 8033 at 28 d Flexural: 615 at 1 d 1159 at 28 d	N/A	N/A	Compressive: 3440 at 7 d 4514 at 28 d	Compressive: 4900 at 14 d 5650 at 28 d Flexural: 590 at 28 d	Compressive: 4400 at 14 d 5300 at 28 d Flexural: 570 at 28 d	Compressive: 2755 at 1 d 6815 at 28 d Flexural: 870 at 28 d
Reference	Speakman and Scott 1998	Cable, Hart, and Ciha 1999	Mack, Hawbaker, and Cole 1998	Cumberledge, King, and Hawk 1996	Vandenbossche and Rettner 1998	Vandenbossche and Rettner 1998	Armaghani and Tu 1999

6.2.7 *Drainage*

As part of the pavement evaluation process, it is always advisable to conduct a thorough drainage survey to identify moisture-related distresses and develop solutions that address these distresses. If poor drainage conditions are contributing to the deterioration of the existing pavement, adding an overlay will not correct the problem and deterioration of the pavement system will continue.

The benefits of including edge drains as part of an UTW overlay project have not been documented. In urban areas, UTW overlays often incorporate drainage inlets that are tied to storm drain systems; in rural areas, regrading and re-establishment of the ditches may be required in order to facilitate drainage.

6.3 Construction of UTW Overlays

This section describes the construction of UTW overlays. The construction process includes preoverlay repairs, surface preparation, PCC placement and finishing, curing, and sawing and sealing. Although conventional PCC paving practices are employed for UTW overlays, some specific construction considerations are particularly critical to the performance of the design and will be highlighted in this section. A construction specification guideline for UTW overlays is available from ACPA (1999b); information on conventional PCC pavement construction is presented in an NHI training course (ACPA 2000b).

6.3.1 *Preoverlay Repair*

As described in section 6.2.3, some preoverlay repair of the existing HMA pavement is required in order to obtain the desired level of performance. This is because the existing HMA pavement will be carrying part of the traffic loading, so any structural deficiencies in the existing HMA pavement must be corrected prior to the placement of the UTW overlay. Among the types of preoverlay repair activities typically required for UTW overlays are (McGhee 1994; ACPA 1998):

- Localized repair of failed areas caused by loss of base or subgrade support.
- Filling of medium- and high-severity potholes.
- Localized repair of medium to severe alligator cracking. If significant alligator cracking exists throughout the project, this suggests a structural inadequacy and the pavement may not be a suitable candidate for UTW.

These activities should follow conventional HMA patching practices. In the case of patching of alligator cracking, it is important that the entire distressed area be removed and replaced through the entire thickness of the HMA pavement.

At least one highway agency has experimented with the use of cold in-place recycling (CIR) of the existing HMA pavement prior to the placement of the UTW overlay. On part of the Route 21 project near Belle Plaine, the Iowa DOT used a continuous recycling train to first mill the existing HMA pavement to a depth of 95 mm (3.75 in), then resize the removed material and mix it with an emulsion, and finally place the rejuvenated material back on the pavement surface (Cable, Hart, and Ciha 1999).

6.3.2 *Surface Preparation*

Milling of the existing HMA pavement is critical to the performance of the UTW overlay. Not only does it remove rutting and restore the surface profile, it also provides a roughened surface to enhance the bonding between the new PCC overlay and the existing HMA (ACPA 1998). This will increase the load-carrying capacity of the pavement system by enabling it to behave as a monolithic structure.

Cold milling should be conducted after the existing HMA pavement has been patched. Cold milling machines use carbide-tipped bits attached to a revolving drum to cut away the existing HMA pavement. A variety of milling machines are available for HMA removal, ranging from small equipment with narrow, 0.3-m (1-ft) wide drums to larger equipment with full-lane width (3.6-m [12-ft]) wide drums. Although some equipment can remove HMA depths up to 305 mm (12 in) in a single pass, common HMA removal depths in preparation for UTW overlays are 25 to 76 mm (1 to 3 in). The amount of HMA removal for a particular project will depend on the type and severity of distress (especially the depth of rutting or other surface distortions) and the thickness of the HMA pavement. After milling, an absolute minimum HMA thickness of 76 mm (3 in) is required, although some suggest a minimum HMA thickness of 152 mm (6 in) (Silfwerbrand 1997).

After the pavement has been milled, the pavement surface must be cleaned to help ensure bonding between the existing HMA and the new PCC overlay. This may be accomplished by air blasting or power brooming, but occasionally water blasting or sand blasting may be required to remove any slurry or residue from the milling. If waterblasting or washing operations are used, the surface must be allowed to dry before the placement of the PCC overlay (ACPA 1998).

6.3.3 *PCC Placement and Finishing*

Once the surface of the existing HMA pavement has been prepared, the PCC overlay is placed. Paving is accomplished using either fixed-form or slipform construction, the selection of which will depend on the size of the project and any geometric constraints. In either case, conventional PCC paving practices and procedures are followed. Primary activities in this part of the operation include spreading, consolidation, screeding, and float finishing.

6.3.4 *Texturing*

Texturing of the finished PCC pavement surface is required on all areas that will be exposed to traffic. For roadways designed for vehicle speeds less than 80 km/hr (50 mi/hr), texturing the surface with a burlap drag, turf drag, or broom should be adequate, provided the corrugations produced are about 1.5 mm (0.06 in) deep (ACPA 1999b).

For roadways designed for vehicle speeds greater than 80 km/hr (50 mi/hr), tining of the PCC pavement surface is required (ACPA 1999b). This provides macrotexture, which contributes to surface friction by tire deformation, and also channels surface water out from between the pavement and the tire. Tining should be conducted as soon as the sheen goes off of the PCC. Additional guidance on the orientation and spacing of surface tining is found in reports by Kuemmel et al. (2000) and by ACPA (ACPA 1999b; ACPA 2000c).

Tining has traditionally been conducted transversely and at uniform intervals, but recent studies suggest that uniformly spaced transverse tining produces irritating pavement noise (Larson and Hibbs 1997; Kuemmel et al. 2000). Consequently, some agencies are experimenting with transverse tining that is randomly spaced and skewed to the centerline of the pavement, the pattern of which must be carefully designed and constructed in order to minimize discrete noise frequencies that are most objectionable to the human ear (Kuemmel et al. 2000). In addition, some agencies are investigating the use of longitudinal tining, which produces lower noise levels than either uniformly or randomly spaced transverse tining (Kuemmel et al. 2000). Current recommendations for tining are as follows:

- The depth of tining should be 3 to 5 mm (0.12 to 0.20 in), and the individual tines should be 3.0 mm (0.12 in) wide (Kuemmel et al. 2000; WisDOT 2001).

- When tining transversely, tines should be spaced randomly at a minimum spacing of 10 mm (0.4 in) and a maximum spacing of 57 mm (2.2 in) apart (WisDOT 2001). Either skewed or nonskewed transverse tining may be conducted, although skewed tining is quieter (WisDOT 2001). A recommended random tining pattern specifically developed to avoid repeating tine patterns over the typical passenger car's wheelbase is available in the Wisconsin DOT Construction and Materials Manual (WisDOT 2001).

- When tining longitudinally, the tining should be done parallel to the centerline of the pavement with tines uniformly spaced at 19 mm (0.75 in) intervals (ACPA 1999b; ACPA 2000c).

However, both AASHTO and FHWA recommend that friction and safety not be compromised to obtain slight, usually short-term, reductions in noise levels (Smith and Hall 2001).

6.3.5 *Curing*

Effective curing of the new UTW overlay promotes continued cement hydration and strength gain by controlling the rate of moisture loss in the PCC slabs. Although curing is important to all PCC pavements, it is even more critical to UTW overlays because their high surface-area-to-volume ratio make them more susceptible to rapid moisture loss.

Curing is most often accomplished through the application of a curing compound immediately after the final texturing of the PCC surface. It is recommended that the curing compound be placed at twice the normal rate in order to reduce moisture loss, or at a maximum application rate of 2.5 m^2/l (100 ft^2/gal) (ACPA 1999b). All exposed PCC surfaces, both vertical and horizontal, should be coated with the curing compound.

6.3.6 *Joint Sawing and Sealing*

Timely joint sawing is required to establish the contraction joints in the PCC pavement and prevent random cracking. Because of the great amount of joint sawing required on UTW overlays, it is recommended that joint sawing commence as soon as the PCC has developed sufficient strength such that the joints can be cut without significant raveling or chipping. This will typically be within about 3 to 6 hours after PCC placement. The contractor must ensure that there are sufficient sawcutting crews available for the work and that all crews are familiar with the prescribed joint sawing patterns. Because of the need to get on the pavement as soon as possible, the use of lightweight "early-entry" saws are particularly advantageous for UTW overlay construction.

Criteria for sawcut depths on UTW overlays have not been established, but a minimum 25-mm (1-in) deep cut appears to perform satisfactorily for both transverse and longitudinal joints (ACPA 1998). The joints are typically sawed to a width of 3 mm (0.12 in). Generally the joints in UTW projects are not sealed, although a few agencies have constructed experimental UTW sections comparing sealed and nonsealed joints.

6.4 Performance of UTW Overlays

This section describes some of the more notable UTW overlays that have been constructed since their inception in the early 1990s. Because this is a new and evolving technology, very little long-term performance data are available for UTW overlays. However, the available data suggest that UTW overlays are a viable pavement rehabilitation alternative for low-volume roadways.

6.4.1 *Kentucky Landfill Project*

Although not the first of the modern UTW projects, this project located on a landfill access road is perhaps the most notable because performance results clearly showed that UTW overlay technology was a viable pavement rehabilitation alternative. Constructed in 1991 on a deteriorated HMA pavement, two 84-m (275-ft) long sections were included: one 50-mm (2-in) thick section and one 89-mm (3.5-in) thick section. Both sections used a joint spacing of 1.8 m by 1.8 m (6 ft by 6 ft), with a small subsection on the thinner overlay using 0.6 m by 0.6 m (2 ft by 2 ft) slabs (Risser et al. 1993).

The existing HMA pavement was milled prior to PCC placement to remove surface distress and rutting. A fast-track, high-strength PCC mixture was used so that the roadway could be opened to traffic within 41 hours. Polypropylene fibers were added to the mixture to provide toughness and post-crack integrity.

Over the evaluation period, the landfill access road carried over 600 trucks per day, 5½ days per week, which is much greater than the volumes and loads to which a conventional low-volume roadway would be exposed. All trucks entering the landfill were weighed, which allowed the precise determination of the loads applied to the pavement. After 1 year of service (and approximately 585,000 ESAL applications), both sections were still in excellent serviceable condition (Risser et al. 1993). Several panels had to be replaced after about 400,000 ESAL applications, but these occurred at a test pit location and in an area of excessive milling. The most remarkable performance was shown by the 50-mm (2-in) slabs with short 0.6 m by 0.6 m (2 ft by 2 ft) joint spacing, which did not exhibit any visible signs of distress after a year of service (Risser et al. 1993).

6.4.2 *Iowa Route 21, Belle Plaine*

The Iowa DOT constructed a major experimental project evaluating thin and ultra-thin whitetopping overlays in 1994 (Cable, Grove, and Heyer 1997; Cable, Hart, and Ciha 1999). This project is located on Iowa Route 21 on an 11.6-km (7.2-mi) stretch of pavement near Belle Plaine, and represents one of the first uses of the technology in a highway application. A total of 41 test sections were established, ranging in length from 61 to 823 m (200 to 2700 ft). Variables evaluated in the study include (Cable, Grove, and Heyer 1997; Cable, Hart, and Ciha 1999):

- HMA surface preparation: milled and patched, patched only, and cold-in-place recycled (CIR).

- PCC slab thicknesses: 50 mm (2 in), 102 mm (4 in), and 152 mm (6 in).

- Joint spacings:
  - 0.6 m by 0.6 m (2 ft by 2 ft) (on the 50-mm [2-in] and 102-mm [4-in] sections)
  - 1.2 m by 1.2 m (4 ft by 4 ft) (on the 50-mm [2-in] and 102-mm [4-in] sections)
  - 1.8 by 1.8 m (6 ft by 6 ft) (on the 102-mm [4-in] and 152-mm [6-in] sections)
  - 3.7 by 3.7 m (12 ft by 12 ft) (on the 152-mm [6-in] sections).

- Synthetic fibers: fibrillated polypropylene, monofilament polypropylene, and none.

- Joint sealant: hot-poured and none.

The existing HMA pavement was 76 mm (3 in) thick, not including a 19 mm (0.75 in) bituminous chip seal immediately beneath the HMA layer. Where the HMA pavement was milled, a layer thickness of 6 mm (0.25 in) was removed, leaving a nominal bituminous material thickness (HMA and chip seal) of 89 mm (3.5 in).

Performance monitoring of these experimental pavement sections is ongoing. Performance observations after 5 years of service include the following (Cable, Hart, and Ciha 2001):

- Milling of the existing HMA pavement provided the highest shear strength between the PCC and the HMA pavement. It also provided excellent control of longitudinal and transverse profiles, as well as overlay thicknesses.

- The 102- and 152-mm (4- and 6-in) thick UTW sections performed well over the 5-year evaluation period.

- Some of the 50-mm (2-in) UTW sections exhibited longitudinal and corner cracking, which often led to debonding of slab corners. Fibers were found to be beneficial to the performance of these sections.

- The 50-mm (2-in) UTW sections with 0.6 m by 0.6 m (2 ft by 2 ft) joint spacing had less cracking than the 50-mm (2-in) UTW sections with 1.2 m by 1.2 m (4 ft by 4 ft) joint spacing. However, the use of 0.6 m by 0.6 m (2 ft by 2 ft) joint spacing for the 50-mm (2-in) UTW sections introduced a longitudinal joint in the wheelpath that is believed to have been the source of cracking.

- The narrow, unsealed joints performed very well, with no signs of significant raveling or joint spalling.

- Deflection testing over the evaluation period indicated that traffic loadings and environmental conditions did contribute to increasing deflections over time, suggesting a reduction in the composite action of the pavement cross section.

Based on the results of the study, the researchers suggest the following design characteristics for good performing UTW overlays (Cable, Hart, and Ciha 2001):

- Surface preparation: Milling of the existing HMA pavement, leaving a minimum HMA thickness of 76 to 102 mm (3 to 4 in).

- PCC overlay thickness: UTW thicknesses between 76 and 102 mm (3 and 4 in), with fibers added to 76-mm (3-in) thick overlays.

- Joint design: Joint spacings of 1.2 to 1.8 m (4 to 6 ft) and narrow joints without joint sealant.

6.4.3 *Ramp D, Interchange of U.S. Route 22 and I-83, Harrisburg, PA*

Although not a traditional UTW design, this project evaluated an UTW overlay of an existing HMA/PCC composite design. Constructed by the Pennsylvania Department of Transportation in 1995, this project is located on ramp D at the interchange of U.S. Route 22 and I-83 in Harrisburg, PA (King 1997). The underlying pavement is a 254-mm (10-in) JRCP that had been overlaid with 83 mm (3.25 in) of HMA in 1992. Initially, less than 25 mm (1 in) of the HMA surface was removed, but the remaining HMA material was so loose and debonded that the decision was made to remove the entire HMA layer and replace it with a 50-mm (2-in) bituminous base course (King 1997). Although this was less than the minimum HMA thickness commonly recommended, it was deemed appropriate given the constraints on the final grade (King 1997). The UTW was placed 89-mm (3.5-in) thick with slabs approximately 1 m by 0.9 m (3.25 ft by 3 ft) (King 1997).

After 2 years of service, the UTW is performing quite well (King 1997). There is some minor spalling at a few of the joints (caused by sawing when the PCC was too green) and some transverse cracks that appeared prior to opening to traffic (caused by a subsequent delay in joint sawing), but overall no other significant signs of distress or deterioration were noted (King 1997). Penn DOT is continuing monitoring of these pavement sections.

6.4.4 *Route 29, Charlottesville, VA*

In 1995, the Virginia Department of Transportation constructed three PCC pavement overlays as part of the Intermodal Surface Transportation Efficiency Act (ISTEA) legislation passed in 1991. One of the projects is an UTW overlay constructed on a rutted HMA pavement on Route 29 near Charlottesville (Sprinkel and Ozyildirim 2000). Experimental design features include slab thickness (50, 76, and 102 mm [2, 3, and 4 in]) and fibers (polyolefin, monofilament polypropylene, and hooked-end steel at two dosage rates) (Sprinkel and Ozyildirim 2000).

After 4 years of performance, extensive cracking was observed on the 50-mm (2-in) UTW sections, consisting primarily of corner cracking (Sprinkel and Ozyildirim 2000). Virtually no cracking was observed on the 76- and 102-mm (3- and 4-in) sections. On the thin 50-mm (2-in) sections, it was further observed that the polyolefin fibers were more effective at holding the cracks together than either the polypropylene or the steel fibers at the lower (30 kg/m^3 [1.9 lb/ft^3]) dosage rate (Sprinkel and Ozyildirim 2000). Based on the results observed to date, it is concluded that UTW overlays are not cost competitive when compared to a conventional HMA overlays, but their use could be justified at selected locations where rutting is a chronic problem (Sprinkel and Ozyildirim 2000).

6.4.5 *119th Street, Leawood, KS*

In 1995, the Kansas Department of Transportation constructed a UTW project on 119th Street in Leawood, a suburb of Kansas City (Dumitru, Hossain, and Wojakowski 2002). The UTW was constructed 50-mm (2-in) thick and incorporated the following design variables into six test sections (Dumitru, Hossain, and Wojakowski 2002):

- Panel size: 0.9 m by 0.9 m (3 ft by 3 ft) and 1.2 m by 1.2 m (4 ft by 4 ft).
- PCC mix design: conventional and fiber reinforced (polypropylene).
- Joint sealing: sealed and unsealed.

Shortly after construction, cracking on two of the sections with larger panels was observed, and this was attributed to subgrade problems (Dumitru, Hossain, and Wojakowski 2002). However, additional cracking on the larger panels was observed in 1997 and 1998, and several of the larger panels had to be replaced. The majority of the cracking was corner cracking, even though a strong bond was found to exist between the PCC and the existing HMA (Dumitru, Hossain, and Wojakowski 2002). By 2001, after 6 years of service, all but two of the sections had been overlaid. Panel size was determined to have a significant effect on performance, but the effects of joint sealing and fibers were inconclusive (Dumitru, Hossain, and Wojakowski 2002)

6.4.6 *I-20, Jackson, MS*

The Mississippi Department of Transportation constructed an experimental UTW overlay on a portion of I-20 near Jackson in 1997 (Crawley and Pepper 1998). The UTW overlay was constructed 102-mm (4-in) thick and contained polyolefin fibers; sections were included with 1.8, 4.6, 6.1, and 12.2 m (6, 15, 20, and 40 ft) slab sizes to evaluate the ability of the fibers to accommodate longer joint spacings (which would reduce sawing and sealing costs) (Crawley and Pepper 1998). Conventional 152- and 203-mm (6- and 8-in) whitetopping overlays were also constructed as part of the project.

The existing pavement was originally constructed in 1967 and had been rehabilitated four times prior to the UTW placement (Crawley and Pepper 1998). The in-place pavement structure consisted of 381 mm (15 in) of HMA over a base and subbase. Approximately 102 mm (4 in) of HMA was removed prior to placement of the UTW.

After 1 year of service, some corner cracking was observed in the sections with the larger slab sizes (6.1 and 12.2 m [20 and 40 ft]), a few of which had deteriorated to the point that they had to be patched (Crawley and Pepper 1998). The cause of this cracking is suspected to be curling.

6.4.7 *Ellaville Weigh Station, I-10, Ellaville, FL*

An experimental UTW project was constructed by the Florida Department of Transportation at the Ellaville Weigh Station on I-10 in 1997 (Armaghani and Tu 1999). The project was built to address recurring rutting (as much as 45 mm [1.8 in]) that developed in the primary travel paths of the weigh station. Three UTW designs were constructed (Armaghani and Tu 1999):

- Design 1: 80 mm (3.1 in) UTW with slab size 1.2 m by 1.2 m (4 ft by 4 ft).
- Design 2: 102 mm (4 in) UTW with slab size 1.6 m by 1.6 m (5.2 ft by 5.2 ft).
- Design 3: 102 mm (4 in) UTW with slab size 1.2 by 1.2 m (4 ft by 4 ft).

These designs were repeated on both the west side (before the weighing platform) and the east side (after the weighing platform) of the weigh station. The west side incorporated fibrillated polypropylene fibers, whereas the east side contained no fibers (Armaghani and Tu 1999).

The existing pavement consisted of an HMA layer (ranging from 80 mm [3.1 in] to 175 mm [6.9 in] thick) over 275 mm (10.8 in) of limerock base (Armaghani and Tu 1999). A variable milling depth was selected such that a minimum 50-mm (2-in) HMA layer remained, although some areas of excessive milling were noted during the construction.

After 13 months of performance, it is reported that the UTW is in good condition despite the heavy and high volume of truck traffic associated with the weigh station (Armaghani and Tu 1999). Only 5.5 percent of the 1800 panels have developed cracking, the majority of which were corner cracking that appeared to be the result of late sawing of the joints. It was generally observed that thicker slabs developed less cracking than thinner slabs and that panel size had no apparent effect on cracking (Armaghani and Tu 1999). Furthermore, the presence of fibers did not appear to affect the cracking potential, although the fibers may help reduce shrinkage and keep cracks tight (Armaghani and Tu 1999).

6.4.8 *Mn/ROAD Thin and Ultra-Thin Whitetopping Project*

In 1997, the Minnesota Department of Transportation constructed several thin and ultra-thin whitetopping overlays as part of the Minnesota Road Research (Mn/ROAD) test facility (Vandenbossche and Rettner 1998; Vandenbossche and Fagerness 2002). The PCC overlays ranged in thickness from 76 to 152 mm (3 to 6 in), and joint spacings ranged from 1.2 m by 1.2 m (4 ft by 4 ft) to 3.1 by 3.7 m (10 ft by 12 ft). Two different PCC mix designs were also employed in this project, one with polyolefin fibers (added at a dosage rate of 15 kg/m^3 [25 lb/yd^3]) and one with polypropylene fibers (added at a dosage rate of 1.8 kg/m^3 [3 lb/yd^3]) (Vandenbossche and Rettner 1998).

The pavements were evaluated in 2001 after about 3½ years and 4.7 million ESAL applications. The thin (152 mm [6 in]) whitetopping test sections are in excellent condition and are not exhibiting any distress (Vandebossche and Fagerness 2002). However, transverse and corner cracking had occurred in the UTW test sections, primarily in the truck lane (Vandenbossche and Fagerness 2002). An analysis of the performance data suggested that the selection of an effective joint pattern, in which the longitudinal joints are kept outside of the wheelpaths, will increase the performance of the overlays. Other factors noted to be important to the performance of the UTW are the quality of the HMA beneath the overlay and the HMA temperature and stiffness (Vandenbossche and Fagerness 2002).

6.4.9 *ACPA Performance Evaluations*

In 1995 and 1996, the American Concrete Pavement Association (ACPA) conducted detailed condition surveys on nine UTW projects: six in Tennessee and three in Georgia. The purpose of these surveys was to examine the early performance of UTW overlays, and the nine projects were selected primarily because they are some of the older UTW overlays (Cole 1997). The condition surveys were conducted in accordance with the pavement condition index (PCI) procedures developed by the Corps of Engineers and adopted by the American Public Works Association.

Based on the results of the condition surveys (which represent about 4 to 5 years of performance data), the following conclusions are drawn (Cole 1997):

- Nine of the ten sections are rated in excellent condition.
- Some cracks have occurred on the various PCC slabs, but most of these cracks are low severity (less than 0.5 mm [0.02 in] wide) and do not appear to be affecting pavement ride quality.
- The first and last panels (approach and leave ends) of the UTW overlays contain a higher percentage of cracking than the rest of the project. This is believed to be the result of impact loading as the wheel crosses from the adjacent HMA pavement to the UTW overlay.
- Sections with the highest PCI have the smallest panel size and a significant underlying HMA thickness.

6.5 Summary

Ultra-thin whitetopping is a relatively new paving technology in which a thin layer of PCC (between 50 and 102 mm [2 and 4 in]) with short joint spacing (typically 0.6 to 1.8 m [2 to 6 ft]) is placed on an existing HMA pavement. The existing HMA pavement is milled to remove distortions and surficial distress and to promote bonding between the PCC overlay and the HMA pavement. The result is a monolithic-type pavement structure with increased load carrying capacity that is suited for use on parking lots, residential streets, low volume roads, general aviation airports, and HMA intersections where rutting is a problem.

An interim design procedure exists for UTW overlays in which the load carrying capacity of a specific UTW design may be estimated. The slab thickness, joint spacing, strength, and subgrade k-value are all needed to determine the estimated total number of trucks that the pavement can carry. If the daily truck applications are known, the life of the UTW can then be estimated.

A variety of PCC mix designs have been used in UTW overlays, depending upon the opening-to-traffic criteria for the specific application. Many UTW projects have employed fast-track mixes that produce compressive strengths of 20.7 MPa (3000 lbf/in^2) in as little as 24 hours. Synthetic fibers (polyolefin or polypropylene) are often added to these mixtures to reduce plastic shrinkage, increase toughness, and increase post-cracking performance.

Prior to the placement of the UTW, the existing HMA pavement must be milled to the specified depth, leaving an absolute minimum HMA thickness of 76 mm (3 in). Depending upon the size of the paving project, conventional fixed-form or slipform paving operations are used in UTW construction. A greater application of curing compound is recommended to help minimize moisture loss, and the timely and effective sawing of all joints is important in preventing random cracking.

A performance review of UTW projects indicates that most are performing very well. Some problems with corner cracking have been noted and are often attributed to late sawing or to the placement of the longitudinal joints in the wheelpaths. Cracking of the approach and leave sides of UTW overlays has also been observed, and this is believed to be caused by impact loading as the wheel moves from the adjacent HMA pavement to the UTW. Using thickened edges at these locations is one way of addressing this problem. Continued performance monitoring is recommended in order to obtain a better indication of the actual service lives of these pavements.

CHAPTER 7. SELECTION OF PCC OVERLAY ALTERNATIVES

7.1 Introduction

The selection of a particular type of PCC overlay as a possible rehabilitation alternative for an existing pavement is a subset of the overall pavement rehabilitation selection process. The basic principles of that overall pavement rehabilitation selection process are summarized in several documents, including the 1993 AASHTO design guide (AASHTO 1993), an ACPA technical bulletin (ACPA 1993), and the reference manuals for two National Highway Institute (NHI) training course (Grogg et al. 2001; Hoerner et al. 2001). Having the implied goal of optimizing pavement rehabilitation, the process involves first identifying rehabilitation alternatives that are technically feasible and then evaluating the candidate alternatives in terms of cost and performance benefits in order to identify the most appropriate option.

Conceptually, a general relationship exists between the existing pavement condition and the required type of rehabilitation, as shown in figure 7-1. That is, as pavements reach a more advanced state of deterioration, more substantial rehabilitation measures are required. However, the selection process is complicated by the need to consider a variety of factors, many of which are difficult to quantify and evaluate in comparable terms. Examples of such factors include user costs, lane closure requirements, traffic control considerations, desired performance life, duration of construction, and local experience with the rehabilitation alternative. The inability to reliably predict the performance of rehabilitated pavements is also a significant shortcoming in the process. PCC pavement rehabilitation selection is the topic of several on-going research projects, including NCHRP Project 10-50, *Guidelines for Selecting Strategies for Rehabilitation of Rigid Pavements Subjected to High Traffic Volumes*, and an FHWA-sponsored research project, *Repair and Rehabilitation of Concrete Pavement*.

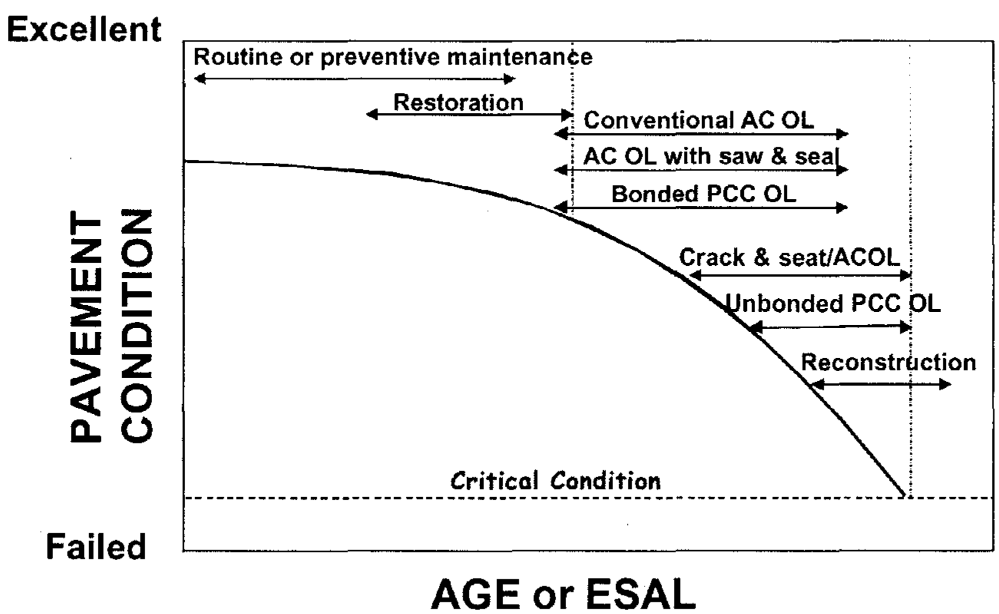

Figure 7-1. Conceptual relationship between pavement condition and required PCC pavement rehabilitation treatments (Darter and Hall 1990).

Presented in this chapter is an overview of recommended practices for PCC overlay type selection. A brief summary of the effectiveness of different types of PCC overlays is first presented, followed by a summary of the overall selection process, including a recommended approach for combining various decision factors.

7.2 Effectiveness of Different Types of PCC Overlays

This section provides a brief summary of the features of different types of PCC overlays that are relevant to overlay type selection, as detailed descriptions of different types of PCC overlays were presented in preceding chapters. In practice, candidate rehabilitation alternatives for pavement rehabilitation considerations are not limited to PCC overlays; therefore, where appropriate, HMA counterparts to PCC overlays are also mentioned. Table 7-1 provides a summary of the advantages and disadvantages of different types of overlays.

Table 7-1. Advantages and disadvantages of different types of pavement overlays.

Treatment	Applicability	Advantages	Disadvantages	Typical Life*
PCC Overlays				
Bonded	• PCC pavement in relatively good condition with no materials-related distress	• Significant increase in structural capacity can be achieved with a relatively thin (75 to 150 mm [3 to 6 in]) overlay	• For pavements in good condition only • Requires extensive preoverlay repairs. Working cracks on existing pavement will reflect through • Bond is essential to good performance • Longer duration of construction than HMA overlays • High initial cost	15 – 25 years
Unbonded	• All PCC pavements	• Relatively insensitive to condition of the underlying pavement—can be applied to PCC pavements in poor condition • Requires minimal preoverlay repairs • High reliability	• Vertical clearances can be a problem • Longer duration of construction than HMA overlays • High initial cost	20 – 30 years
Whitetopping	• All HMA pavements	• Longer design life than HMA • Can be applied on badly deteriorated HMA pavements • Eliminates rutting and shoving problems • High reliability	• Vertical clearances can be a problem • Longer duration of construction than HMA overlays • High initial cost	20 – 30 years
Ultra-Thin Whitetopping	• HMA pavements in fair to good condition	• Longer design life than HMA • Eliminates rutting and shoving problems	• Debonding can lead to premature failure • Requires a thicker HMA pavement with adequate structural capacity • Longer duration of construction than HMA overlays • High initial cost	5 – 15 years *(estimated)*
HMA Overlays				
HMA overlay without slab fracturing	• PCC pavement in fair to good condition	• Easy to construct • Short duration of construction • Low cost	• Susceptible to reflection cracking • Existing structural distresses must be repaired full depth to avoid reflection cracking • May accelerate materials-related distress	8 – 15 years
HMA overlay with slab fracturing	• All badly deteriorated PCC pavements	• Shorter duration of construction than PCC overlays • High reliability (rubblization)	• Breaking or cracking and seating may not always prevent reflection cracking • A relatively thick overlay is needed after rubblization to obtain desirable performance • Vertical clearances can be a problem	8 – 25 years

* Life estimates based on data provided in Hall et al. (2001).

7.2.1 *Bonded PCC Overlays*

Bonded overlays are appropriate for PCC pavements that are in good condition but are in need of structural enhancement. Bonded overlays can also be used to address various types of functional deficiencies, including the following:

- Poor surface friction.
- Surface roughness (other than faulting).
- Surface "rutting" caused by studded tires.
- Excessive noise levels.

Studies on the field performance of bonded PCC overlays have shown mixed results (Hutchinson 1982; Voigt et al. 1989; Peshkin and Mueller 1990; McGhee 1994). While some projects have exhibited excellent performance, several others failed within the first few years after construction. Common causes of some of these premature failures include the inappropriate use of bonded overlays on excessively deteriorated pavements, an inadequate amount of preoverlay repair, and failure to achieve long-term bonding between the two PCC layers.

Structurally, HMA overlays (without slab fracturing) are similar to bonded PCC overlays, so if a 10- to 15-year service life is acceptable, then an HMA overlay could be used to obtain similar performance as a bonded PCC overlay. However, PCC pavements overlaid with HMA are subjected to much lower temperature gradients (Nishizawa et al. 2000). Thus, while HMA overlays do not provide the same level of reduction in load stresses as bonded PCC overlays, a reduction in *combined* stresses can be achieved with a moderate thickness HMA overlay because of the lower thermal curling stresses.

7.2.2 *Unbonded PCC Overlays*

An unbonded overlay is a feasible rehabilitation alternative for PCC pavements in practically any condition, including those with materials-related distresses (MRD) such as D-cracking or reactive aggregate. Unbonded overlays are particularly effective on badly deteriorated PCC pavements because they can be placed with minimal preoverlay repairs.

Unbonded overlays are the most common type of PCC overlay in use today. Their performance has generally been very good, and they are considered a long-term rehabilitation solution that is expected to provide a level of service and performance life comparable to that of new PCC pavements (ACPA 1990b; McGhee 1994; ERES 1999a). Although relatively insensitive to the condition of the underlying pavement, some limited preoverlay repair may be required. In lieu of conducting peroverlay repairs, the existing pavement may also be fractured prior to overlaying, which may be appropriate if the existing pavement is severely deteriorated structurally or if it is exhibiting materials-related problems.

An alternative to an unbonded overlay is the placement of an HMA overlay, with or without slab fracturing. These can be placed very quickly and at a lower initial cost than unbonded PCC overlays. For severely deteriorated PCC pavements or PCC pavements with MRD, slab fracturing may be the most economical and reliable approach to preparing the existing pavement for the overlay. On the other hand, the main advantage of the unbonded PCC overlay option over the HMA overlay option is the longer expected service life.

7.2.3 *Whitetopping Overlays*

Whitetopping is an effective treatment for rehabilitating deteriorated HMA pavements that exhibit severe structural deterioration, such as rutting, shoving, and alligator cracking. Little preoverlay repair is required prior to placing a whitetopping overlay, although some milling may be needed if significant rutting exists or if other profile corrections are needed (ACPA 1991b; ACPA 1998). Structurally, whitetopping overlays are similar to conventional PCC pavements constructed on an asphalt-treated base. Whitetopping overlays have been successfully constructed with JPCP, JRCP, and CRCP designs, but JPCP designs are most common. The majority of the whitetopping projects have provided good to excellent performance (Lokken 1981; Hutchinson 1982; McGhee 1994; ACPA 1998).

7.2.4 *UTW Overlays*

UTW overlays are best suited to HMA pavements on local roads, intersections, or parking lots that exhibit severe rutting, shoving, and potholing problems. Because the underlying HMA pavement is an integral part of the structural system for UTW overlays, a minimum HMA thickness of 76 mm (3 in) (after milling) is required for UTW projects (Grogg et al. 2001), although some suggest a minimum HMA thickness of 152 mm (6 in) (Silfwerbrand 1997). Proper preparation of the existing HMA pavement is also essential to ensure good performance of UTW overlays. This includes repair of any failed or severely deteriorated areas to provide adequate (and uniform) load-carrying capacity and milling of the HMA surface to promote good bonding. Although long-term performance data are limited, the short-term performance of UTW projects has generally been good (Cole 1997).

7.3 Selection Process

The AASHTO pavement design guide (AASHTO 1993) provides general guidelines for pavement rehabilitation selection. Similar guidelines are also provided in an ACPA technical bulletin *Pavement Rehabilitation Strategy Selection* (ACPA 1993). Figure 7-2 illustrates the step-by-step process described in the AASHTO guide. The process involves the following steps:

- Phase 1: Problem Definition. In this step the condition of the pavement is established, the needs are determined, and any project constraints are identified.

- Phase 2: Potential Problem Solutions. This step sorts through all available solutions and develops a "short list" of feasible solutions that address both the needs and constraints of the project.

- Phase 3: Select Preferred Solutions. This step considers both monetary and nonmonetary factors to select the alternative deemed most appropriate for the project design conditions and constraints.

In general, the pavement-related aspects of this process are well defined, and there are numerous references providing detailed information on pavement evaluation procedures and on the design and construction of pavement rehabilitation techniques (see, for example, Hall et al. 2001; Hoerner et al. 2001; and Grogg et al. 2001).

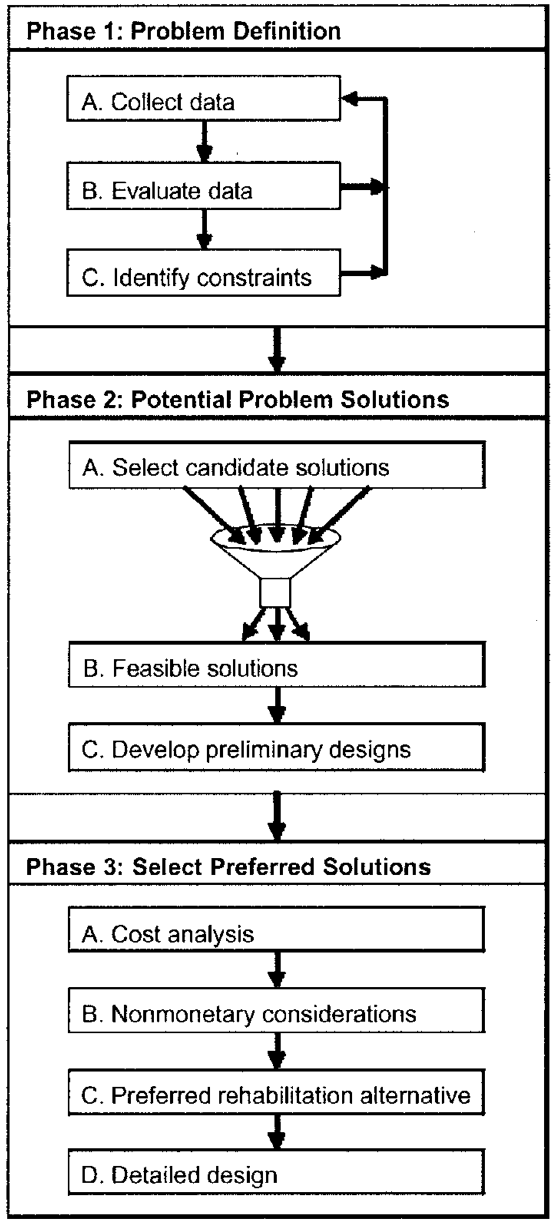

Figure 7-2. Rehabilitation decision process (AASHTO 1993).

The complexity of the rehabilitation selection process lies in the selection stage (Phase 3 in figure 7-2), which involves consideration of agency costs, user costs, and nonmonetary factors to identify the preferred rehabilitation alternative. Examples of nonmonetary factors include service life, duration of construction, and local experience with the rehabilitation alternative, any of which may limit the feasibility of some rehabilitation alternatives. Currently, there are no universally accepted procedures for combining the different costs (agency and nonagency cost) and nonmonetary factors in identifying the preferred rehabilitation alternative. Practices vary from agency to agency, and for now pavement rehabilitation selection is considered more of an art than science (AASHTO 1993).

The following subsections briefly describe the AASHTO procedure for the selection of the preferred rehabilitation alternative, following the flowchart shown in figure 7-2.

7.3.1 *Define the Problem*

As shown in figure 7-2, the first step in identifying feasible rehabilitation alternatives is understanding the problem. This involves conducting a detailed evaluation of the pavement section to assess its condition and to identify the causes of deterioration. This step establishes the engineering criteria for feasible rehabilitation alternatives. The technical feasibility is determined based on the following factors (Zollinger et al. 2001b):

- Pavement condition (distress type, severity, and extent).
- Causes of distress (structural, construction-related, material-related, or functional).
- Expected traffic levels.
- Type of climate (temperature, moisture).
- Rehabilitation design life.

The above factors can be compared against the rehabilitation treatment characteristics summarized in table 7-1 to identify technically feasible overlay options.

In the AASHTO procedure, the constraints that limit feasibility of rehabilitation alternatives are also identified at this stage. The constraints may include the following (AASHTO 1993):

- Project funding.
- Lane closure availability.
- Desirable performance life.
- Geometric requirements.
- Constructibility (utilities, clearances, and available materials and equipment).
- Contractor expertise and manpower.
- Agency policies.

In addition, consideration may also be given to how a rehabilitation alternative may affect the network as a whole. It may occasionally be necessary to select an alternative that is not optimal for a project because of overall network constraints.

7.3.2 *Identify Feasible Alternatives*

In the second step of the AASHTO rehabilitation selection process, the feasible rehabilitation alternatives are identified. These are rehabilitation alternatives that not only address the distresses and deficiencies of the pavement, but also meet the constraints of the project. For example, a bonded overlay would not be selected as a feasible alternative for a badly deteriorated PCC pavement, nor would a thin HMA overlay be selected as a feasible alternative if the desired performance life is 20 years. Only solutions that meet both the pavement needs and the project constraints should be selected. Also at this point, the development of preliminary designs may be required for the purposes of estimating the cost of the candidate rehabilitation alternatives (for use in later cost analyses).

7.3.3 *Select Preferred Alternative*

The final step of the AASHTO project is the selection of the rehabilitation alternative deemed most appropriate for the pavement and the project constraints. As previously mentioned, there is no universally accepted procedure for this, but it is often based on both monetary (agency and nonagency costs) and nonmonetary factors. The consideration of both monetary and nonmonetary factors is discussed in the following sections.

7.3.3.1 Cost Analysis

The consideration of costs plays a key role in the rehabilitation selection process. Costs include not only "hard" costs incurred by the agency (such as construction, maintenance, and rehabilitation costs), but also user costs, which are costs incurred by the users of the pavement facility over the analysis period. User costs consist of the following components (Walls and Smith 1998; Hall et al. 2001):

- Vehicle operating costs, which are costs related to operating the vehicle, including consumption of fuel and oil, and wear on the vehicle.

- Delay costs, which are costs due to reduced speeds, queuing, or the use of alternate routes.

- Crash costs, which are costs associated with damage to the user's vehicle and public property, as well as injuries resulting from crashes.

Note that user costs include the costs incurred during normal usage (in-service user costs) as well as those incurred during rehabilitation construction (work zone user costs). However, for the purposes of rehabilitation treatment selection, the user costs of interest are those that are different between the competing rehabilitation alternatives; in other words, the user costs that are similar among all alternatives are left out in the user cost calculation. Therefore, the main user cost item most often included in a cost analysis is the user delay cost, which is dependent on the number of available traffic lanes, the total traffic volume, and the length of time that the work zone is in place (Walls and Smith 1998).

User costs can be a major factor for consideration on urban projects where traffic congestion is a major concern (TRB 1998). Depending on the traffic control plan and the duration of construction, the work zone user costs can be significantly different for different rehabilitation alternatives. Furthermore, the user costs are often so much greater than the agency costs that they completely overwhelm any differences in agency costs among different rehabilitation alternatives. However, since the user costs are not direct agency costs, it is recommended that they be considered on a separate basis (rather than being combined with the agency costs) in the decision making process (Zollinger et al. 2001a). One approach is to use different weighting factors in combining agency costs and user costs; another is to treat user costs as a separate evaluation factor and assess them in a manner similar to the way nonmonetary factors are considered.

In evaluating the costs of various rehabilitation alternatives, the use of a life-cycle cost analysis (LCCA) is recommended (AASHTO 1993; Walls and Smith 1998). LCCA is an economic analysis procedure that allows a comparison of different rehabilitation alternatives with different cost streams on a common basis. The procedure involves converting costs incurred at different times within the analysis period to a present worth (PW) considering the time cost of money, and adding up the PW of all items in the cost stream for each alternative. LCCA results are either left as PW or alternatively may be expressed as an equivalent uniform annual cost (EUAC). PW and EUAC are calculated as follows (Meredith et al. 1973; Peterson 1985; Hall et al. 2001):

$$PW = F \cdot \frac{1}{(1+i)^n} \tag{7-1}$$

where:

PW = Present worth
F = Future cost
i = Discount rate
n = Year in which the cost is incurred

$$EUAC = P \cdot \left[\frac{i \cdot (1+i)^n}{(1+i)^n - 1} \right] \tag{7-2}$$

where:

$EUAC$ = Equivalent Uniform Annual Cost
P = Present value (PW)
i = Discount rate
n = Year in which the cost is incurred

The discount rate is defined as the interest rate used in calculating the present worth of expected yearly benefits and costs, and represents the time value of money. In the public sector (where money often is not directly borrowed for a given project), the discount rate represents the opportunity cost of an alternative use (investment) of the project funds. It is often approximated as the difference between the commercial interest rate and inflation rate as given by the consumer price index (hence the term discount rate since it is an interest rate that has been discounted for inflation).

The use of an appropriate discount rate for an LCCA is critical to the selection of the preferred rehabilitation alternative because it can have a significant effect on the outcome. The use of a low discount rate (for example, 2 to 3 percent) favors projects with large initial costs, whereas the use of a high discount rate (say, 6 to 8 percent) favors projects that have lower initial costs but higher future (maintenance or rehabilitation) costs. Typical values used by states range from 3 to 5 percent, with an overall average of about 4 percent (Walls and Smith 1998; Hall et al. 2001).

The choice of analysis period also affects the LCCA results. The analysis period is the time period over which the life-cycle costs (LCC) are calculated, and is not necessarily the same as the "life" of the treatment. In general, the analysis period should be long enough to include at least one additional rehabilitation activity (Zollinger et al. 2001a). If the PW method is used to compare LCC, the same analysis period must be used for all rehabilitation alternatives being compared. If the EUAC method is used, a different analysis period may be used for different alternatives. Although analysis periods for new pavement design are often 20 to 50 years, the analysis period for rehabilitation work is usually shorter, such as 10 to 20 years and will depend on the future use of the facility, the need for geometric improvements, and other factors.

To ensure that all rehabilitation alternatives are compared on an equal basis, the value of the pavement structure at the end of the analysis period must be included. This "salvage value" consists of two components: residual value and serviceable life (Zollinger et al. 2001a). The residual value is the net value from recycling the existing pavement. The differential residual value between different rehabilitation strategies is not significant, especially when discounted over the analysis period (Zollinger et al. 2001a). The serviceable life is the remaining service life of a rehabilitation alternative at the end of the analysis period. For example, if the design life of the last rehabilitation extends 5 years beyond the end of the analysis period, the alternative has a remaining service life of 5 years. The salvage value of the remaining serviceable life can be taken as the percent remaining life multiplied by the cost of rehabilitation, which can be assumed to be credited at the end of analysis period (Walls and Smith 1998).

Traditionally, LCCA has been conducted as a deterministic procedure in which all input values are predetermined and remain static throughout the analysis. Therefore, any errors in predicted life of rehabilitation treatments or variations in treatment costs are ignored. However, the performance lives of pavement overlays are subject to variability, and the consideration of that variability (and the variability associated with other factors such as discount rate and cost data) may be important to obtain realistic results. Any effects of variability in the input values can be overcome, to a large extent, by using a probabilistic LCCA procedure (Walls and Smith 1998). In this approach, thousands of simulations are run to model the variability in input values such as performance life and timing of maintenance activities. The result is a probabilistic distribution of possible outcomes, which allows agencies to examine the risk associated with each alternative. The FHWA Interim Technical Bulletin *Life Cycle Cost Analysis in Pavement Design* (Walls and Smith 1998) provides detailed technical guidance on LCCA procedures, including the use of probabilistic concepts.

The FHWA considers the use of life-cycle cost analysis good engineering practice in pavement type and rehabilitation selection. In Section 106 of TEA-21, the FHWA recommends evaluating the total economic worth of a usable project segment by analyzing initial costs and discounted future costs, such as maintenance, user costs, reconstruction, rehabilitation, restoring, and resurfacing costs, over the life of the project segment.

7.3.3.2 Consideration of Nonmonetary Factors

Although LCC is usually the key determining factor in rehabilitation selection, various other factors can also play an important role in the decision process. Nonmonetary factors that can influence rehabilitation selection include the following (AASHTO 1993; Hall et al. 2001; Zollinger et al. 2001a):

- Service life.
- Duration of construction.
- Traffic control problems.
- Environmental impact.
- Maintainability.
- Political concerns.
- Agency experience with the rehabilitation design.
- Contractor experience and capability.
- Constructibility (e.g., geometric restrictions; availability of materials and equipment).

In general, the above factors are difficult to quantify in monetary terms. However, in some cases, some of the factors above may be specified as project requirements that candidate alternatives must satisfy. For example, a 30-year design life with minimal maintenance may be a requirement in urban areas where lane closures for pavement rehabilitation pose immense traffic control problems. The maximum duration of construction may also be specified as a project requirement.

7.3.3.3 Selection Approach

There are several different approaches that can be used to make the final selection of the preferred rehabilitation alternative. This might include the consideration of LCCA results only, the consideration of some other engineering economic parameters (e.g., benefit/cost ratio, internal rate of return), or the consideration of costs while subjectively considering nonmonetary factors. There is some difficulty, however, in combining those nonmonetary factors as part of a comprehensive evaluation procedure.

One approach that is often used to select the preferred rehabilitation alternative considering multiple selection criteria (such as LCCA and nonmonetary factors) is a decision matrix, an example of which is shown in table 7-2 (AASHTO 1993). In this procedure, evaluation factors are selected and relative importance ratings are assigned to each (for example, service life is one of the criterion shown in table 7-2 and it is assigned an importance of 25 percent). Each rehabilitation alternative is then rated under each evaluation criterion (the number on the upper left corner of each cell in table 7-2) on a fixed scale according to how well it meets the criteria (a scale of 0 to 100 is used in the example shown in table 7-2). The rating is then multiplied by the relative importance to determine the score under each evaluation criterion (the number on the lower right corner of each cell in table 7-2). The total score for each alternative is obtained by summing the individual scores across the columns, and the alternatives are ranked according to the total score to determine the preferred alternative.

The decision matrix can be a useful tool for identifying the alternative that best satisfies the evaluation criteria. One limitation of this method is that the ratings under each evaluation criterion do not necessarily reflect the relative merits of different alternatives. For some of the factors (such as cost or service life), either the calculated value or the anticipated performance can be used to determine the score that reflects the relative merit. However, there may be no good objective measure of relative merit for some of the factors (e.g., "proven design in state climate"). A more robust, systematic procedure for determining the preferred alternative is currently being developed under an FHWA project, *Repair and Rehabilitation of Concrete Pavement* (Zollinger et al. 2001b).

Table 7-2. Example decision matrix (AASHTO 1993).

	Criteria							
	Initial Cost	Duration of Construction	Service Life	Repairability & Maintenance Effort	Rideability & Traffic Orientation	Proven Design in State Climate	Total Score	Rank
Relative Importance	20%	20%	25%	15%	5%	15%	100.0	
Alternative 1	60 / 12.0	60 / 12.0	100 / 25.0	80 / 12.0	90 / 4.5	100 / 15.0	80.5	1
Alternative 1A	60 / 12.0	60 / 12.0	100 / 25.0	80 / 12.0	90 / 4.5	100 / 15.0	80.5	1
Alternative 2	60 / 12.0	60 / 12.0	70 / 17.5	50 / 7.5	60 / 3.0	40 / 6.0	58.0	5
Alternative 2A	60 / 12.0	60 / 12.0	70 / 17.5	50 / 7.5	60 / 3.0	40 / 6.0	58.0	5
Alternative 3	60 / 12.0	40 / 8.0	100 / 25.0	80 / 12.0	100 / 5.0	90 / 13.5	75.5	2
Alternative 4	60 / 12.0	80 / 16.0	40 / 10.0	20 / 3.0	40 / 2.0	20 / 3.0	46.0	6
Alternative 5	40 / 8.0	60 / 12.0	40 / 10.0	50 / 7.5	50 / 2.5	30 / 4.5	44.5	8
Alternative 6	70 / 14.0	80 / 16.0	60 / 15.0	50 / 7.5	80 / 4.0	40 / 6.0	62.5	4
Alternative 7	100 / 20.0	100 / 20.0	20 / 5.0	20 / 3.0	40 / 2.0	40 / 6.0	56.0	6
Alternative 8	30 / 6.0	60 / 12.0	100 / 25.0	100 / 15.0	100 / 5.0	30 / 4.5	67.5	3

7.4 Current Practice

For the most part, the choice among PCC overlays is usually clearly defined by engineering criteria. For existing PCC pavements, the possible PCC overlay choices are bonded and unbonded overlays. In general, the condition of the existing pavement dictates whether a bonded PCC overlay is a practical option. Where more than one type of PCC overlay is feasible, the selection may be made based on cost, because the impact of construction (which affects most of the nonmonetary factors) is similar for all PCC overlays. For cost evaluations, LCC is recommended in most guidelines and is the accepted standard practice.

When other rehabilitation alternatives (such as HMA overlay alternatives, PCC pavement restoration [CPR] alternatives, and reconstruction) are also considered, the consideration of user costs and nonmonetary factors becomes more relevant. Rehabilitation selection is typically made considering all feasible alternatives. The user costs and nonmonetary factors are given a high degree of importance at most highway agencies (SHAs), especially on urban projects where lane closures would cause significant congestion problems (FHWA 1998; TRB 1998). Several SHAs have explicit policies that limit traffic delays due to pavement rehabilitation projects (FHWA 1998). However, there are no generally accepted means of evaluating the various factors involved in identifying the preferred rehabilitation alternative. For the most part, an informal process is used (Zollinger et al. 2001b).

7.5 Summary

Although some aspects of pavement rehabilitation selection are well defined, the selection process is complicated by the need to consider numerous factors that are difficult to quantify and evaluate in comparable terms. The process involves the following steps:

- Phase 1: Problem Definition. In this step the condition of the pavement is established, the needs are determined, and any project constraints identified.

- Phase 2: Potential Problem Solutions. This step sorts through all available solutions and develops a "short list" of feasible solutions that address the needs and constraints of the project.

- Phase 3: Select Preferred Solutions. This step considers both monetary and nonmonetary factors to select the alternative deemed most appropriate for design conditions and constraints.

In general, the pavement-related aspects of this process are well defined, and the guidelines provided in numerous references can be used to identify technically feasible and preferable rehabilitation alternatives.

The selection process is much simpler if only PCC overlay alternatives are considered, but there is no practical value in limiting rehabilitation choices in that way. The choice among PCC overlays is often clearly defined by the engineering criteria. Where more than one type of PCC overlay is feasible, the selection may be based on LCC because the impact of construction (which affects most of the nonmonetary factors) is similar for all PCC overlays. When other rehabilitation alternatives (such as HMA overlay alternatives, CPR alternatives, and reconstruction) are also considered, the consideration of user costs and nonmonetary factors becomes more relevant. The decision matrix shown in table 7-2 can be a useful tool for identifying the alternative that best satisfies multiple selection criteria. One limitation of this approach is that it is difficult to rate different alternatives such that the relative merits of each alternative are properly represented in the rating for many of the factors.

Although most SHAs regard user cost and nonmonetary factors as very important decision factors in rehabilitation selection, there are no generally accepted means of combining these factors. For the most part, an informal process is used, although systematic procedures for rehabilitation selection are currently being developed under on-going research projects.

CHAPTER 8. SUMMARY AND FUTURE RESEARCH NEEDS

8.1 Summary

This report describes the four types of PCC overlays that are commonly used in highway pavement applications: bonded PCC overlays, unbonded PCC overlays, conventional whitetopping, and ultra-thin whitetopping. Each of these has unique design and construction characteristics, as summarized below:

- Bonded PCC overlays are thin overlays (typically 76 to 102 mm [3 to 4 in] thick) that are bonded to an existing PCC pavement, creating a monolithic structure. They are used to increase the structural capacity of an existing PCC pavement or to improve its overall ride quality, and should be used where the underlying pavement is free of structural distress and in relatively good condition. Achievement of adequate bond and the matching of joints between the two PCC layers are two items critical to the performance of bonded PCC overlays.

- Unbonded PCC overlays contain an interlayer between the existing PCC pavement and the new PCC overlay. This separation layer is placed to ensure independent behavior between the two slabs, thereby minimizing the potential for reflection cracking. Unbonded PCC overlays are typically constructed between about 152 to 305 mm (6 to 12 in) thick and can be placed on PCC pavements in practically any condition, including those in advanced stages of deterioration or with significant materials-related problems (such as D-cracking or reactive aggregate).

- Conventional whitetopping is the placement of a PCC overlay on an existing distressed HMA pavement. These types of overlays are generally designed as new PCC pavement structures and are constructed 102 to 305 mm (4 to 12 in) thick. Conventional whitetopping is generally placed on existing HMA pavements with significant deterioration.

- UTW is a rehabilitation process in which a thin layer of PCC (between 50 to 102 mm [2 to 4 in] thick) is placed over a distressed HMA pavement. The existing HMA surface is milled in order to enhance the bond between the PCC overlay and the existing HMA pavement to create a monolithic structure. UTW overlays employ short slabs, typically 0.6 to 1.8 m (2 to 6 ft) squares. Closely related to UTW overlays are *thin* whitetopping overlays, which is a 102 to 203-mm (4 to 8-in) thick PCC overlay placed on a milled HMA pavement. As with UTW, the bond between the PCC overlay and HMA pavement is relied upon in the design procedure, and shorter joint spacing (between 1.8 and 3.7 m [6 and 12 ft]) is used. Thin whitetopping overlays have been used most often on state highways and secondary routes.

Four chapters of this report are devoted to each PCC overlay type, describing recommended applications, critical design elements, current overlay design methodologies, recommended construction practices, and performance highlights. Another chapter presents general guidelines for the selection of PCC overlays as a potential rehabilitation treatment for existing distressed pavements. Taken together, this document addresses the current "state of the technology" of PCC overlays of both existing PCC pavements and existing HMA pavements.

8.2 Research Needs

This report, and the research cited herein, highlight several critical research needs on the different types of PCC overlays. Some of these needs are not new, having been identified in previous PCC overlay synthesis documents prepared by Hutchinson (1982) and McGhee (1994), and therefore represent areas that have yet to be fully addressed. Others relate to technologies that have only emerged during the past decade. The key research needs include the following:

- Use of a bonding agent for bonded PCC overlays. Although recent research appears to suggest that a bonding agent is not necessary to ensure good bond between the PCC overlay and the existing PCC pavement, more information is needed on the surface preparation requirements and range of conditions for which a bonding agent may not be needed. Furthermore, information on the impact of environmental conditions on achieving adequate bonding is also needed.

- The performance of bonded PCC overlays is strongly affected by the condition of the existing PCC pavement. Consequently, more precise definitions are needed on the levels of distress in the existing pavement that can be present and still ensure good performance of the PCC overlay.

- Bonding between HMA separator layer and unbonded PCC overlay. There is evidence to suggest that some degree of bonding occurs between an HMA separator layer and an unbonded PCC overlay. The degree of bonding may contribute to the load-carrying capacity of the unbonded PCC overlay, which, if significant and accounted for, could influence the unbonded overlay thickness requirements. Research is needed on the degree of bonding that occurs and the longevity of that bond in contributing to the load-carrying capacity of the pavement structure.

- Bonding between conventional whitetopping overlay and existing HMA pavement. Similar to that described above for unbonded PCC overlays, some degree of bonding occurs between a conventional whitetopping overlay and the existing HMA pavement. However, current design procedures assume an unbonded condition, resulting in a somewhat thicker overlay. Again, research is needed to determine the degree of bonding that occurs, its longevity, and its effect on performance.

- Properties of an effective separator layer for unbonded PCC overlays. A variety of materials have been used as a separator layer, with a 25-mm (1-in) thick dense-graded HMA layer being most commonly used. However, other materials have been used and in some cases have provided good performance. Research on the specific characteristics of a separator layer to ensure good performance and the required separator thickness to ensure separation are needed.

- Partially bonded PCC overlays for highway applications. Although not used extensively in highway applications, partially bonded PCC overlays are enjoying widespread use in airfield applications. It is suggested that research on the potential use and applicability of partially bonded PCC overlays for highways be conducted.

- Design procedure for UTW overlays. A design procedure for UTW overlays does not currently exist, although a procedure does exist for determining the load carrying capacity of various UTW designs. It is suggested that a formal design procedure for UTW be developed and perhaps even packaged in the new 2002 design guide.

- Long-term performance of PCC overlays. Although the long-term performance of several PCC overlay types has been established in some studies, a nationwide evaluation of the long-term performance of PCC overlay types is strongly recommended. In particular, the long-term performance of UTW has not been established.

- Although some studies have been conducted, continued research on the role of fibers in PCC overlays should be evaluated, specifically whether they allow the construction of thinner overlays or the use of longer joint spacings.

- Effective mix designs and curing. Continued research on effective mix designs for PCC overlays and their effective curing is recommended, particularly for thinner overlays (bonded and UTW). This is because the thinner slabs are more susceptible to shrinkage as well as to early-age curling, so additional research on these factors appears warranted.

- Timing of maintenance and rehabilitation treatments. As with new pavements, little information is currently available regarding the optimal time for performing maintenance and rehabilitation of PCC overlays. Research is needed to determine the most effective maintenance and rehabilitation treatments for PCC overlays as well as the timing of such treatments in order to achieve maximum benefit.

- Selection of pavement rehabilitation alternatives. The selection of rehabilitation alternatives for an existing pavement is often based on subjective measures and constraints. Continued research on the development of more formal guidelines and techniques for pavement rehabilitation selection is recommended.

Research on several of these items is currently being conducted under ongoing FHWA, IPRF, and state-sponsored research projects.

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APPENDIX A

ANNOTATED BIBLIOGRAPHY

APPENDIX A
ANNOTATED BIBLIOGRAPHY

Introduction

This *Annotated Bibliography* presents summaries of recent research projects and field studies pertaining to the design, construction, maintenance, and performance of portland cement concrete (PCC) pavement overlays. The reports are grouped by the following topics:

- General Information on PCC Overlays.
- Bonded Concrete Overlays.
- Unbonded Concrete Pavement Overlays.
- Ultra-Thin Whitetopping Overlays.
- Whitetopping Overlays.
- PCC Pavement Rehabilitation Strategy Selection.

Many of the reports contained herein are applicable to more than one subject area, but each is listed only once under its primary subject heading.

A comprehensive literature search was conducted to produce this *Annotated Bibliography.* Literature searches were conducted on several different databases, including the Transportation Research Information Services (TRIS), the Engineering Index (EI Compendex), and the National Transportation Information Service (NTIS) databases. The focus of the literature search was on reports published in the last 15 years, and so most of the reports are from 1985 through the end of 2001. However, several older reports from the period of 1980 to 1985 also turned up during the literature search on PCC pavement overlays and are also included in the *Annotated Bibliography.*

Over 350 total potential records were identified during the literature search, including many articles appearing in popular pavement trade magazines or publications. These articles were generally not included in the *Annotated Bibliography* either because they are generally abbreviated versions of more complete papers that appear elsewhere or because they are very general in nature and do not provide significant information of a technical nature.

Within each topic given above, the publications in the *Annotated Bibliography* are listed alphabetically by the last name of the primary author or by the name of the organization preparing the report. The abstracts included for each record are generally those provided by the author as part of the report or article.

Recommended PCC Overlay Reference Documents

In reviewing the various documents in preparation this report, a "short list" of references that provide useful information on the design, construction, and performance of PCC overlays was compiled. These documents are listed below in alphabetical order and found with complete annotation in the bibliography:

- American Association of State Highway and Transportation Officials (AASHTO). 1993. *AASHTO Guide for Design of Pavement Structures*. American Association of State Highway and Transportation Officials, Washington, DC.

- American Concrete Pavement Association (ACPA). 1990a. *Guidelines for Bonded Concrete Overlays*. Technical Bulletin TB-007P. American Concrete Pavement Association, Arlington Heights, IL.

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Sources of Reference Documents

The reference documents included in this *Annotated Bibliography* may be obtained from one of the sources shown in table A-1.

Table A-1. Sources for references documents.

American Association of State Highway and Transportation Officials (AASHTO) P.O. Box 96716 Washington, DC 20090-6716 (800) 231-3475 www.transportation.org	American Concrete Institute P.O. Box 9094 Farmington Hills, MI 48333-9094 (248) 848-3800 www.aci-int.org
American Concrete Pavement Association 5420 Old Orchard Road Skokie, IL 60077-1083 (847) 966-2272 www.pavement.com	American Society of Civil Engineers P.O. Box 79404 Baltimore, MD 21279-0404 (800) 548-2723 www.pubs.asce.org
FHWA R&D Report Center Philadelphia Court Lanham, MD 20706 (301) 577-0818 (301) 577-1421 (fax) marl.green@fhwa.dot.gov	National Technical Information Service 5285 Port Royal Road Springfield, VA 22161 (703) 605-6000 www.ntis.gov
Portland Cement Association 5420 Old Orchard Road Skokie, IL 60077-1083 (800) 868-6733 www.portcement.org	Transportation Research Board P.O. Box 289 Washington, DC 20055 (202) 334-3213 www.trb.org *(general)* www.trb.org/trb/bookstore *(bookstore)*

Applicable National Highway Institute Training Courses

The National Highway Institute (NHI) sponsors several training courses that provide additional information on the various aspects of PCC overlay design and construction. These courses are listed in table A-2. Additional information on these courses can be obtained from the National Highway Institute at the following address:

Federal Highway Administration
National Highway Institute
4600 North Fairfax Drive, Suite 800
Arlington, VA 22203
Tel: 703-235-0500
Fax: 703-235-0593
Web Address: www.nhi.fhwa.dot.gov

Table A-2. Listing of related NHI training courses.

NHI Course Number	Course Title	Course Length, days
131008	*Techniques for Pavement Rehabilitation*	3.5
131029	*AASHTO Pavement Overlay Design*	3.0
131033	*Construction of Portland Cement Concrete Pavements*	2.5
131062	*PCC Pavement Evaluation and Rehabilitation*	2.5
131063	*HMA Pavement Evaluation and Rehabilitation*	2.5

General Information on PCC Overlays

American Association of State Highway and Transportation Officials (AASHTO). 1993. *AASHTO Guide for Design of Pavement Structures.* American Association of State Highway and Transportation Officials, Washington, DC.

This design guide provides a comprehensive set of procedures that can be used for the design and rehabilitation of both rigid and flexible pavements. Major topics discussed in the guide include general pavement design and management principles, pavement design procedures for new construction or reconstruction, pavement design procedures for rehabilitation of existing pavements, and mechanistic-empirical design procedures.

American Concrete Pavement Association (ACPA). 1993. *Reconstruction Optimization Through Concrete Inlays.* Technical Bulletin TB-013.0. American Concrete Pavement Association, Skokie, IL.

These guidelines cover many available options for inlaying both concrete and asphalt pavements with concrete. Engineers often refer to concrete inlays as overlays or reconstruction. Inlays options do encompass features of these strategies. However, it is the optimization available through inlay strategies that warrants their specific explanation.

Barenberg, E. J. 1981. "Rehabilitation of Concrete Pavements By Using Portland Cement Concrete Overlays." *Transportation Research Record 814.* Transportation Research Board, Washington, DC.

Overlays of portland cement concrete (PCC) pavements are growing in popularity with paving engineers. This shift away from asphalt to concrete as an overlay material is due in part to some recent shortages in asphalt cement and the concomitant increase in the cost of asphaltic concrete materials. Also, some engineers simply prefer concrete surfaces to asphalt for certain applications. PCC overlays are classified as bonded, partially bonded, or unbonded. Within these three classifications are continuously reinforced concrete, jointed concrete, and fibrous concrete overlays. Post-tensioned pre-stressed slabs have also been used as overlays. Not all combinations of overlays and levels of bonding are compatible with all pavement types and all levels of distress. Thus, each job must be evaluated as a separate project that uses the appropriate constraints. To evaluate the relative merits of the different types of overlays, a systematic approach to decision making must be used. The limitations and constraints of the different types of PCC overlays are discussed and a possible decision criterion approach is described for use in evaluating the best overlay alternative.

Campbell, R. L. 1994. *Overlays on Horizontal Concrete Surfaces: Case Histories.* Technical Report REMR-CS-42. U.S. Army Corps of Engineers, Washington, DC.

This study documents the current practices for overlaying horizontal concrete surfaces as a first phase in the development of performance criteria for concrete overlays. The case histories presented were typically for overlays completed within the last 10 years and located at Corps of Engineers civil works projects. Overlays documented included bonded conventional, low-slump, fly-ash, silica-fume, polymer-modified, and fiber-reinforced concretes. Unbonded overlays were also documented. Although the information obtained for each case history varied and was sometimes limited, an attempt was made to provide the following basic information for each repair: (a) project description, (b) cause and extent of damage, (c) description of repair materials and procedures, (d) cost, and (e) performance of repair.

Concrete Reinforcing Steel Institute (CRSI). 1988. *Performance of CRC Overlays: A Study of Continuously Reinforced Concrete Resurfacing Projects in Four States.* Concrete Reinforcing Steel Institute, Schaumburg, IL.

The design and performance of CRC overlays in four states (Arkansas, Texas, Illinois, and Oregon) is documented in this report. The projects range in age from 8 to 23 years and in thickness from 6 to 9 in. The results of the evaluation indicate that CRC overlays can provide reliable and cost effective performance under varying environmental and traffic conditions. Distresses most commonly observed were related to poor drainage conditions, pumping, and loss of support.

Grogg, M. G., K. D. Smith, S. B. Seeds, T. E. Hoerner, D. G. Peshkin, and H. T. Yu. 2001. *HMA Pavement Evaluation and Rehabilitation.* Reference Manual, NHI Course 131063. National Highway Institute, Arlington, VA.

This document serves as the Reference Manual for the FHWA/NHI training course *HMA Pavement Evaluation and Rehabilitation*. The course provides detailed information to assist pavement engineers in identifying and selecting the reliable and cost effective rehabilitation alternatives for existing HMA pavements. This Reference Manual contains four blocks of material. The first block contains an introduction to the course, as well as an introduction to HMA pavements. Block 2 discusses the pavement evaluation process, describing ways of evaluating and characterizing the condition of the existing HMA pavement. Block 3 presents key design and construction information on common HMA pavement maintenance and rehabilitation activities, such as crack sealing, surface treatments, overlays, and recycling. Finally, Block 4 describes a methodology for selecting the preferred rehabilitation alternative from a short list of feasible alternatives, featuring the use of life cycle cost analysis.

Hall, K. D. and N. Banihatti. 1998. *Structural Design of Portland Cement Concrete Overlays for Pavements.* Final Report, MBTC FR-1052. Arkansas Department of Transportation, Little Rock, AR.

The most common method used to rehabilitate existing portland cement concrete (PCC) pavements is to place an asphalt concrete overlay. However, problems with premature rutting and the early appearance of reflective cracks in the asphalt overlay have made PCC overlays a viable alternative rehabilitation method. Most PCC overlay failures can be attributed to causes other than improper overlay thickness, suggesting that existing design procedures such as the AASHTO procedure provide sufficient overlay thickness to satisfy design requirements. In this study, major factors affecting overlay performance were identified; guidelines for considering those factors in design are presented. In addition, user-friendly computer spreadsheets were developed to aid designers in completing AASHTO-based thickness design for unbonded and bonded PCC overlays.

Hoerner, T. E., K. D. Smith, H. T. Yu, D. G. Peshkin, and M. J. Wade. 2001. *PCC Pavement Evaluation and Rehabilitation.* Reference Manual, NHI Course 131062. National Highway Institute, Arlington, VA.

This document serves as the Reference Manual for the FHWA/NHI training course *PCC Pavement Evaluation and Rehabilitation*. The course provides detailed information to assist pavement engineers in identifying and selecting the reliable and cost effective rehabilitation alternatives for existing PCC pavements. This Reference Manual contains four blocks of material. The first block contains an introduction to the course, as well as an introduction to PCC pavements. Block 2 discusses the pavement evaluation process, describing ways of evaluating and characterizing the condition of the existing PCC pavement. Block 3 presents key design and construction information on common PCC pavement maintenance and rehabilitation activities, such as crack sealing, surface treatments, overlays, and recycling. Finally, Block 4 describes a methodology for selecting the preferred rehabilitation alternative from a short list of feasible alternatives, featuring the use of life cycle cost analysis.

Hutchinson, R. L. 1982. *Resurfacing with Portland Cement Concrete.* NCHRP Synthesis of Highway Practice 99. Transportation Research Board, Washington, DC.

Portland cement concrete (PCC) to resurface existing pavements can be traced to as early as 1913. A relatively low-maintenance service life of 20 years can be expected, and many resurfacings have provided 30 to 40 years of service. Although used in practically every state, portland cement concrete resurfacings have not been used as widely as asphalt concrete resurfacings because of higher initial cost and construction complexity. However, several developments within the last 10 to 15 years have positively affected the suitability of concrete resurfacings as a viable rehabilitation technique. This synthesis presents a summary of the current concrete overlay design and construction practices, describes the current performance of concrete overlays, and discussed the effect of traffic delay assessment in the selection of the type of resurfacing.

Lokken, E. C. 1981. "Concrete Overlays for Concrete and Asphalt Pavements." *Proceedings, Second International Conference on Concrete Pavement Design.* Purdue University, West Lafayette, IN.

All types of concrete overlays are described in this paper. The performance of many of these projects is summarized from surveys conducted over a number of years. Based on the results of these surveys, it is concluded that concrete resurfacings can be built to last more than 20 years. This should result in lifecycle costs that are lower than those for resurfacings requiring multiple applications over the same time period and provide a higher lever of serviceability and minimum disruption to traffic during the life of the concrete resurfacing.

Jasienski, A., R. Debroux, F. Fuchs, and J. Lejeune. 1997. "Belgian Applications of Concrete Inlays on Motorways." ***Proceedings, Sixth International Purdue Conference on Concrete Pavement, Design and Materials for High Performance.*** **Indianapolis, IN.**

All motorway managers are faced with the problem of rutting in dense asphalt road surfacings, which is due among other things to the steady increase in heavy goods vehicle traffic. As a result, the residual service life to the slow (right-hand) lane decreases much faster than that of the overtaking (left-hand) lane, which is used mainly by light vehicles having a considerably milder action on road structures. It is, therefore, necessary to define specific and economically viable approaches to the structural strengthening of slow lanes. Since 1993, the so-called "inlay" technique has been applied on four sites in the Walloon Region of Belgium. This technique consists of replacing the dense asphalt surfacing of the slow lane with a cement concrete pavement. The first application took place near Mons in July 1993, over a 1,600 m stretch of the carriageway to Brussels of the Brussels-Paris motorway (E19). A second site was opened on the same motorway in August 1995, this time in the direction of Paris and over a total of 4,000 m. In October 1995 and July 1996, respectively, two further jobs were carried out in the direction of Aachen on the Brussels-Aachen motorway (E40), each over a total length of 2,000 m.

McGhee, K. H. 1994. ***Portland Cement Concrete Resurfacing.*** **NCHRP Synthesis of Highway Practice 204. Transportation Research Board, Washington, DC.**

This synthesis report will be of special interest to pavement designers, materials engineers, and others seeking information on portland cement concrete resurfacings (overlays) placed over both portland and asphalt cement concrete pavements. Information is presented on the various practices in use for the design, material selection, and construction techniques associated with each pavement type. Additional information is provided on resurfacing experience and performance, including an Appendix cataloging more than 700 existing resurfacing projects in North America. Transportation agencies in the United States are continuing to develop pavement management systems which take an objective and structured approach to life-cycle cost analysis requirements for pavement rehabilitation project analysis. This report of the Transportation Research Board also discusses the considerations involved in the selection of technically feasible resurfacing alternatives. Based on the longitudinal experience of 375 resurfacing projects that were cataloged in 1982 and the more than 700 projects identified in 1993, much useful information on the performance characteristics of portland cement concrete resurfacing is presented.

Pearson, A. 1990. "Concrete Overlay Technology and Construction." ***Proceedings,*** **Conference of the Australian Road Research Board. Australian Road Research Board, Nunawading, Australia.**

This paper is the result of a study tour of the United States of America to investigate and report on concrete overlays of both rigid and flexible pavements, from the technical and practical points of view. This study tour was carried out by the author in April/May 1989. The three distinct classes of concrete overlays, unbonded concrete overlays, partially bonded concrete overlays, bonded concrete overlays, are investigated from basic design principles through to practical construction implications. Conclusions are reached that pavement engineers now have another option in rehabilitation and reconstruction procedures. With the aging pavement network in Australia, concrete overlay technology and construction is a viable option that should be considered by today's pavement engineers.

Seeds, S. B., N. C. Jackson, D. G. Peshkin, K. D. Smith, and M. J. Wade. 1998. ***Techniques for Pavement Rehabilitation.*** **Sixth Edition. National Highway Institute, Arlington, VA.**

This training course notebook provides detailed information on the rehabilitation of pavement structures. It includes the concepts that assist pavement engineers in developing the most reliable and cost effective rehabilitation alternatives for both flexible and rigid pavements. The approach to determining an appropriate rehabilitation solution is presented, including existing pavement evaluation and condition assessment, development of rehabilitation alternatives, and selection of the most appropriate strategy. Individual rehabilitation techniques are described in detail for both flexible and rigid pavements.

Sriraman, S. and D. G. Zollinger. 1999. *Performance of Continuously Reinforced Concrete Pavements, Volume IV—Resurfacings for CRC Pavements.* FHWA-RD-98-100. Federal Highway Administration, McLean, VA.

This report is one of a series of reports prepared as part of a recent study sponsored by the Federal Highway Administration (FHWA) aimed at updating the state-of-the-art of the design, construction, maintenance, and rehabilitation of continuously reinforced concrete (CRC) pavements. The scope of work of the FHWA study included the following: (1) Conduct of a literature review and preparation of an annotated bibliography on CRC pavements and CRC overlays; (2) Conduct of a field investigation and laboratory testing related to 23 existing in-service pavement sections to evaluate the effect of various design features on CRC pavement performance, to identify any design or construction related problems, and to recommend procedures to improve CRC pavement technology; (3) Evaluation of the effectiveness of various maintenance and rehabilitation strategies for CRC pavements; and (4) Preparation of a Summary Report on the current state of the practice for CRC pavements. This report, Volume IV in the series, provides a framework for the design of CRC overlays of flexible pavements and jointed concrete pavements, and the design of overlays (bonded, unbonded, and HMA) of CRC pavements.

Tayabji, S. D. and P. A. Okamoto. 1985. "Thickness Design of Concrete Resurfacing." *Proceedings, Third International Conference on Concrete Pavement Design and Rehabilitation.* Purdue University, West Lafayette, IN.

The four basic types of concrete overlays are presented in this paper. New thickness design procedures are presented for both bonded and nonbonded resurfacings. The design procedure is based upon the resurfaced structure providing a thickness equivalent to that needed for new construction in light of the anticipated traffic.

Voigt, G. F., S. H. Carpenter, and M. I. Darter. 1989. *Rehabilitation of Concrete Pavements, Volume II—Overlay Rehabilitation Techniques.* Report No. FHWA-RD-88-072. Federal Highway Administration, McLean, VA.

Extensive field, laboratory, and analytical studies were conducted into the evaluation and rehabilitation of concrete pavements. Field studies included over 350 rehabilitated pavement sections throughout the U.S., and the construction of the two field experiments. A laboratory study was conducted on anchoring dowel bars in full-depth repairs. Analyses of field and laboratory data identified performance characteristics, improved design and construction procedures, and provided deterioration models for rehabilitated pavements. A concrete pavement advisory system was developed to assist engineers in project level evaluation and rehabilitation. This volume presents a summary of the performance of various overlay techniques, including bonded concrete overlays, unbonded concrete overlays, and cracked and seated pavements with asphalt concrete overlays.

Whiting, D., A. Todres, M. Nagi, T. Yu, D. Peshkin, M. Darter, J. Holm, M. Andersen, and M. Geiker. 1993. *Synthesis of Current and Projected Concrete Highway Technology.* SHRP Report SHRP-C-345. Strategic Highway Research Program, Washington, DC.

This synthesis summarized the results of an extensive search and review of available literature in the field of concrete materials, construction practices, and major application areas as applied to highway construction technology. The syntheses covers current and projected developments in materials systems, including cements, aggregates, admixtures, fibers, and sealers. General topic areas in the fields of concrete production and highway construction covered by this synthesis include mix proportioning, batching and transport, placement, finishing, and curing.

The synthesis includes information on specific applications areas in the highway industry. These applications focus on repair and reconstruction and include full-depth repairs, slab replacement, partial-depth repairs, overlays, and recycling. Quality control of concrete, including traditional approaches as well as new test methods and quality assurance schemes, is also discussed in detail.

Bonded PCC Overlays

Alkier, K. W. and W. V. Ward. 1991. *Final Construction Report: Experimental Thin Bonded Concrete Overlay Pavement in Houston, Texas.* Report No. TX-91/561-1F. Texas Department of Transportation, Austin, TX.

An experimental thin bonded continuous concrete pavement was constructed in Houston, Texas, July–August 1983. It was monitored until 1990. The final evaluations of this TBCO test section in March and December of 1990 seem to confirm what earlier findings reported; the pavement is in very good condition and is expected to continue to provide excellent service for the foreseeable future. After more than seven years of continued heavy traffic of about 140,000 vehicles per day, the overall condition and appearance seem identical to any other typical CRCP of the same age in the Houston area, made from the same material and subjected to the same traffic load. The overall useful life expectancy of this TBCO test section is judged to be about 15 to 20 years from date of construction in 1983. After that length of time, increasing transverse and longitudinal pavement cracking will combine to form blocks and punchouts of varying sizes, leading eventually to the need for extensive repairs and maintenance.

Allison, B. T., B. F. McCullough, and D. W. Fowler. 1993. *Feasibility Study for a Full-Scale Bonded Concrete Overlay on IH-10 in El Paso, Texas.* Research Report 1957-1F. Texas Department of Transportation, Austin, TX.

This report outlines the research and recommendations concerning the rehabilitation of IH-10 through the downtown area of El Paso. The project of isolating an appropriate rehabilitation method was broken into three tasks. Task 1 included collecting background information such as traffic data and environmental information, documenting current pavement conditions, and determining District 24's long-term objectives. Task 2 included using the background information collected to consider the various methods of rehabilitation that would be appropriate to use in the downtown area of El Paso. Task 3 included developing a preliminary set of bonded concrete overlay design plans that are cost-effective and meet the district's needs for a long-term rehabilitation plan.

American Concrete Pavement Association (ACPA). 1990. *Guidelines for Bonded Concrete Overlays.* Technical Bulletin TB-007P. American Concrete Pavement Association, Arlington Heights, IL.

These guidelines cover the overlay of existing portland cement concrete (PCC) pavements with a bonded concrete overlay. This involves the placement of a thin concrete layer atop the existing surface to form a monolithic section. A monolithic section improves load carrying capacity and provides a new surface for improved rideability, skid resistance and lighting characteristics.

Anderson, C. 1993. *Non-Grouted Bonded PCC Overlay, City of Oskaloosa.* Report No. HR-528. Iowa Department of Transportation, Ames, IA.

Based upon the success the Iowa Department of Transportation has had using thin bonded, low slump, dense portland cement concrete on bridge decks for rehabilitation, it was decided to pursue research in the area of bonded portland cement concrete resurfacing of pavements. Since that time, in an effort to reduce costs, research was conducted into eliminating the grouting operation. On this project a non-grouted overlay was used to modernize an existing urban street. This research project is located in the City of Oskaloosa on 11th Avenue from South M Street to South Market Street. Construction of the project went well and the non-grouted overlay has performed very well to date.

Bagate, M., B. F. McCullough, and D. Fowler. 1985. "Construction and Performance of an Experimental Thin-Bonded Concrete Overlay Pavement in Houston." *Transportation Research Record 1040.* Transportation Research Board, Washington, DC.

The use of thin-bonded concrete overlay (TBCO) pavements is rapidly emerging as a viable means of rehabilitating concrete pavements. During the summer of 1983, an experimental 1000-ft section of thin-bonded concrete overlay pavement was placed on I-610, a 4-lane divided freeway in Houston, TX. Five design sections were constructed. Concrete reinforcement and overlay thickness were used at three and two levels, respectively. Presented in this paper are several aspects of the experimental project, including the experimental design, project specifications, construction, and initial performance results.

Bagate, M., B. F. McCullough, and D. W. Fowler. 1987. A *Mechanistic Design for Thin-Bonded Concrete Overlay Pavements*. Research Report 457-3. Texas State Department of Highways and Public Transportation, Austin, TX.

This report is concerned with the design of concrete overlays of old concrete pavements with some remaining fatigue life considering three criteria: 1) wheel load stresses, 2) volume change stresses, and 3) interface bond stresses. The finite element method is used for the wheel load stresses and accounts for a more precise modeling of continuously reinforced concrete pavements, jointed reinforced concrete pavements, and jointed concrete pavements with various loading configurations. A computer program is presented which performs the required structural analysis. The program has been verified and calibrated using field date from the bonded overlay project on I-610 in Houston.

Bergren, J. W. 1981. "Bonded Portland Cement Concrete Resurfacing." *Transportation Research Record 814*. Transportation Research Board, Washington, DC.

The experience of Iowa in developing and refining the procedures involved in bonded concrete overlay construction are presented in this paper. The methods of evaluating the condition of the underlying pavement and determining the resurfacing layer thickness are discussed. Several projects utilizing portland cement concrete resurfacing to satisfy different roadway needs are described. Several methods of surface preparation, bonding methods, and the bond test results are included and discussed. It is concluded that bonding a 2 to 3 in (51 to 76 mm) layer of portland cement concrete to an existing concrete pavement is a viable alternative to bituminous resurfacing for the rehabilitation and restoration of concrete pavements.

Betterton, R. M., M. J. Knutson, and V. J. Marks. 1985. "Fibrous Portland Cement Concrete Overlay Research in Greene County, IA." *Transportation Research Record 1040*. Transportation Research Board, Washington, DC.

This project was constructed in October 1973 to evaluate the performance of steel fiber-reinforced concrete (fibrous concrete). The 33 fibrous concrete sections, four continuously reinforced concrete sections, two mesh-reinforced concrete sections, and two sections with transverse reinforcing were rated relative to each other on a scale of 0 to 100 at ages of 5 and 10 years. All sections are essentially unbonded or debonded from the underlying slab. All experimental overlay sections experienced only limited additional deteriorated in the 5 to 10 year period. The 4 in thick, nonfibrous, mesh, continuously reinforced concrete pavement overlay sections provided the best performance in this research project. A nonfibrous, 5-in thick, transverse bar-reinforced overlay section with no longitudinal steel performed almost as well. The best performance of a fiber-reinforced concrete section was obtained with fiber quantities of 160 lb per yd^3 of concrete. In general, the thicker, nonfibrous pavement overlay sections constructed at a lower unit cost than the fibrous sections performed better than the fiber-reinforced concrete overlays.

Cable, J. K. 1995. *Bond Development in Concrete Overlays*. Report No. HR-561. Iowa Department of Transportation, Ames, IA.

Data collection to determine the rate of bond strength development between concrete overlays and existing pavements and the evaluation of nondestructive testing methods for determining concrete strength were the objectives of this study. Maturity meters and pulse velocity meters were employed to determine the rate of flexural strength gain and determine the time for opening of newly constructed pavements to traffic. Maturity measurements appear to provide a less destructive method of testing. Pulse velocity measurements do require care in the preparation of the test wells and operator care in testing. Both devices functioned well under adverse weather and construction conditions and can reduce construction traffic delay decisions. Deflection testing and strain gaging indicate differences in the reaction of the overlay and existing pavement under grouting versus nongrouted sections. Grouting did enhance the rate of bond development with Type III cement outperforming the Type II grout section. Type III and Type II cement grouts enhanced resistance to cracking in uniformly supported pavements where joints are prepared prior to overlays achieving target flexural strengths. Torsional and direct shear testing provide additional ways of measuring bond development at different cure times.

Calvert, G., O. J. Lane, and C. Anderson. 1990. *Thin Bonded Concrete Overlay With Fast Track Concrete.* Final Report, Project HR-531. Iowa Department of Transportation, Ames, IA.

Pavements have been overlaid with thin bonded portland cement concrete (PCC) for several years. These projects have had traffic detoured for a period of 5-10 days. These detours are unacceptable to the traveling public and result in severe criticism. The use of thin bonded fast track overlay was promoted to allow a thin bonded PCC overlay with minimal disruption of local traffic. This project demonstrated the concept of using one lane of the roadway to maintain traffic while the overlay was placed on the other and then, with the rapid strength gain of the fast track concrete, the construction and local traffic is maintained on the newly placed, thin bonded overlay. The goals of this project were: (1) Traffic usage immediately after placement and finishing; (2) Reduce traffic disruption on a single lane to less than 5 hours; (3) Reduce traffic disruption on a given section of two-lane roadway to less than 2 days; (4) The procedure must be economically viable and competitive with existing alternatives; (5) Design life for new construction equivalent to or in excess of conventional pavements; and (6) A 20 year minimum design life for rehabilitated pavements.

Chanvillard, G., P. C. Aitcin, and C. Lupien. 1989. "Field Evaluation of Steel Fiber Reinforced Concrete Overlay With Various Bonding Mechanisms." *Transportation Research Record 1226.* Transportation Research Board, Washington, DC.

An experimental rehabilitation project was conducted on the Transcanadian Highway where old concrete pavement was re-covered with a thin, steel fiber reinforced concrete overlay. In this project, 18 different construction conditions were investigated. The surface of the old pavement was either sandblasted or scarified. Three different types of steel fibers were used, and all the overlay was bonded with a thin cement grout. In addition, two lanes were repaired with some mechanical bonding provided by 37.5-mm (1 ½-in.) steel nails. The most significant results obtained during the construction period as well as all data recorded over the succeeding two winters are reported and analyzed in this paper.

Choi, D. U., D. W. Fowler, and J. O. Jirsa. 1999. "Interface Shear Strength of Concrete at Early Ages." *ACI Structural Journal*, Volume 96, No. 3. American Concrete Institute, Farmington Hills, MI.

Bonded concrete overlays have been used to rehabilitate concrete pavements, bridge decks, and slabs. The development of interface shear strength at early ages following placement of new concrete overlaying existing concrete was studied experimentally. Thirty-two pushoff tests were conducted to determine interface strength. Test variables were age of overlay and overlay curing method. Special powder-driven shear connectors were used across the interface in most specimens to provide some reinforcement between old and new concrete layers. Tests were performed when the age of the overlays was between 12 hr and 35 days. Test results showed that the interface shear strength was 1,370 kPa or higher 24 hr after casting, and strength improved rapidly during the first 3 days. The interface strength was influenced significantly by the overlay curing method. Test results suggested that good curing of the overlays at early ages, especially during the first 3 days when the interface strength is increasing rapidly, resulted in higher interface shear strengths.

Darter, M. I. and E. J. Barenberg. 1980. *Bonded Concrete Overlays: Construction and Performance.* Report No. GL-80-11. U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

This report summarizes the industry experience and current state of the art of bonded concrete overlays. This type of overlay has been constructed since the early 1900s. A review and summary of surface preparation methods, joint and crack treatments, bonding methods, concrete overlay mixtures, curing methods, jointing techniques, performance of overlays to date, and the use of reinforcement in the overlays is presented. A list of conclusions and future research needs is also offered.

Delatte, N. J. 1996. *High Early Strength Bonded Concrete Overlay Designs and Construction Methods for Rehabilitation of CRCP.* Ph.D. Dissertation. University of Texas, Austin, TX.

For rehabilitation of concrete pavements, resurfacing with a bonded concrete overlay (BCO) may provide significantly longer life and reduced maintenance costs. Two important issues are bonding and rapid reopening of resurfaced sections. The project objective was to determine a method or methods for constructing a bonded concrete overlay under extreme weather conditions in El Paso, Texas, that would ensure early opening to traffic and achieve design requirements for long-term performance, and to investigate methods of detecting and mapping delaminations nondestructively. A BCO test slab was designed and constructed for a full-scale test of feasible design and construction alternatives. The month of June was selected for the test because severe environmental conditions were expected. The experimental variables encompassed in eight test sections were reinforcement of concrete (plain, polypropylene fiber reinforced, and steel fiber reinforced), use of shear connectors (nails and anchors), reinforcement, surface preparation, and day or night construction. A weather station was used to record air temperature, relative humidity, and wind speed. By combining these data with concrete temperatures, evaporation rates could be calculated. Several nondestructive testing methods were used to detect and map delaminations. The results developed from the test are presented in terms of observations during construction, weather and slab temperatures, coring and pull-off testing, delamination detection, cracking, and slab loading. The early-age behavior and long-term performance of BCOs is reviewed, along with materials selection and mix design criteria. Recommendations are made for construction and quality control of BCOs for early opening to traffic.

Delatte, N. J. 1999. "Interface Stresses and Bond Strength in Bonded Concrete Overlays." *Preprint Paper No. 99-0534.* 78th Annual Meeting of the Transportation Research Board, Washington, DC.

Bonded concrete overlays (BCOs) have been used to rehabilitate concrete highway and airfield pavements for a number of years. A properly designed and constructed BCO can add considerable life to an existing pavement, by taking advantage of the remaining structural capacity of the original pavement. This will only occur, however, if the BCO is of sufficient thickness and is well bonded to the original pavement. The objective of a BCO is to increase the pavement thickness and stiffness, and to reduce tensile stresses in the pavement, thus increasing fatigue life. The design assumption is that the BCO bonds perfectly with the original pavement, producing a monolithic structure. Without bond, there is very little structural benefit from an overlay. Laboratory testing at the University of Texas at Austin and field testing in El Paso, Texas provided considerable data on the development of bond strength in overlays and in patterns of debonding. Subsequently, finite element modeling was used to calculate bond stresses caused by the volumetric contraction of the overlay due to shrinkage and thermal effects. The tensile stresses developed in the overlay by this contraction were also compared to the tensile strengths measured for several repair materials during testing at the U.S. Army Corps of Engineers Waterways Experiment Station. Good correlation was noted between high tensile stresses predicted by the model and extensive cracking observed in the repair materials.

Delatte, N. 2001. "High Performance Concrete for Bonded Pavement Overlay Applications." *Proceedings,* Second International Symposium on Maintenance and Rehabilitation of Pavements and Technological Control. Auburn, AL.

Deteriorating asphalt and concrete pavement infrastructure worldwide demands innovative and economical rehabilitation solutions. A properly designed and constructed bonded overlay can add considerable life to an existing pavement, by taking advantage of the remaining structural capacity of the original pavement. For bonded concrete overlays on existing concrete pavement and ultra-thin whitetopping on asphalt pavement, characteristics of the overlay concrete have important implications for early age behavior and long-term performance. Bond strength and resistance to cracking are important for overlay performance. In many cases these overlays are constructed on heavily traveled pavements, making early opening to traffic important. Therefore, early strength development without compromising durability is necessary. Satisfactory performance will only occur if the overlay is of sufficient thickness and is well bonded to the original pavement. The design assumption is that the overlay bonds perfectly with the original pavement, producing a monolithic structure. The tensile stresses developed in the overlay by this contraction may induce cracking in the overlay, as well as loss of bond. Without bond, there is very little structural benefit from an overlay, and the overlay may break apart rapidly under heavy traffic. Significant stresses may be caused by the volumetric contraction of the overlay due to shrinkage and thermal effects. Laboratory and field research for a bonded overlay on an interstate highway in El Paso, Texas has provided insight into design of high performance concrete for these applications. This paper reviews experimental results as well as current best practices to develop recommendations for high performance concrete materials selection and mixture proportioning for bonded overlay applications.

Delatte, N. J., D. W. Fowler, and B. F. McCullough. 1996a. *High Early Strength Bonded Concrete Overlay Designs and Construction Methods for Rehabilitation of CRCP*. Research Report 2911-4, Texas Department of Transportation, Austin, TX.

For rehabilitation of concrete pavements, resurfacing with a bonded concrete overlay (BCO) may provide significantly longer life and reduced maintenance costs. Two important issues are bonding and rapid reopening of resurfaced sections. The project objective was to determine a method or methods for constructing a bonded concrete overlay under extreme weather conditions in El Paso, Texas, that would ensure early opening to traffic and achieve design requirements for long-term performance, and to investigate methods of detecting and mapping delaminations nondestructively. A BCO test slab was designed and constructed for a full-scale test of feasible design and construction alternatives. The month of June was selected for the test because severe environmental conditions were expected. The experimental variables encompassed in eight test sections were reinforcement of concrete (plain, polypropylene fiber reinforced, and steel fiber reinforced), use of shear connectors (nails and anchors), reinforcement, surface preparation, and day or night construction. A weather station was used to record air temperature, relative humidity, and wind speed. By combining these data with concrete temperatures, evaporation rates could be calculated. Several nondestructive testing methods were used to detect and map delaminations. The results developed from the test are presented in terms of observations during construction, weather and slab temperatures, coring and pull-off testing, delamination detection, cracking, and slab loading. The early-age behavior and long-term performance of BCOs is reviewed, along with materials selection and mix design criteria. Recommendations are made for construction and quality control of BCOs for early opening to traffic.

Delatte, N. J., D. W. Fowler, and B. F. McCullough. 1996b. "Full-Scale Test of High Early Strength Bonded Concrete Overlay Design and Construction Methods." *Transportation Research Record 1544*. Transportation Research Board, Washington, DC.

For rehabilitation of concrete pavements, resurfacing with a bonded concrete overlay (BCO) may provide significantly longer life and reduced maintenance costs. Two important issues are bonding and rapid reopening of resurfaced sections. The project objectives were to determine a method of constructing a BCO under extreme weather conditions in El Paso, Texas, that would ensure early opening to traffic and achieve design requirements for long-term performance, and to investigate methods of detecting and mapping delaminations nondestructively. A BCO test slab that was designed and constructed for a full-scale test of feasible design and construction alternatives is reported. The month of June was selected for the test because severe environmental conditions were expected. The experimental variables encompassed in eight test sections were reinforcement of concrete (plain, polypropylene fiber-reinforced, and steel fiber-reinforced), use of shear connectors (nails and anchors), reinforcement, surface preparation, and day or night construction. A weather station was used to record air temperature, relative humidity, and wind speed. By combining these data with concrete temperatures, evaporation rates could be calculated. Several nondestructive testing methods were used to attempt to detect and map delaminations. The results developed from the test are presented in terms of observations during construction, weather and slab temperatures, coring and pull-off testing, delamination detection, cracking, and slab loading. Recommendations are made for construction and quality control of BCOs for early opening to traffic.

Delatte, N. J., D. W. Fowler, and B. F. McCullough. 1997. "Criteria for Opening Expedited Bonded Concrete Overlays to Traffic." *Transportation Research Record 1574*. Transportation Research Board, Washington, DC.

For rehabilitation of concrete pavements, resurfacing with a bonded concrete overlay (BCO) may provide significantly longer life and reduced maintenance costs. Two important issues to consider in rehabilitation are bonding and rapid reopening of resurfaced sections. The purpose of accelerated or expedited concrete paving is to limit the duration of lane closure and inconvenience to the public. Expedited BCOs offer an economical method for substantially extending rigid pavement life. Research for expedited BCOs in El Paso and Fort Worth, Texas, has been carried out for the Texas Department of Transportation by the Center for Transportation Research at the University of Texas. Results of previous expedited BCO construction are reviewed. Laboratory testing for this project included a high-early-strength mix design, bond development of that mix design, and early-age fatigue strength of half-scale BCO models. A 122-m-long test strip was cast with eight different expedited BCO designs, and accelerated traffic loading was imposed at 12 hr. Recommendations are made for construction and quality control of BCOs for early opening to traffic.

Delatte, N. J., D. W. Fowler, B. F. McCullough, and S. F. Gräter. 1998. "Investigating Performance of Bonded Concrete Overlays." ***Journal of Performance of Constructed Facilities*****, Volume 12, No. 2. American Society of Civil Engineers, New York, NY.**

A bonded concrete overlay (BCO) is a concrete pavement rehabilitation method used to extend the life of an existing concrete pavement. The BCO should bond fully with the existing concrete, leading to a thicker composite pavement section, a much stiffer pavement, and a considerable decrease in pavement stresses. For one project, cost estimates for a BCO were half as much as for full-depth replacement of a pavement. In some cases, BCOs have delaminated shortly after construction. This paper proposes a framework for identifying the causes of early age delamination in BCOs. The early age behavior of newly constructed BCOs is examined. The factors affecting the long-term performance of the BCO are the quality of the surface preparation, the materials used in the BCO, and the curing of the BCO. Weather monitoring during BCO construction is recommended. Methods of detecting and mapping delaminations are discussed. Several BCO delamination case studies are analyzed using this framework. The model is useful not only for investigating BCO performance but also for understanding and preparing BCO construction specifications.

Delatte, N. J., S. F. Gräter, M. Treviño-Frias, D. W. Fowler, and B. F. McCullough. 1996. ***Partial Construction Report of a Bonded Concrete Overlay on IH-10, El Paso,*** **and** ***Guide for Expedited Bonded Concrete Overlay Design and Construction.*** **Research Report 2911-5F. Texas Department of Transportation, Austin, TX.**

Expedited bonded concrete overlays offer an economical alternative for rehabilitating concrete pavements while minimizing user costs. The construction of a bonded concrete overlay in El Paso has provided the opportunity to research pavement design, mix design, construction methods and specification development for future overlay construction. Valuable information collected during the initial construction of the bonded concrete overlay on IH-10 in El Paso is reported. Although this project did not proceed as an expedited overlay, it was planned and researched as such. Unfortunately, a combination of factors led to some areas of the overlay to become unbonded. The causes of these delaminations that occurred during this construction are identified and recommendations for future overlay construction are made. Project selection, design, construction, and quality control are addressed in this guide for expedited bonded concrete overlays. Scheduling construction to avoid marginal or severe environmental conditions is also addressed. The methods presented in this report may be used to construct bonded concrete overlays which can be opened to traffic twelve to 24 hours after concrete placement.

Delatte, N. J. and J. T. Laird. 1999. "Performance of Bonded Concrete Overlays." ***Preprint Paper No. 99-0715.*** **78th Annual Meeting of the Transportation Research Board, Washington, DC.**

A bonded concrete overlay (BCO) is a concrete pavement rehabilitation method used to extend the life of an existing concrete pavement. The BCO should bond fully with the existing concrete, leading to a thicker composite pavement section, a much stiffer pavement, and a considerable decrease in pavement stresses. For one project, cost estimates for a BCO were half as much as for full-depth replacement of a pavement. In some cases BCOs have debonded shortly after construction. If this occurs, the design assumptions are violated and the increase in pavement life may not be achieved. This paper discusses some of the causes of early age debonding in BCOs. The factors affecting the long-term performance of the BCO include the quality of the surface preparation, the materials used in the BCO, and the curing of the BCO. Research conducted by the University of Texas at Austin has led to recommendations for quality control to ensure satisfactory performance. Weather monitoring during BCO construction is recommended to identify time periods when weather conditions threaten bond development, and construction should be halted. Some methods of detecting and mapping debonding are discussed. The recommendations are used to analyze case studies of BCO. The lessons learned are useful not only for investigating BCO performance but also for understanding and preparing BCO construction specifications.

Delatte, N. J., D. M. Wade, and D. W. Fowler. 2000. "Laboratory and Field Testing of Concrete Bond Development for Expedited Bonded Concrete Overlays." ***ACI Materials Journal*****, Volume 97, No. 3. American Concrete Institute, Farmington Hills, MI.**

In preparation for construction of a 0.8 km bonded concrete overlay (BCO) on IH-10 in downtown El Paso, Texas, the Center for Transportation Research at The University of Texas at Austin carried out an extensive laboratory and field testing program for the Texas Department of Transportation (TxDOT). The purpose of the testing program was to develop an appropriate high-early-strength concrete mixture proportioning for BCO construction, investigate its strength gain and bond development characteristics, and develop appropriate construction specification recommendations for TxDOT. The heavy traffic on IH-10 made it imperative to develop materials and methods that would allow rapid opening to traffic. Part of the laboratory testing program is documented in this paper. Among laboratory test methods investigated, the guillotine test was found to be useful for determining bond strength, with some difficulties in aligning the bond plane in the testing device. Because of the difficulty of extracting field samples for guillotine testing, however, the pulloff test method was used instead. The high-early-strength concrete tested was shown to be suitable for bonded concrete overlay construction.

Delatte, N. J., M. S. Williamson, and D. W. Fowler. 2000. "Bond Strength Development with Maturity of High-Early-Strength Bonded Concrete Overlays." ***ACI Materials Journal*****, Volume 97, No. 2. American Concrete Institute, Farmington Hills, MI.**

The development of concrete-to-concrete bond is important for the performance of bonded concrete overlays as well as for bridge deck overlays. Bond development is important for repairs of concrete structures with cementitious repair materials. The test methods used, along with their difficulty and relative reliability, will also be of interest to other researchers investigating concrete-to-concrete bond. Methods presented in this paper may be used to estimate the bond development between a bonded concrete overlay and its underlying substrate at early ages on the basis of concrete maturity. Compressive, splitting tensile, tension bond, and shear bond strength development of concrete are related to mixture proportioning and curing temperature. For a given concrete, they may be predicted by the maturity method, provided curing is adequate and that the effects of other variables can be controlled. Shear bond strength was found to be approximately twice the value of tension bond strength.

Freeman, T. E. 1996. ***Evaluation of a Thin-Bonded Portland Cement Concrete Pavement Overlay.*** **VTRC 97-R9. Virginia Department of Transportation, Charlottesville, VA.**

This report discusses the performance of the Virginia Department of Transportation's first modern rehabilitation project involving a thin-bonded portland cement concrete overlay of an existing jointed concrete pavement. The performance of the rigid overlay, which was constructed in a fast-track mode to minimize lane closure time, was evaluated by detailed condition surveys conducted annually throughout a 6-year analysis period to identify, document, and monitor the occurrence of distress. The roughness of the overlay was also measured annually with an accelerometer-based inertial road profiler to permit an examination of the effects of surface deterioration on ride quality. After 6 full years of service, which included only minimal maintenance, the pavement overlay remained in good overall condition. Although the ride quality of the overlay remained virtually unchanged throughout the period, a significant increase in the occurrence of low-to-moderate-severity joint spalls, corner breaks, and to a lesser extent transverse cracks were noted during the fifth and sixth years. The extrusion of compression seals and the subsequent infiltration of water into the pavement structure probably contributed to the observed localized failure of the overlay/substrate bond in the vicinity of joints. This condition, in turn, weakened the pavement's structural capacity at panel edges and thereby resulted in the formation of corner breaks and cracks parallel with and near transverse joints. The consideration of thin-bonded concrete overlays constructed in a fast-track mode is recommended as a viable rehabilitation alternative for jointed concrete pavements that are not severely distressed. However, careful attention to joint installation and, in particular, joint maintenance is recommended for similar future rehabilitation projects.

Gausman, R. H. 1986. *Status of Thin-Bonded Overlay.* FHWA-EP-2-1. Federal Highway Administration, Washington, DC.

The Demonstration Projects Division of the FHWA has been involved in the evaluation of a number of thin-bonded overlays since 1979. This report presents the status of those projects and includes a summary of some of the more important findings to date. Included in the findings are: 1) Overlays can be effectively bonded to a clean pavement in good condition, 2) Performance to date of overlays on pavements in marginal condition has been good, 3) Early tests to eliminate bonding grout and retain bond strengths have been positive, 4) Placement of thin-bonded overlays in very hot weather is not recommended, 5) For proper performance, quality surface preparation is critical, 6) If pressure relief joints are to be included, close attention must be paid to their design, and 7) Costs are somewhat high, but represent first attempts and should be lower as more projects are built. General preparation and construction date, as well as a brief performance evaluation, are presented for project in Texas, New York, Louisiana, California, Wyoming, Iowa, Georgia, and South Dakota.

Glauz, D. L. 1995. "A Latex-Modified Concrete Overlay on Plain-Jointed Concrete Pavement." *Cement, Concrete and Aggregates*, Volume 17, No. 2. American Society for Testing and Materials Committee, Philadelphia, PA.

A latex-modified concrete overlay was applied over Interstate 5 in Northern California at a Strategic Highway Research Program Specific Pavement Studies (SPS) test site. The objective was to assess a potentially lower cost latex overlay system. The cost of the overlay was to be reduced by eliminating surface prewetting and the wet burlap cure. The test section was placed 300-m long across three lanes. Unique features of the overlay were: 1) it was placed on dry substrate, 2) latex content was 43 percent higher than conventional practice, 3) cure was by curing compound rather than wet burlap, and 4) it was on a plain jointed portland cement concrete pavement. Bond to the substrate was very good. Curing compound provided a sufficient curing membrane when applied at the proper rate. The system was evaluated yearly. To date, the only problem has been at the joints, which present a serious problem for bonded overlays. Delaminations or spalls have occurred often at the transverse joints. The objective was met although the system is inappropriate for use on jointed pavements. A latex-modified concrete overlay can be placed on a dry substrate and bond well. It can be cured successfully with a curing compound that conforms to American Society for Testing and Materials C 309. This type of overlay is unsuitable for jointed pavements, though, unless careful attention is given to the joint detail.

Halm, H. J. 1981. "Bonded Concrete Resurfacing." *Proceedings, Second International Conference on Concrete Pavement Design.* Purdue University, West Lafayette, IN.

This paper details the history of thin bonded concrete overlays, which date back to 1913. Beginning in the 1970s, Iowa began a rigorous program of research which is described herein. The projects have demonstrated that concrete resurfacings are a viable and economical approach to the rehabilitation of concrete pavements.

Harris, G. 1992. *Performance of a Nongrouted Thin Bonded PCC Overlay.* Report No. HR-291. Iowa Department of Transportation, Ames, IA.

The use of a thin bonded concrete overlay atop an older surface has been widely incorporated for pavement rehabilitation in Iowa since the early 70s. Two test sections were constructed in 1985 on county road T61 on the Monroe-Wapello County line without the use of grout as a bonding agent to determine if adequate bond could be achieved and structural capacity uncompromised. Both test sections have performed well with one section having higher bond strengths, lower roughness values, higher structural capacity, and less debonding at the joints than the other section. Overall, both ungrouted sections have performed well under substantial truck traffic with minimal surface distress. More attention should be given, however, to rectifying apparent debonding at the joints when no grout is used as a bonding agent.

Hubbard, T. and G. Williams. 1999. *Evaluation of Bonded Overlay and Dowel Bar Retrofit on I-40.* Construction Report. Oklahoma Department of Transportation, Oklahoma City, OK.

The Oklahoma Department of Transportation (ODOT) has completed a rehabilitation project on a 1000-ft long section of existing jointed PCC pavement. The section is located in the westbound lanes of I-40 in Canadian County. Work done on the rehabilitated section consisted of retrofitting load transfer devices (dowel bars with expansion caps) between the slabs and placing a thin, bonded fiber-reinforced PCC overlay. The rehabilitation done on this project has the potential for restoring ride quality, improving load transfer between slabs, and improving surface friction characteristics using a faster, less labor-intensive process. The completed overlay will be evaluated with the goal of projecting the long-term performance of the rehabilitated section.

Huddleston, J. L., D. W. Fowler, and B. F. McCullough. 1995. *Effects of Early Traffic Loading on a Bonded Concrete Overlay.* Research Report 2911-3. Texas Department of Transportation, Austin, TX.

The bonded concrete overlay (BCO) is emerging as a method of pavement repair which minimizes costs and traffic disruption. When existing methods of expedited paving are combined with rapid-setting concrete, the potential exists to return traffic to a pavement within 24 hours of closure. In order to determine the long-term effects of early traffic loading on the IH-10 BCO in El Paso, Texas, a fatigue study of BCO beams was conducted. Seven beam specimens, with overlays ranging from 12 hours to 7 days of age, were tested. The progression of beam deflections and cracking were monitored throughout each test to determine trends in fatigue damage with progressively younger overlays. Results indicate that rapid strength and stiffness gain in the overlay concrete minimize fatigue damage in young BCOs. Final deflections and cracking for beams loaded at twelve hours compared favorably with fully cured beams, indicating that traffic application at early ages is not detrimental to long-term pavement behavior.

Ibukiyama, S., S. Kokubun, and K. Ishikawa. 1989. "Introduction of Recent Thin Bonded Concrete Overlay Construction and Evaluation of Those Performances in Japan." *Proceedings, Fourth International Conference of Concrete Pavement Design and Rehabilitation.* Purdue University, West Lafayette, IN.

The concrete overlay paving results in Japan have become successful through the technical development of the steel fiber reinforced concrete (hereinafter referred to as SFRC) overlay, which was started in the mid 1970's. Typical examples are overlays for worn down asphalt paved surface and overlays for deteriorated concrete slabs on bridges. In the past, adapting concrete overlay for use on concrete pavement was not considered possible because of low performance road planer and insufficient bond with existing slabs. However, recently, a heavy duty road planer has been developed and materials and construction methods for overlays have improved, therefore, construction of concrete overlay has become practical and possible on concrete pavement. Therefore, the yearly road repair losses caused by using asphalt materials in cold weather regions have been eliminated, along with it now being possible to economically reclaim a durable road surface. This paper presents a recent example of the application of thin bonded concrete overly (hereinafter referred to as TBCO) onto jointed Portland cement concrete pavement (hereinafter referred to as PCCP) and continuously reinforced concrete pavement (hereinafter refereed to as CRCP) and reports on these results in Japan.

Johnson, M. L. 1980. *Bonded, Thin-Lift, Non-Reinforced Portland Cement Concrete Resurfacing.* Final Report, Project HR-191. Iowa Department of Transportation, Ames, IA.

A research project involving the construction of 2,3,4, and 5 in (51,76, 102, and 127 mm) of bonded portland cement concrete overlay was carried out in 1977 in Clayton County, Iowa. The sections on this 1.3 mile (2.1 km) project were constructed using a variety of reinforcements, surface preparation, concrete water reducing agents, and joint sawing. At the time of the report, the pavement was performing very well. Extensive test data from the different sections and observations regarding the different variables are summarized in this report.

Kaler, M. K., J. Lane, and M. L. Johnson. 1986. *Performance of Nongrouted Thin Bonded PCC Overlays.* Construction Report, Project HR-291. Iowa Department of Transportation, Ames, IA.

In an effort to reduce the construction costs associated with thin bonded overlay techniques, elimination of the grouting operation has been proposed. Preliminary work with nongrouted successfully bonded overlays has included field trials on several pavement overlay projects. Additionally, a nongrouted section was an experimental feature of a project constructed in 1985. Preliminary results indicate that sufficient bond can be achieved on nongrouted bonded concrete overlays.

King, W. M. 1992. "Design and Construction of a Bonded Fiber Concrete Overlay of CRCP (Louisiana, Interstate Route 10, August 1990)." ***Report No. FHWA/LA-92/266.*** **Louisiana Department of Transportation, Baton Rouge, LA.**

The purpose of this study was to evaluate a bonded steel fiber reinforced concrete overlay on an existing 8-inch CRC pavement on Interstate 10 south of Baton Rouge, LA. The existing 16-year-old CRC pavement had carried twice its design load and contained only a few edge punch-out failures per mile. A 4-inch concrete overlay was designed for a 20-year service life. An additional level of reinforcement-bonding was provided which utilized curb type reinforcement bars epoxied into the existing slab. The primary purpose in the additional reinforcement was to provide positive bonding at the slab edges where thin overlays have a tendency to debond to curling and/or warping. A 9-inch tied concrete shoulder was added to increase the pavement's structural capacity.
The overall Serviceability Index of the pavement increased from 3.4 to 4.4 with measured Profile Index levels typically below the 5-inch per mile specification. Test revealed excellent bond strengths, and reduced edge deflections by 60 percent under a 22,000 pound moving single axle loading. Cores taken over transverse cracks in the overlay indicated reflection cracking from the transverse cracks in the original pavement. The final results reveal an estimated 35 percent of these cracks have reflected through and debonding has not occurred at the pavement edges. Anticipation of reflective cracking was one consideration in using the steel fibers which provide three-dimensional reinforcement.

King, W. M., W. H. Temple, and S. L. Cumbaa. 1992. "Design and Construction of a Bonded Fiber Concrete Overlay of CRCP (Louisiana, Interstate Route 10, August 1990)." ***Proceedings, Performance and Prevention of Deficiencies and Failures, 1992 Materials Engineering Conference.*** **American Society of Civil Engineers, New York, NY.**

The purpose of this study was the design and construction of a bonded steel fiber reinforced concrete overlay on an existing 8-inch CRC pavement on Interstate 10 south of Baton Rouge, LA. The existing sixteen year old CRC pavement, which is estimated to have carried twice its design load contained several edge punch-out failures per mile. The research objectives were to provide an overlay with a high probability for long term success by using a high strength concrete mix with internal reinforcement and good bonding characteristics. A 4-inch concrete overlay was designed to contain 7.5 sacks of cement and 85 pounds of steel fibers per cubic yard with locally available river gravel as coarse aggregate. An additional level of reinforcement-bonding was provided which utilized curb type reinforcement bars epoxied into the existing slab 8 inches from the outside edge. Water-cement grout was applied to the cleaned surface producing bond strengths in excess of 900 psi. The concrete overlay in combination with 9-inch tied concrete shoulders reduced edge deflections by 60 percent under a 22,000 pound moving single axle load applied 2 feet from the edge. An additional level of reinforcement bars epoxied into the existing slab 8-inches from each applied 2-feet from the edge. In general, the Serviceability Index of the pavement increased from 3.4 to 4.4 with measured Profile index levels typically below the 5-inch per mile specification. The bonded overlay has been in service since August 1990 and carries 41,000 ADT interstate traffic. Anticipation of reflective cracking was one consideration in using the steel fibers which provide three-dimensional reinforcement. Cost figures are provided for surface preparation and for placement of the overlay.

Koesno, K. and B. F. McCullough. 1987. ***Evaluation of the Performance of the Bonded Concrete Overlay on Interstate Highway 610 North, Houston, Texas.*** **Research Report 920-2. Texas State Department of Highways and Public Transportation, Austin, TX.**

The objective of the study was to evaluate the performance of the bonded concrete overlay project on IH 610 North in Houston and implement the findings in other studies on bonded concrete overlay. Field measurements were conducted periodically and laboratory testings were performed on the cores obtained from experimental sections. Then an assessment of overlay pavement life was made to arrive at conclusions and recommendations that would enable the Texas State Department of Highways and Public Transportation to design overlays for rehabilitation programs on CRCP.

Koesno, K. and B. F. McCullough. 1988. "Evaluation of the Performance of Bonded Concrete Overlay on Interstate Highway 610 North, Houston, Texas." ***Transportation Research Record 1196.*** **Transportation Research Board, Washington, DC.**

The objective of the study was to evaluate the performance of the bonded concrete overlay project on IH 610 North in Houston and implement the findings in other studies on bonded concrete overlay. The performance of the bonded concrete overlay was monitored on ten experimental sections selected from the 3-½ mi project and ranging from 400 to 600 feet long. Periodic field measurements were conducted, and an assessment of overlay pavement life was made. The resulting conclusions and recommendations were to be used by the Texas State Department of Highways and Public Transportation to design overlays for rehabilitation programs on continuous reinforced concrete pavement (CRCP).

Lane, O. J. 1987. ***Thin Bonded Concrete Overlay with Fast Track Concrete.*** **Condition and Performance Report, Project HR-531. Iowa Department of Transportation, Ames, IA.**

This report describes the construction of an overlay on U.S. 71 in Buena Vista County, Iowa in 1986. The work involved rehabilitation of an older 20-ft (6 m) wide pavement by placing a 4-in (102 mm) thick bonded concrete overlay monolithically with 2 ft (0.6 m) of widening on each side of the pavement. The work was performed one lane at a time to keep traffic flowing. At the time of this report, debonding was minor and it was concluded that a service life of 20 to 30 years could be expected for this project.

Lange, D. A. and H. C. Shin. 2001. "Early Age Stresses and Debonding in Bonded Concrete Overlays." ***Preprint Paper No. 01-0410.*** **80th Annual Meeting of the Transportation Research Board, Washington, DC.**

Bonded concrete overlays are a cost-effective strategy for rehabilitating and/or strengthening old concrete pavements. However, volume changes of the overlay concrete may cause debonding at the interface between old and new concrete or surface cracking. Experimental measurements and numerical analyses were pursued to understand the distribution of stress arising from thermal change or drying shrinkage under restrained conditions. Laboratory overlay specimens were made to measure opening displacement at the interface. Debonding profiles at the interface were determined using a dye technique. A finite element model was developed to study debonding behavior and crack tendency due to volume changes. Physical experiments were used to define nonlinear gradients of drying shrinkage and bond strengths for input into the model. The results serve as an illustration of how HPC mixtures may be problematic for overlay applications if full wet curing is not utilized.

Lau, C. M., T. F. Fwa, and P. Paramasivam. 1994. "Interface Shear Stress In Overlaid Concrete Pavements." ***Journal of Transportation Engineering*****, Volume 120, No. 2. American Society of Civil Engineers, New York, NY.**

Shear stresses at the interface between an existing concrete pavement and an overlay are calculated using the finite-element method for different loading conditions. The effects of vertical wheel loads, temperature loading, and wheel braking forces are considered. Perfect bonding between the overlay and the pavement is assumed. The pavement-overlay slab is modeled using thin plate elements for calculating the stresses due to the vertical wheel load and temperature gradient. In analysis for stresses due to wheel braking loads, the slab is modeled with solid elements. The results are checked against Cerrutti's closed-form solution in the case of horizontal wheel braking. The computed shear stresses at the pavement-overlay interface for typical values of pavement and overlay thicknesses, wheel loads, and thermal gradients are compared to reported values of shear strength of laboratory-prepared overlaid specimens. The shear stresses at the interface are found to be small in relation to the interfacial bond strength of laboratory-prepared overlaid specimens. The findings suggest that debonding in overlay construction in the field is likely to be caused by stress concentrations due to local defects.

Lundy, J. R. and B. F. McCullough. 1989. "Delamination in Bonded Concrete Overlays of Continuously Reinforced Pavement." ***Proceedings, Fourth International Conference of Concrete Pavement Design and Rehabilitation.*** **Purdue University, West Lafayette, IN.**

The Texas State Department of Highways and Public Transportation (SDHPT) has placed over 25 miles of bonded concrete overlays (BCO) in the Houston area. Following the successful construction of a 1000-ft test section in 1983, the SDHPT overlaid approximately 3.2 miles of an eight-lane freeway. Debonding of some areas were reported within 1 year of completion. After debonding was discovered in 1987, a study was initiated to determine the location, extent, and cause of the delaminations. Furthermore, the investigation was to determine if debonding was a progressive phenomenon. The survey results showed the use of portland cement grout as a bonding agent reduces the chance of debonding under conditions present in Houston. Also, the use of steel fiber reinforcement with siliceous river gravel or crushed limestone aggregate with welded wire fabric reduces the likelihood of delamination when compared to siliceous river gravel and welded wire fabric.

Lundy, J. R., B. F. McCullough, and D. W. Fowler. 1991. ***Delamination of Bonded Concrete Overlays at Early Ages.*** **Research Report 1205-2. Texas Department of Transportation, Austin, TX.**

A procedure is developed by which the likelihood of delamination of bonded concrete overlays on continuously reinforced concrete pavements is reduced. The procedure compares the early-age interface stress to the expected interface bond strength for a variety of environmental conditions. When the calculated stress exceeds the expected strength, it is recommended that overlay placement be curtailed until the possibility of debonding is reduced. A finite-element method program is used to determine the early-age stresses resulting from temperature and shrinkage-induced volume changes. Stresses were determined for a variety of environmental and material combinations and overlay thicknesses. Analyses show that a significant reduction in stress results from the use of overlay materials which have a lower modulus and thermal coefficient than those of the existing slab. The stresses for a given combination of materials and environmental conditions are compared to the interface bond strength at early ages. Early-age interface shear and tensile strengths are estimated from 7-day strength test results. The estimated strength, together with the variability of the interface strength, are used to calculate the likelihood of delamination for a given type of overlay and time of placement. This likelihood can be reduced through the selection of a different overlay material or time of placement.

Makahaube, J. S., S. Nazarian, and D. B. Rozendal. 1993. ***Investigation of Parameters Affecting the Interface Bonding of Thin Concrete Overlays Due to Vehicular Vibration.*** **Research Report 1920-2. Texas Department of Transportation, Austin, TX.**

This research is a further study of parameters affecting the interface bonding of thin concrete overlays. The specimens were tested in the flexural mode, while in the previous study, the direct shear test was applied to the specimens. The parameters investigated were surface condition, surface texture, pre-vibration cure time, overlay thickness, and vibration amplitude. The parameters were combined in different sequences and their effects on the interface bonding of concrete overlays are reported. The effects of surface condition were studied by comparing the shear stress obtained with dry surfaces and wet surfaces. The intention of this investigation was to determine the optimal moisture condition at the interface immediately before the placement of the overlay. The effects of surface texture were investigated to determine if scarifying the interface would improve the bond between the overlay and the base concrete. The pre-vibration cure time which corresponds to the traffic closure after pouring was also studied. Finally, the effects of thickness of overlay were studied by comparing the effectiveness of interface bonding.

Marks, V. J. 1987. ***Thin Bonded Portland Cement Concrete Overlay.*** **Progress Report, Project HR-520. Iowa Department of Transportation, Ames, IA.**

A 3-in (76 mm) thick, bonded PCC overlay and integral widening was used to rehabilitate a 4.5-mile (7.3 km) section of Iowa's Route 141 in Dallas County. There was a substantial amount of cracking in the old, 20-ft (6 m) wide PCC pavement. Most of the widening, which was tied to the original slab by dowel bars, was placed as a 4 ft (1.2 m) wide section on one side. Coring has shown that the overlay is well bonded. Testing with the Delamtech has shown less than 1 percent debonding. Mid-panel transverse cracks in the old pavement have reflected through the overlay, as expected, and some new transverse cracking has occurred. This cracking has not caused any significant problems. In general, at the time of the report the overlay was performing quite well. The construction process included placement of longitudinal subdrains, shot blasting of the surface, and air blasting just prior to placement of the grout. A sand-cement grout was used and the overlay varied from 3- to 4.33-in (76 to 110 mm).

Marks, V. J. 1989. ***A Fifteen Year Performance Summary of Fibrous PC Concrete Overlay Research in Greene County, Iowa.*** **Research Project HR-165, Iowa Department of Transportation, Ames, IA.**

The Greene County, Iowa overlay project, constructed in October 1973, was evaluated in October 1978, after 5 years of service, in October 1983 after 10 years of service, and most recently in October 1988 after 15 years of service. The 33 fibrous concrete sections, four CRCP sections, two mesh reinforced and two plain concrete sections with doweled reinforcement were rated relative to each other on a scale of 0 to 100. All experimental overlay sections had performed quite well in the period from five through 15 years, experiencing only limited additional deterioration. The 4-in thick nonfibrous mesh reinforced continuous reinforced concrete pavement overlay sections provided the best performance in this research project. Another nonfibrous 5-in thick bar reinforced overlay section performed second best. The best performance of a fibrous reinforced concrete section was obtained with 160 pounds of fiber per cubic yard. The use of 750 pounds of cement per cubic yard provided no benefit over the use of 600 pounds of cement per cubic yard.

The performance of the fibrous overlays was directly related to fiber content of the concrete mix. The 160 pounds per cubic yard provided the best performance with the poorest performance exhibited by the 60 pounds of fiber per cubic yard. There is no significant difference in the performance of the 2 ½ in long and 1 in long fibers. The 3 in thick fibrous concrete overlays yielded substantially better performance than the 2 in fibrous overlays. Substantial bonding was not achieved on any of the fibrous concrete overlay sections and, therefore, no conclusion can be reached in regard to the type of bonding. In general, however, the thicker, nonfibrous pavement overlay sections performed better than the fibrous reinforced concrete overlays. The additional cost of the fibrous concrete overlays cannot be justified based upon the comparative performance of the fibrous and thicker nonfibrous overlay sections.

Marks, V. J. 1990. ***Thin Bonded Portland Cement Concrete Overlay.*** **Final Report HR-521. Iowa Department of Transportation, Ames, IA.**

A four and one-half inch thick, bonded portland cement concrete (PCC) overlay and integral widening were used to rehabilitate a 4.5 mile section of Iowa route 141 from US 169 to Iowa 210 in Dallas County. There was a substantial amount of cracking in the old 20 feet wide PCC pavement. Most of the widening, which was tied to the original slab by dowel bars, was placed as a four feet wide section on each side. Coring has shown that the overlay is well bonded and testing with the Delamtech has shown less than 1 percent debonding. Midpanel transverse cracks in the old pavement have reflected through the overlay (as expected). Some new transverse cracking has occurred. This cracking has not caused any significant problems. In general, the overlay is performing quite well.

McCormack, M. T. 1991. ***Thin Bonded Portland Cement Concrete Overlay and Concrete Pavement Restoration (CPR).*** **Final Report, Project 84-43. Pennsylvania Department of Transportation, Harrisburg, PA.**

The objective of this report is to evaluate the performance and effectiveness of the techniques used in PennDOT Demonstration Project 84-43. This demonstration project, conducted in June 1985 on SR 322 in Hershey, Pennsylvania, utilized various methods for extending the life of concrete pavements that have minor defects. These methods included: a thin bonded concrete overlay, concrete pavement restoration (CPR) patching, spall repair, joint rehabilitation, and prefabricated pavement base drain installation. This report summarizes test results and distress surveys performed after five years of service, analyzes construction and maintenance costs, and estimates remaining pavement life. Test results suggest that the thin bonded concrete overlay performed at least as well as expected. Based on economic analysis, its use as a CPR technique should be evaluated on a project-by-project basis. In some cases a bituminous overlay alternative may be more appropriate. The prefabricated pavement base drain and other CPR techniques used on this project also performed sufficiently well.

McCullough, B. F. and D. W. Fowler. 1994. *Bonded Concrete Overlay (BCO) Project Selection, Design, and Construction.* Research Report 920-6F. Texas Department of Transportation, Austin, TX.

This report demonstrates that a bonded concrete overlay (BCO) can be a viable and economical rehabilitation strategy for an in-service PCC pavement. In addition, it provides a review of state-of-the-art methods and guidelines for design, construction, and maintenance of BCOs. Although the information and experience has been primarily developed for CRCP, the concepts are applicable to all types of portland cement concrete (PCC) pavements. The report first reviews the advantages and limitations associated with BCOs, followed by a detailed summary of Texas' experience with BCOs. It surveys Texas projects, evaluates in-service behavior and performance characteristics, and emphasizes the steps taken in the first 10-72 hours after concrete placement. Next, the report describes the criteria for selecting the conditions that maximize BCO performance. It then outlines the process used for designing thickness, reinforcement, and interface (a user-friendly automated process is furnished in the appendix). Finally, the report describes specifications, BCO construction control, and the maintenance procedures to follow when repairing distress on an existing PCC pavement scheduled to receive an overlay.

McCullough, B. F. and R. O. Rasmussen. 1999a. *Fast-Track Paving: Concrete Temperature Control and Traffic Opening Criteria for Bonded Concrete Overlays, Volume I: Final Report.* FHWA-RD-98-167. Federal Highway Administration, McLean, VA.

It has been theorized that early-age behavior caused by temperature and moisture changes can significantly affect the performance of a portland cement concrete pavement (PCCP) or bonded concrete overlay (BCO) over its service life. During the first 72 hours following placement, the strength of PCC is relatively low in comparison to the strength that it will eventually achieve. During this early-age period, critical stresses can develop which may lead to pavement damage and ultimately, a loss of performance. This research focuses on modeling early-age behavior of both concrete pavements and BCOs subjected to stresses from moisture and thermal changes. It includes the development of a two-part, versatile, comprehensive set of guidelines that provide direction in the proper selection of design and construction variables to minimize early-age damage to the PCCP and BCO. The first part of these guidelines is qualitative in nature and is based upon the results of this effort, past experience, and engineering judgment. They are intended to identify design and construction inputs that are most likely to lead to good behavior during the early-age period.

The second part of the guidelines is comprised of many complex models that have been developed to predict early-age behavior in jointed plain concrete pavements and BCOs. These models are used to verify good behavior from the selection of inputs made using the qualitative guidelines. These models include a PCC temperature development model which accounts for heat generation from the hydrating paste, solar insulation, surface convection, irradiation, and dynamic specific heat and thermal conductivity values. Several mechanical properties are also modeled including thermal coefficient of expansion, drying shrinkage, creep, strength, and modulus of elasticity (using maturity methods). Finally, restraint to free movement caused by slab-base friction and curling are modeled directly. The end product from this research is a comprehensive software package termed HIgh PERformance PAVing (HIPERPAV). This package, which incorporates the complex models developed, can be used as a stand-alone product to verify the overall effect of specific combinations of design, construction, and environmental inputs on early-age behavior of a PCCP and BCO. This report serves as the final report for this project.

McCullough, B. F. and R. O. Rasmussen. 1999b. *Fast-Track Paving: Concrete Temperature Control and Traffic Opening Criteria for Bonded Concrete Overlays, Volume II: HIPERPAV User's Manual.* FHWA-RD-98-168. Federal Highway Administration, McLean, VA.

It has been theorized that early-age behavior caused by temperature and moisture changes can significantly affect the performance of a portland cement concrete pavement (PCCP) or bonded concrete overlay (BCO) over its service life. During the first 72 hours following placement, the strength of PCC is relatively low in comparison to the strength that it will eventually achieve. During this early-age period, critical stresses can develop that may lead to pavement damage, and ultimately loss of performance. This research focuses on modeling early-age behavior of both concrete pavements and BCOs subjected to stresses from moisture and thermal changes. It includes the development of a two-part, versatile, comprehensive set of guidelines that provide direction in the proper selection of design and construction variables to minimize early-age damage to the PCCP and BCO. The first part of these guidelines is qualitative in nature and is based upon the results of this effort, past experience, and engineering judgment. They are intended to identify design and construction inputs that are most likely to lead to good behavior during the early-age period.

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McGhee, K. H. and C. Ozyildirim. 1992a. *Construction of Thin-Bonded Portland Cement Concrete Overlay Using Accelerated Paving Techniques.* FHWA/VA-IR2 (VTRC 92-IR2). Virginia Department of Transportation, Charlottesville, VA.

The report describes the Virginia Department of Transportation's (VDOT's) first modern experience with the construction of thin-bonded portland cement concrete overlays of existing concrete pavements and with the fast track mode of rigid pavement construction. The study was conducted in cooperation with the Federal Highway Administration (FHWA) and used a paving mixture verified in an FHWA mobile laboratory. The study showed that the fast track mode will permit lane closure times as short as 48 hours. Of special interest was the finding that adequate strength of the bond between the old pavement and the overlay is not dependent on the use of a bonding grout.

McGhee, K. H. and C. Ozyildirim. 1992b. "Construction of a Thin-Bonded Portland Cement Concrete Overlay Using Accelerated Paving Techniques." *Transportation Research Record 1335.* Transportation Research Board, Washington, DC.

The Virginia Department of Transportation's first modern experience with the construction of thin-bonded portland cement concrete overlays of existing concrete pavements through the use of fast-track paving is described. The study was conducted in cooperation with FHWA, and paving mixtures tested in an FHWA mobile laboratory were used. The study showed that the pavement could be overlaid and opened to traffic within 48 hours. Of special interest was the finding that the overlay concrete bonds to the base concrete with or without the use of a bonding grout.

Moody, E. D. and B. F. McCullough. 1993. "Bonded Concrete Overlays for Rehabilitation of Continuously Reinforced Concrete Pavement." ***Proceedings, Innovation in Repair Techniques of Concrete Structures, 1993 ASCE Annual Convention.*** **American Society of Civil Engineers, New York, NY.**

As a result of the 1991 Intermodal Surface Transportation Efficiency Act (ISTEA), significant funding is now available for the rehabilitation of the nation's Interstate highway system. Much of this system has been in service since the 1960s, providing over 30 years of service despite traffic levels that have greatly exceeded the original designs. Many engineers are now faced with selecting the most cost effective rehabilitation alternatives for these highways. These are difficult engineering decisions that involve millions of taxpayer dollars and unfortunately the knowledge base from which to draw from is limited.

Neal, B.F. 1983. ***California's Thin Bonded PCC Overlay.*** **Report No. FHWA/CA/TL-83-04. California Department of Transportation, Sacramento, CA.**

The placement of a thin bonded concrete overlay on Interstate 80 in the Sierra Mountains of California is described in this report. The existing pavement experiences excessive wear in the wheelpaths due to the widespread use of tire chains in the winter months. Because the pavement was otherwise structurally sound and asphalt concrete overlays had not performed well in this environment, it was decided to construct a 3-in bonded overlay. Constructed during June 1981, this pavement experienced extensive, severe delamination almost immediately and had to be replaced with an asphalt concrete overlay. It was believed that much of the failure was due to environmental effects related to the high temperature differentials and wind. Surface preparation was also found to be an important factor.

Nelson, P. K. and R. O. Rasmussen. 2002. "Delamination Stresses at the Interface of Bonded Concrete Overlays." ***Preprint Paper.*** **81st Annual Meeting of Transportation Research Board, Washington, DC.**

A two-dimensional finite element model, NSLIP 2000, has been developed to predict the shear and normal stresses at the interface between a concrete overlay and the underlying pavement. The interfacial stresses that develop are the result of temperature and moisture gradients, and traffic loading. NSLIP 2000 uses the material properties of the overlay and the underlying pavement, their dimensions, and the interfacial bond properties to predict these interfacial stresses. The mechanistic nature of this model allows it to be applied to bonded concrete overlay systems and to whitetopping pavements, where a concrete overlay is bonded to the underlying hot-mix asphalt pavement. This model can be used as a tool in the FHWA software HIPERBOND to optimize the time of overlay construction, to select appropriate overlay materials and to promote the long-term performance of pavements rehabilitated with bonded concrete overlays.

Obuchowski, R. H. 1983. "Construction of Thin Bonded Concrete Overlay." ***Transportation Research Record 924.*** **Transportation Research Board, Washington, DC.**

In 1981, a 3-in (76 mm) thick bonded concrete overlay was placed on Interstate 81 north of Syracuse. This overlay was placed to remedy widespread longitudinal and transverse joint deterioration caused by porous coarse aggregate in the existing concrete pavement. Deteriorated pavement at the joints was removed to a depth of 3 in (76 mm) by milling. The overlay was bonded to the existing pavement with a cement-sand grout after surface preparation by scarification, sandblasting, and cleaning. Some pressure relief joints were constructed prior to overlaying. At the time of the report, adequate bond had been achieved and cracking was not a problem.

Ozyildirim, C., C. Moen, and S. Hladky. 1997. ***Investigation of Fiber-Reinforced Concrete for Use in Transportation Structures.*** **Report FHWA/VTRC 97-R15. Virginia Department of Transportation, Charlottesville, VA.**

This report presents the results of a laboratory investigation to determine the properties of fiber-reinforced concretes (FRCs) with steel (hooked-end), polypropylene (monofilament and fibrillated), and the recently introduced polyolefin fibers (monofilament) for use in pavement and bridge deck overlay applications. Concrete properties in the unhardened and hardened states were evaluated and compared. Although the ultimate splitting tensile strength, compressive strength, and first crack strength were higher in most of the FRCs, when strength values were adjusted for changes in air content, only a few batches had higher strengths. The addition of fibers resulted in great improvements in flexural toughness and impact resistance. Parallel with this study, three FRC pavement overlays were applied in Virginia in 1995. The FRCs used in these projects were similar to those used in this laboratory investigation, with similar fiber volumes, types, and sizes. To implement the findings of the study successfully, the performance of these FRC pavement overlays is being monitored.

Peshkin, D. G. and A. L. Mueller. 1990. "Field Performance and Evaluation of Thin Bonded Overlays." ***Transportation Research Record 1286.*** **Transportation Research Board, Washington, DC.**

Bonded concrete overlays of jointed concrete pavements currently are not used widely for pavement rehabilitation. As part of a major FHWA research project, between 1987 and 1988 an extensive performance evaluation of 16 different bonded overlay designs at 10 locations in 6 states was carried out. The projects were located in three different environmental zones and involved a variety of low- to high-volume pavements. The overlays ranged in age from 3 to 11 years at the time of the surveys. The field performance surveys consisted of the following elements: a comprehensive field survey to identify, measure, and map pavement surface distresses; a debonding survey; the measurement of roughness and a panel present serviceability rating; deflection testing with a falling weight deflectometer; and a materials testing and sampling program. Historical traffic data were collected to estimate the accumulated 18-kip equivalent single axle loads (ESALs) on the pavement before and after the overlay was placed. Environmental data were also collected to describe the nature of the environmental forces to which the various sections were subjected. Previous research projects or state reports were reviewed to characterize the construction conditions and the preoverlay condition of the pavements. The results of the field survey are presented for each pavement section based on all of the major measured parameters. Overall, it was found that bonded concrete overlays showed mixed success. Some of the projects appeared to be nearing failure, based on the accumulation of surficial distresses and the apparent widespread debonding that was observed. In general, it was found that debonding was cause for concern on many of the projects.

Peshkin, D. G., A. L. Mueller, K. D. Smith, and M. I. Darter. 1990. ***Structural Overlay Strategies for Jointed Concrete Pavements, Volume III—Performance Evaluation and Analysis of Thin Bonded Concrete Overlays.*** **FHWA-RD-89-144. Federal Highway Administration, McLean, VA.**

A major field study and evaluation has been conducted into the effectiveness of three structural overlay types for portland cement concrete (PCC) pavements. These include sawing and sealing asphalt concrete (AC) overlays of PCC pavements, cracking and seating PCC pavements prior to AC overlay, and constructing a thin bonded PCC overlay on top of the existing PCC pavement. Condition surveys, deflection testing, and roughness measurements were performed on a total of 55 sections. The performance of these sections was evaluated and the effectiveness of each overlay type analyzed. Based on the field data, guidelines were developed for the use of these structural overlays.

This volume documents the field evaluation of 16 sections of thin bonded concrete overlay projects at 10 locations in 6 states. Compared to other overlay techniques, a concrete overlay has the potential for an extended service life, increased structural capacity, and lower life cycle costs. Field data items collected and analyzed include pavement distress, deflections, layer and material samplings, drainage, roughness, serviceability, and overlay debonding. Other information, such as design and construction information, preoverlay condition, and traffic volumes, were collected from the participating State Highway Agencies. All of this information was closely examined to determine and evaluate the relative performance levels of each concrete overlay. Based on the findings from the performance evaluation, revised design and construction guidelines were developed.

Rasmussen, R. O., B. F. McCullough, J. M. Ruiz, and P. J. Kim. 1999. ***Fast-Track Paving: Concrete Temperature Control and Traffic Opening Criteria for Bonded Concrete Overlays, Volume III: Addendum to the HIPERPAV User's Manual.*** **FHWA-RD-99-200. Federal Highway Administration, McLean, VA.**

This is an addendum to the User's Manual of the comprehensive software package termed HIgh PERformance PAVing (HIPERPAV). This package, which incorporated the complex models developed, can be used as a stand-alone product to verify the overall effect of specific combinations of design, construction, and environmental inputs on early-age behavior of a portland cement concrete pavement (PCCP) and bonded concrete overlay (BCO). This report provides color illustrations and an update of information in the User's Manual.

Rasmussen, R. O., B. F. McCullough, and J. Weissman. 1995. *Development of a Bonded Concrete Overlay Computer-Aided Design System.* Research Report 2911-1. Texas Department of Transportation, Austin, TX.

Bonded concrete overlays (BCOs) are increasingly being used to rehabilitate concrete pavements. Among other benefits, BCOs can reduce life-cycle costs and can expedite construction (thus lowering user costs and delays). Until recently, the design of BCOs has been a tedious process. Several design methods are available, including the 1993 Guide for Design of Pavement Structures and the Rigid Pavement Rehabilitation Design System (RPRDS). These design procedures have been automated into a user-friendly software package entitled Bonded Concrete overlay Computer-Aided Design (BCOCAD). This report documents the development and implementation of the BCOCAD program.

Rasmussen, R. O., B. F. McCullough, D. G. Zollinger, and S. Yang. 1997. "A Foundation for High Performance Bonded Concrete Overlay Design and Construction Guidelines." *Proceedings, Sixth International Purdue Conference on Concrete Pavement, Design and Materials for High Performance.* Indianapolis, IN.

It has been theorized that early-age behavior due to temperature and moisture changes can significantly affect the performance of a Bonded Concrete Overlay (BCO) over its service life. During the first 72 hours following placement, the bond strength between the BCO and the existing pavement is relatively low in comparison to the strength that it will eventually achieve after this initial period. During this "early-age" period, significant critical stresses at the bond interface can develop. This can lead to premature delamination of the BCO which can subsequently result in a loss of performance of the pavement system.

Very few methods exist to predict bond stress and strength which could serve as tools to prevent delamination from occurring. Those that do exist are generally too simplistic in nature for practical application due to their inherent assumptions. These existing models generally fail to account for the complex interactions between the numerous mechanisms involved, resulting in a significant loss of accuracy. Therefore, an ideal tool for this type of analysis would allow for flexibility in the large number of inputs that determine these phenomena. In addition the ideal tool should be immediately implementable by prompting for inputs which are readily available to the practitioner.

This paper presents the results of an FHWA research project which has focused on modeling early-age behavior of BCOs subjected to stresses from moisture and thermal changes. The end-products of this research effort included the development of a two-part, versatile, comprehensive set of guidelines which provide direction in the proper selection of design and construction variables to minimize early-age delamination in BCO systems.

Rowden, L. R. 1996. *Thin Bonded Concrete Overlay and Bonding Agents.* FHWA/IL/PR-123. Illinois Department of Transportation, Springfield, IL.

This report presents the construction procedures and initial performance evaluation of a 4-inch (10-cm) bonded concrete overlay placed on Interstate 80 near Moline, Illinois. Preconstruction testing consisted of Falling Weight Deflectometer, permeability to chloride, and distress surveys. Surface preparation included: full-depth patching, partial-depth patching, bituminous material removal, shot blasting, and sand blasting. During construction of the overlay, concrete temperature, air temperature, wind speed, relative humidity, rate of evaporation, concrete slump, concrete air content, water-cement ratio, placement time, and overlay thickness were recorded. Testing conducted after construction of the overlay included: compressive strength, split tensile strength, distress surveys, location of delamination, Falling Weight Deflectometer, California Profilograph, International Roughness Index, friction, drying shrinkage of concrete, and bond strength.

Shirazi, H. H., M. Rasoulian, and B. King. 1996. "Design and Construction of a Bonded Fiber Concrete Overlay of CRCP." *Proceedings, Materials for the New Millennium, 1996 Materials Engineering Conference.* American Society of Civil Engineers, New York, NY.

A study was conducted on an existing 16-year old 8-inch CRC pavement to ascertain the probability of long term success by using concrete mix with high cement content, internal reinforcement, and with good bonding characteristics. Additional reinforcement-bonding was provided utilizing curb type reinforcement bars epoxied into the existing slab to provide positive bonding at the slab edge to prevent debonding due to curling and/or warping. To increase the pavement's structural capacity, a 9-inch tied concrete shoulder was added. The overall Serviceability Index increased from 3.4 to 4.4 with measured Profile Index levels typically below the 5-inch per mile specification. Tests revealed excellent bond strengths and reduced edge deflections by 60 percent.

Smith, T. E. and S. D. Tayabji. 1998. *Assessment of the SPS-7 Bonded Concrete Overlays Experiment: Final Report.* FHWA-RD-98-130. Federal Highway Administration, McLean, VA.

This report presents an assessment of the Long-Term Pavement Performance (LTPP) SPS-7 experiment. This report is intended to serve as background material for a meeting of State agencies to be held to review the status of the SPS-7 experiment. The four SPS-7 projects are described in detail and an assessment is provided on the availability and quality of data for these four projects. The scope of work for this study did not include data analysis.

Solanki, A. I., D. W. Fowler, and B. F. McCullough. 1987. *A Study of the Effect of Construction Variables on the Bond Behavior of CRCP Overlays.* Research Report 457-4. Texas State Department of Highways and Public Transportation, Austin, TX.

This report summarizes the findings of Report 457-4, "A Study of the Effect of Construction Variables on the Bond Behavior of CRCP Overlays," and describes a series of research activities concerned with the development of the bond between an existing CRCP pavement and a new CRCP overlay. This report includes a summary of activities related to preparation of the surface, and determination of the effect of moisture level, grout condition, vibration level, and locations in CRCP overlays. Results of general linear model (GLM) analysis are used to find the best and worst interactions of variables. Various texture measurement and bond evaluation devices are evaluated in terms of time, economy, and repeatability.

South Dakota Department of Transportation (SDDOT). 1985. *Construction of a Thin-Bonded Portland Cement Concrete Overlay in South Dakota.* Experimental Project SD 85-03. South Dakota Department of Transportation, Pierre, SD.

Given the high cost of new pavement construction and the increased need for rehabilitation of existing pavements, the South Dakota Department of Transportation decided to try constructing a thin bonded PCC overlay on Highway 38A, near Sioux City. Construction took place in June 1985. This pavement was originally scheduled for the construction of an asphalt concrete overlay. Instead, it was decided to place 3 to 4 in (76 to 102 mm) of thin-bonded overlay. Preoverlay preparation included the placement of full- and partial-depth repairs and repair of longitudinal cracking. The pavement was shot blasted prior to the placement of the cement-water grout. Problems encountered included random centerline cracking and some reflection cracking of transverse cracks. It was recommended that the joints be sawcut as soon as possible, and that the longitudinal joint sawcut depth be at least one-half the overlay thickness and located over the old joint.

Sprinkel, M. M. 1998. *Very-Early-Strength Latex-Modified Concrete Overlays.* VTRC 99-TAR3. Virginia Transportation Research Council, Charlottesville, VA.

This report describes the installation of the first two very-early-strength latex-modified concrete (LMC-VE) overlays constructed in Virginia. The overlays were prepared with a special blended cement rather than the Type I/II cement used in the conventional latex-modified concrete (LMC) overlay. Mixture proportions, installation equipment, and procedures are similar to those used for conventional LMC overlays except that the concrete loses slump rapidly and the curing period is only 3 hours rather than 72. Early-age compressive strength tests indicated that traffic could be placed on the overlay within 3 hours rather than the 4 to 7 days for conventional LMC overlays. Tests of the bond strength and permeability indicate that the overlays are performing satisfactorily. Pending continuing favorable test results, it is anticipated that LMC-VE overlays can be used in situations in which it is desirable to accelerate construction, to reduce inconvenience to motorists, to allow for installation during off-peak traffic periods such as at night, to provide a more rapid cure in cold weather, to provide low permeability, and to provide high early strength.

Sprinkel, M. M. 2000. *Thin Bonded Overlays and Surface Lamination Projects for Pavements, Final Report.* Federal Highway Administration, Washington, DC.

This document summarizes findings from pavement projects that were constructed under the Thin Bonded Overlay and Surface Lamination (TBO) program, which was authorized under Section 6005 (a) (e) (7) of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA). This report is based primarily on information contained in the TBO project construction and evaluation reports that were available through 2000. At the time this report was written, construction reports had been received for 13 of 18 pavement TBO projects, with final evaluation reports had been received for 7 pavement TBO projects. Broadly considered, the 18 projects included:

- 7 projects demonstrating hydraulic cement concrete (HCC) placed on HCC.
- 9 projects demonstrating HCC on AC (whitetopping).
- 1 project demonstrating an asphalt surface treatment (Novachip)
  - placed on AC at 2 sites.
  - placed on HCC at 1 site.
  - placed on Ralumac at 1 site.
- 1 project demonstrating an epoxy overlay (Italgrip) on HCC.

This project has demonstrated the viability of AC and HCC concrete overlays on AC and HCC pavements. The majority of the TBO projects were very successful, demonstrating that overlays can be constructed on pavements that are adequately bonded, provide good surface condition, provide low roughness, provide good skid resistance, and extend the pavement service life.

Sprinkel, M. M. and C. Ozyildirim. 1999. *Evaluation of the Installation and Initial Condition of Hydraulic Cement Concrete Overlays Placed on Three Pavements in Virginia.* Report No. VTRC 99-IR3. Virginia Department of Transportation, Charlottesville, VA.

Hydraulic cement concrete pavement overlays were placed in the summer of 1995 at the following locations in Virginia: I-295 near Richmond; I-85 near Petersburg; and Rt. 29 near Charlottesville. Overlays were placed on I-295 SBL (near mile marker 29) and I-85 SBL (near mile marker 51) in Virginia to prevent spalling caused by a shy cover over the reinforcement and to enhance the structural integrity. Both locations are continuously reinforced concrete pavement. An overlay was also placed on Rt. 29 NBL (1.6 km south of Charlottesville) in Virginia to correct a rutted asphalt pavement. The construction was funded with 20 percent Virginia Department of Transportation maintenance funds and 80 percent special Intermodal Surface Transportation Efficiency Act (ISTEA) Section 6005 federal funds specifically allocated to demonstrate overlay technologies. ISTEA funds were also used to evaluate the installation and initial conditions of the overlays and to prepare the report. The variables in this study were concrete mix design, overlay thickness, and base material. Mineral admixtures and steel and plastic fibers were used to improve the mechanical properties and durability of the overlay concrete. Overlay thickness and base material were varied to determine their effect on overlay performance.

Sprinkel, M. M. and C. Ozyildirim. 2000. *Evaluation of Hydraulic Cement Concrete Overlays Placed on Three Pavements in Virginia.* Report No. VTRC 01-R2. Virginia Department of Transportation, Charlottesville, VA.

Three hydraulic cement concrete pavement overlays were placed in the summer of 1995 at three locations in Virginia. Two of the overlays were placed on continuously reinforced concrete pavement to prevent spalling caused by a shy cover over the reinforcement and to enhance the structural integrity. The third overlay was placed to correct a rutted asphalt pavement. The variables in the study were concrete mix design, overlay thickness, and base material. Mineral admixtures and steel and plastic fibers were used to improve the mechanical properties and durability of the overlay concrete. Overlay thickness and base material were varied to determine their effect on overlay performance. Overlays that were 51 and 102 mm (2 and 4 in) thick worked well on hydraulic cement concrete pavements. Overlays that were 76 and 102 mm (3 and 4 in) thick worked well on asphalt concrete pavements. These overlays can be used to extend the life of the pavements.

Sprinkel, M. M. and C. Ozyildirim. 2001. "Evaluation of Hydraulic Cement Concrete Overlays Placed on Three Pavements in Virginia." ***Proceedings,*** **Seventh International Conference on Concrete Pavements, Orlando, FL.**

Three hydraulic cement concrete pavement overlays were placed in the summer of 1995 at three locations in Virginia. Two of the overlays were placed on continuously reinforced concrete pavement to prevent spalling caused by a shy cover over the reinforcement and to enhance the structural integrity. The third overlay was placed to correct a rutted asphalt pavement. The construction was funded with 20 percent Virginia Department of Transportation maintenance funds and 80 percent special ISTEA Section 6005 federal funds specifically allocated to demonstrate overlay technologies. ISTEA funds were also used to evaluate the installation and initial conditions of the overlays and to prepare the report. The variables in the study were concrete mix design, overlay thickness, and base material. Mineral admixtures and steel and plastic fibers were used to improve the mechanical properties and durability of the overlay concrete. Overlay thickness and base material were varied to determine their effect on overlay performance. Overlays that were 51 and 102 mm (2 and 4 in) thick worked well on hydraulic cement concrete pavements. Overlays that were 76 and 102 mm (3 and 4 in) thick worked well on asphalt concrete pavements. These overlays can be used to extend the life of the pavements.

Sprinkel, M., C. Ozyildirim, S. Hladky, and C. Moen. 1997. "Pavement Overlays in Virginia." ***Proceedings, Sixth International Purdue Conference on Concrete Pavement, Design and Materials for High Performance.*** **Indianapolis, IN.**

This paper describes the installation and performance of three hydraulic cement concrete overlays in Virginia. Two of the overlays were placed on continuously reinforced concrete pavement, and the other was placed on rutted asphalt pavement. The overlays were placed to increase the flexural strength of the pavement and demonstrate the feasibility of concrete overlays for extending the service life of pavements. One of the overlays was also placed to prevent spalling caused by a shy cover over the reinforcement.

The concretes used in the overlays were strong and highly resistant to the intrusion of solutions, characteristic of high performance concretes. Hooked-end steel, polyolefin, and monofilament and fibrillated polypropylene fibers were added to the concrete used in 31-m test sections to improve its toughness and ability to control cracks.

The overlay thicknesses were as follows: 51 mm for project A, 102 mm for project B, and sections of 51 mm, 76 mm, and 102 mm for project C. Each of the test sections was 610 m long. Concretes were tested in the unhardened state for slump and air content and in the hardened state for compressive strength, permeability to chloride ions, freeze-thaw durability, length change, and shear bond strength. The fiber-reinforced concrete was tested in flexure in accordance with ASTM C1018. The overlays were tested for tensile bond strength, skid resistance, and stiffness measured by falling weight deflectometer.

Tests revealed that all of the overlay concretes were strong and had low to very low permeabilities. Freeze-thaw resistance and shear and tensile bond strengths were all acceptable. Pavement stiffness and skid resistance were improved with the addition of the overlays. The addition of fibers to the overlay concretes improves their toughness and appears to be restricting crack widening at 1 year after placement.

Suh, Y-C., J. R. Lundy, B. F. McCullough, and D. W. Fowler. 1988. A ***Summary of Bonded Concrete Overlays.*** **Research Report 457-5F. Texas State Department of Highways and Public Transportation, Austin, TX.**

The objectives of the study described in this report are to determine the warrants for the use of bonded concrete overlays, to provide recommendations for their construction, to evaluate the advantages and disadvantages of the various materials used, and to examine the use of different thicknesses of overlay. Data was obtained from lab tests, field tests, and a field placement of bonded overlays over a CRC pavement. Variables examined included method of surface preparation, surface moisture condition, use of grout, level of vibration, and type of reinforcement.

Silfwerbrand, J. and O. Petersson. 1993. "Thin Concrete Inlays on Old Concrete Roads." ***Proceedings, Fifth International Conference of Concrete Pavement Design and Rehabilitation.*** **Purdue University, West Lafayette, IN.**

The purpose of this paper is to evaluate thin concrete overlays as a repair method on old worn-out concrete roads. A test section has been carried out in a location south of Sweden that used fiber-reinforced, high strength concrete. The performance of the test section to date has been positive. Bond strength tests indicate a good bond was achieved between the new and the old concrete. Tests on the fiber-reinforced, high-strength concrete indicated a high tensile strength, a high modulus of elasticity, a high shrinkage, and low creep.

Tayabji, S.D. and C. G. Ball. 1988. "Field Evaluation of Bonded Concrete Overlays." ***Transportation Research Record 1196.*** **Transportation Research Board, Washington, DC.**

A field program of strain and deflection measurements was conducted by the Construction Technology Laboratories (CTL) for the Iowa Department of Transportation. The objective of the field measurement program was to obtain information on bonded concrete resurfaced pavements that can be used as a data base for verifying bonded resurfacing thickness design procedures. Data gathered during the investigation included a visual condition survey, engineering properties of the original and resurfacing concrete, lead related strain and deflection measurements, and temperature-related curl (deflection) measurements. Field load testing was conducted by CTL at five sites in Iowa during April 1986. This report presents the results of field testing, analysis of results, and recommendations to incorporate study results in Iowa design procedures for bonded concrete overlays. Results of the investigation indicate that the four overlaid pavement sections evaluated as part of the reported study are performing as monolithic pavements with high interface shear strength at the interface. The strength of the existing pavement at all of the four overlaid test sections was high. In addition, cores obtained from sections 4 and 5 did not indicate D-cracking related damage in the overlay concrete. Comparison of the condition surveys for Section 1 (non-overlaid JRCP) and Section 2 (overlaid JRCP) indicate that all cracking in the existing pavement is not reflected through the overlay and that the cracks that did reflect through have remained tightly closed. Similarly, the condition survey of sections 4 and 5 indicate that crack reflected through the overlay continue to remain tightly closed even after almost seven years of service. The field investigation conducted by CTL verifies that for properly constructed bonded overlays, pavement strengthening is achieved and that the overlaid pavement behaves monolithically as a full-depth concrete pavement.

Temple, W. H. and S. L. Cumbaa. 1985. ***Thin Bonded PCC Resurfacing.*** **FHWA/LA-85/181. Louisiana Department of Transportation and Development, Baton Rouge, LA.**

The purpose of this study was to evaluate the construction techniques and performance characteristics of the Louisiana DOTD's first PCC resurfacing project, which was constructed over a short section of an existing 9-in (229 mm) dowelled concrete pavement with 20-ft (6 m) joint spacing, located on U.S. 61 north of Baton Rouge. The old pavement surface was cleaned by shot blasting. The resurfacing was 4 in (102 mm) thick and was placed on top of a water-cement grout immediately prior to the overlay. At the time of the report, approximately 16 percent of the exterior slab corners had experienced various degrees of debonding, resulting in minor cracking. It was recommended that this type of resurfacing not be performed during the hottest months of the year, that the longitudinal joint be sawcut or otherwise induced, and that additional research should evaluate the effect of a light water spray applied to the existing pavement surface immediately prior to the placement of the grout in order to inhibit drying to the grout.

Temple, W. H., S. L. Cumbaa, and W. M. King. 1992. "Design And Construction Of Bonded Fiber Concrete Overlay Of Continuously Reinforced Concrete Pavement." ***Transportation Research Record 1335*****, Transportation Research Board, Washington, DC.**

The purpose of this research was to study the design and construction of a bonded, steel-fiber-reinforced concrete overlay on an existing 8-in continuously reinforced concrete pavement (CRCP) on Interstate 10 south of Baton Rouge, Louisiana. The existing 16-year-old CRCP, which is estimated to have carried twice its design load, contained several edge punch-out failures per mile. The research objectives were to provide an overlay with a high probability for long-term success by using a high-strength concrete mix with internal reinforcement and good bonding characteristics. A 4-in concrete overlay containing steel fibers was designed. An inverted U-shaped reinforcing bar was added at the edge of the pavement to provide positive edge bonding. Shot blast surface cleaning of the existing tine surface easily met a specification requiring an average texture depth of 0.045 in. Water-cement grout was applied to the cleaned surface, producing bond strengths in excess of 900 psi. The concrete overlay in combination with 9-in tied concrete shoulders reduced edge deflections by 60 percent under a 22,000-lb moving single axle load applied 2 ft from the edge. In general, the serviceability index of the pavement increased from 3.4 to 4.4, with measured profile index levels typically below the 5-in./mi specification. The bonded overlay has been in service since August 1990 and carries average daily traffic of 41,000 vehicles. Cores taken over transverse cracks in the overlay indicated reflection cracking from transverse cracks in the original pavement. Anticipation of reflective cracking was one consideration in using the steel fibers, which provide three-dimensional reinforcement.

Teo, K. J., D. W. Fowler, and B. F. McCullough. 1989. ***Monitoring and Testing of the Bonded Concrete Overlay on Interstate Highway 610 North in Houston, Texas.*** **Research Report 920-3. Texas State Department of Highways and Public Transportation, Austin, TX.**

The objectives of this study were to determine the extent of delamination of the bonded concrete overlay on IH610 North in Houston, to determine if the delaminations are progressive, to identify probable causes of the delaminations, and to recommend remedial measures. Condition surveys were conducted periodically and laboratory tests were performed on the cores obtained from the monitored areas. Statistical analyses were then performed on the condition surveys and laboratory test data. From these analyses, conclusions and recommendations were made to enable the Texas State Department of Highways and Public Transportation to design overlays for rehabilitation programs on CRCP.

Treviño, M., B. F. McCullough, and T. Krauss. 1998. ***Full-Scale Bonded Concrete Overlay on IH-30 in Ft. Worth, Texas.*** **Research Report 572-1. Texas Department of Transportation, Austin, TX.**

The Fort Worth District has many miles of continuously reinforced concrete pavements (CRCP) that have given excellent performance throughout the years. At present many are in need of some type of rehabilitation, and the application of bonded concrete overlays (BCO) may be appropriate to extend their life. Bonded concrete overlays have proven to be a viable technical and economical solution for pavement rehabilitation in large metropolitan areas in Texas. However, the rehabilitation of a roadway always causes traffic disturbances and implies user-associated costs. The economical and technical feasibility of a BCO as a solution for rehabilitation for the heavily urbanized and traveled pavement sections on Interstate Highway 30 is presented in this report.

Treviño, M., B. F. McCullough, and T. Krauss. 2000. ***Full-Scale Bonded Concrete Overlay on IH-30 in Ft. Worth, Texas.*** **Research Report 9-572-1. Texas Department of Transportation, Austin, TX.**

This report presents the research and recommendations regarding the rehabilitation of an urban section of IH-30 in west Fort Worth, Texas. Bonded concrete overlays have proven to be a viable solution for the rehabilitation of heavily traveled pavement sections. The main objective of this project is to evaluate the technical and economical feasibility of an expedited bonded concrete overlay and to monitor its performance. In addition to the materials characterization of the existing pavement and the rehabilitation design, the report presents a strategy to expedite the BCO in conjunction with the widening of the road. The expedited construction will allow the opening of the rehabilitated pavement to traffic as early as possible and, therefore, can minimize user-associated costs. The economic analysis included in the report shows the feasibility of implementing a BCO in lieu of a full-depth reconstruction.

van Metzinger, W. A., J. R. Lundy, B. F. McCullough, and D. W. Fowler. 1991. *Design and Construction of Bonded Concrete Overlays*. Research Report 1205-4F, Texas Department of Transportation, Austin, TX.

This report summarizes studies of the performance of bonded concrete overlays (BCO), develops information on the failure mechanism of BCO, and documents an improved design model for BCO. Three pavements that received BCO placed in Houston, Texas, were closely observed, and the performance of these pavements was studied. The oldest of these pavements was about 7 years, and performance information on the older BCO pavements was used in the development of a design model and in suggested specifications adopted for the construction of the third BCO. The study of the failure mechanism examined the mechanisms that cause delamination and cracking. This study of the failure mechanism permitted the development of a design procedure through the use of a finite-element program.

van Metzinger, W. A. and B. F. McCullough. 1991. "Performance of Bonded Concrete Overlays on Continuously Reinforced Concrete Pavement." *Concrete International*, Volume 13, No. 12. American Concrete Institute, Farmington Hills, MI.

In tests conducted by the Center for Transportation Research at the University of Texas at Austin, field data from two different locations in Houston, Texas, were analyzed to evaluate the performance of bonded concrete overlays on continuously reinforced concrete pavements. One project is located on the North Loop of I-610, between East T.C. Jester Boulevard and I-45. A total of 3 one half miles were overlaid at the end of 1985. The second project is located on the South Loop of I-610 in Houston, between Cullen Boulevard and Calais Street. The project was divided into 5 sections, each approximately 200 ft long, all in four eastbound lanes. Design methodology, test methods and study results are discussed.

van Metzinger, W. A., B. F. McCullough, and D. W. Fowler. 1991. *An Empirical-Mechanistic Design Method Using Bonded Concrete Overlays for the Rehabilitation of Pavements*. Research Report 1205-1. Texas Department of Transportation, Austin, TX.

In this report, a design model for bonded concrete overlays was developed. The development included observations of the performance of existing bonded concrete overlays with varying depths, reinforcement types, and various bonding treatments. These observations were studied in detail and then analyzed statistically. The model development involved establishing the failure mechanisms which cause delamination and cracking in the bonded overlays. A finite-element program was then used to model the cracking and delamination of bonded overlays subjected to a variety of wheel loads and environmental stresses. The failure mechanisms, together with the stress information, were compared to performance observations and used to develop the design procedure.

Verhoeven, K. 1989. "Thin Overlays of Steel Fiber Reinforced Concrete and Continuously Reinforced Concrete, State of the Art in Belgium." *Proceedings, Fourth International Conference of Concrete Pavement Design and Rehabilitation*. Purdue University, West Lafayette, IN.

Resurfacing of existing roads with cement concrete is common use in Belgium since more than 10 years. About 500 km of roads (on an equivalent width of 7 m) have been overlaid during this period with "classical" continuously reinforced concrete pavement (CRCP—20 cm thick, 0.67 percent longitudinal steel). These overlays were constructed on old concrete roads (with partial bond) as well as on bituminous pavements presenting rutting problems.

With a renewed interest for the material steel fibrous reinforced concrete (SFRC) in the beginning of this decennium—interest which was essentially created by better incorporation possibilities and subsequent lower fiber contents—new uses in thin resurfacings (< 15 cm) were considered. Together with SFRC, alternatives were examined such as thin continuously reinforced concrete (TCRCP) with a thickness between 12 and 16 cm. Here the advantage of no joints has to be considered against the more difficult circumstances of construction due to the necessity of placing the reinforcement on roads where resurfacing has to be done under traffic.

Verhoeven, K. and Y. Vancraeynest. 1993. "Thin Steel Fiber Reinforced Concrete Overlays and Inlays on Old Pavements: A Ten-Year Belgium Experience." ***Proceedings, Fifth International Conference of Concrete Pavement Design and Rehabilitation.*** **Purdue University, West Lafayette, IN.**

Invented in the late 1960s, fiber reinforced concrete has been studied and tested extensively and put to subsequent use in a variety of demonstration projects. Between 1982 and 1987, Belgium constructed 129,000 m^2 of test sections using steel fiber concrete overlays. In the years that followed, decrees and prescriptions were drawn up on the basis of evaluations, which have led to several practical realizations. This paper presents significant findings and conclusions from over 10 years of experience with steel fiber concrete overlays in Belgium.

Voigt, G. F., M. I. Darter, and S. H. Carpenter. 1988. "Field Performance of Bonded Concrete Overlays." ***Transportation Research Record 1110.*** **Transportation Research Board, Washington, DC.**

Bonded concrete overlays provide two improvements to an existing pavement: increased structural capacity and a new riding surface. The importance of these benefits and improved construction technology has encouraged several States to construct bonded concrete overlays. Data from bonded overlays of jointed pavements were collected from eleven projects located in Iowa, Louisiana, New York, South Dakota, and Wyoming. This paper describes the design, construction procedures, and performance of several of the bonded overlay projects in the database. Several models are presented for distresses present in bonded overlays.

Volle, T. H. 2000. ***Performance of Thin Bonded Concrete Overlays in Illinois.*** **FHWA/IL/PRR-134. Illinois Department of Transportation, Springfield, IL.**

In recent years, two bonded concrete overlays (BCO) have been constructed in Illinois. The first was constructed in 1994-1995 on a section of Interstate-80 (I-80), east of Moline, Illinois. The second bonded concrete overlay was placed in 1996 on a section of Interstate-88 (I-88), further east of Moline. Both the I-80 and the I-88 BCO sections were placed on 8-in (203-mm) thick continuously reinforced concrete pavements. The I-80 BCO is 4 in (102 mm) thick and includes six experimental sections with various percentages of microsilica added to the standard mix design and different bonding agents used between the original pavement and the BCO. The I-88 BCO is 3 in (76 mm) thick and includes two experimental sections involving different surface preparation methods prior to BCO placement. This report summarizes the performance of the bonded concrete overlays to date. Visual distress surveys were conducted annually on selected test sections of the I-80 and I-88 overlays. The I-80 and I-88 overlays were also tested annually for International Roughness Index values. Condition Rating Surveys were conducted every two years on I-80 and I-88 to define the overall condition of the pavement. The results of the tests and surveys are included in this report.

Volle, T. H. 2001. "Thin Bonded Concrete Overlays in Illinois: Preliminary Report on Performance." ***Transportation Research Record 1778.*** **Transportation Research Board, Washington, DC.**

In recent years, two bonded concrete overlays (BCO) have been constructed in Illinois. The first was constructed in 1994–1995 on a section of I-80 east of Moline. The second was placed in 1996 on a section of I-88 further east of Moline. The I-80 BCO was a 4-in JPCP overlay placed on an 8-in CRCP, and the I-88 was a 3-in JPCP overlay placed on an 8-in CRCP. Both projects incorporate different experimental features. This paper summarizes the performance of the bonded concrete overlays to date. Visual distress surveys were conducted annually on selected test sections of the I-80 and I-88 overlays. The I-80 and I-88 overlays were also tested annually for International Roughness Index values. Condition Rating Surveys were conducted every two years on I-80 and I-88 to define the overall condition of the pavement. The results of the tests and surveys are included in this report.

Wade, D. M., D. W. Fowler, and B. F. McCullough. 1995. *Concrete Bond Characteristics for a Bonded Concrete Overlay on IH-10 in El Paso*. Research Report 2911-2. Texas Department of Transportation, Austin, TX.

Plans are currently underway for rehabilitating a heavily traveled section of Interstate Highway 10 in the El Paso, Texas, District. This section of highway is to be repaired using a bonded concrete overlay. This two-year study is investigating pavement design, traffic control, and construction methods that will yield a durable pavement at minimum cost and minimum burden to the public. This report describes the design of a cost-effective bonded concrete overlay mix—one capable of meeting the specifications set forth by the Texas Department of Transportation (TxDOT). The design is such as to ensure that the overlay strengthens sufficiently to permit a quick return to traffic loading. The report also describes an investigation of the factors that affect the bond performance of the overlay to be constructed.

Warner, J., S. Bhuyan, W. G. Smoak, K. R. Hindo, and M. M. Sprinkel. 1998. "Surface Preparation For Overlays." *Concrete International*. Volume 20, Number 5. American Concrete Institute, Farmington Hills, MI.

In the repair and rehabilitation of concrete structures, additional concrete is often placed on existing concrete in the form of an overlay or augmentation. The performance of the new composite is directly dependent upon good bond and full transfer of shear stresses at the interface of the new and old concrete. It is well understood that a clean and well-textured prepared surface at the bond line is essential to obtain good bond. However, not as well known is the fact that while certain equipment can provide surfaces for good bonding, other techniques can produce inadequate surfaces resulting in poor bond or possible delamination. The latter methods should be minimized or avoided whenever possible.

Wells, J. A., R. D. Stark, and D. Polyzois. 1999. "Getting Better Bond in Concrete Overlays." *Concrete International*, Volume 21, No. 3. American Concrete Institute, Farmington Hills, MI.

This article looks at which kinds of surface preparations and bonding agents work best for producing strong bonds between existing slabs and repair overlays. The four different methods of surface preparation investigated were: (1) light brooming and vacuuming; (2) vigorous hand wire brushing and vacuuming; (3) waterblasting at 27.5 MPa (4000 psi); and (4) shotblasting to ICRI Minimum CSP-5, using 390 medium-heavy blast. The six pretreatments were: (1) none (dry); (2) saturated surface dry (SSD); (3) cement-sand slurry at 0.42 water-cement ratio (w/c); (4) cement-sand-acrylic latex slurry; (5) proprietary cement-silica fume modified styrene butadiene paste (SBR); and (6) a proprietary structural grade two-component, moisture-insensitive epoxy conforming to ASTM C 881.

Whitney, D. P., P. Isis, B. F. McCullough, and D. W. Fowler. 1992. *An Investigation of Various Factors Affecting Bond in Bonded Concrete Overlays*. Research Report 920-5, Texas Department of Transportation, Austin, TX.

Data produced from this study are analyzed statistically for trends showing significant influences on the bond performance in bonded concrete overlays (BCO). The study examines the effects of three bonding agents and their application rates, moisture, three substrate surface textures, two intervals between the application of the bonding agents and the overlay, and three substrate temperatures. Recommendations are given to increase the likelihood of well-bonded BCO when placement must occur in adverse conditions. Bond strengths are compared, using three field and two laboratory methods. The advantages and disadvantages of each method are discussed. A computerized remote telemetric data acquisition system for collecting field data in heavily congested highway construction projects was developed and implemented into a concurrent BCO construction project.

Unbonded PCC Overlays

Ardani, A. 1992. *Evaluation of Unbonded Concrete Overlay.* Report No. CDOT-DTD-R-92-8. Colorado Department of Transportation, Denver, CO.

This report describes the testing, construction, and 7 years of performance evaluations of an unbonded concrete overlay with and without tied shoulders. Unbonded concrete overlay was used successfully on a thirteen mile stretch of I-25 north of Denver in Colorado. Visual investigation and distress survey was performed and results are summarized. In general, the unbonded overlay has performed quite well, with little distress to date. It appears that tied shoulders are doing what is expected, increasing the load carrying capacity of the driving lane by transferring the load to the shoulder. The results of this study demonstrated that unbonded overlays, if properly constructed, can be a viable method for resurfacing badly deteriorated rigid pavements. The use of unbonded overlays, where suitable, is recommended, along with close attention being paid to design and construction details.

American Concrete Pavement Association (ACPA). 1990. *Guidelines for Unbonded Concrete Overlays.* Technical Bulletin TB-005P. American Concrete Pavement Association, Arlington Heights, IL.

Unbonded portland cement concrete (PCC) overlays are an effective rehabilitation method for existing deteriorated PCC pavements. This overlay method involves the placement of a separation interlayer between the existing pavement and the new overlay, which prevents reflection cracking in the overlay. Because the two layers are separated, the condition of the underlying pavement is less critical and therefore less pre-overlay repair is required. This document presents guidelines for the construction of an unbonded PCC overlay on an existing PCC pavement. Information is presented on its effective design, recommended levels of pre-overlay repair, materials, and construction.

Crawley, A. B. and J. P. Sheffield. 1983. "Continuously Reinforced Concrete Overlay of Existing Continuously Reinforced Concrete Pavement." *Transportation Research Record 924.* Transportation Research Board, Washington, DC.

The design and construction of a 6-in unbonded continuously reinforced concrete (CRC) overlay of a 20-year-old continuously reinforced concrete pavement are described. This is the first time a CRC overlay has been placed over an existing CRCP. The existing CRCP was an experimental project when built and had several features that were being tried for the first time in Mississippi. One of these features, smooth wire fabric reinforcement, led to the need for the overlay. The CRC overlay project had several items new to Mississippi, including a) a new, statistically oriented quality assurance specification for rigid pavement; b) the closing of one side of an Interstate highway to traffic; and c) plain concrete shoulders paved monolithically with the CRC mainline overlay. The distress in the 20-year-old pavement, design and construction procedures, contract award provisions, traffic control features, and post-construction evaluation are discussed, and some interim recommendations are presented.

ERES Consultants, Inc. 1999. *Evaluation of Unbonded Portland Cement Concrete Overlays.* NCHRP Report 415. Transportation Research Board, Washington, DC.

This report contains the findings of a study that was performed to evaluate existing methods for rehabilitating portland cement concrete pavements with unbonded concrete overlays and to develop guidelines for their use. The report provides a comprehensive description of the research and includes detailed guidelines for the design and construction on unbonded portland cement concrete overlays. The contents of this report will be of immediate interest to pavement design and construction engineers and others involved in the design, construction, and rehabilitation of concrete pavements.

Engstrom, G. M. 1993. *Unbonded Concrete Overlays—Minnesota Experience.* Interim Report. Minnesota Department of Transportation, Maplewood, MN.

In an attempt to find new, cost-effective pavement rehabilitation techniques, the Minnesota Department of Transportation has constructed ten unbonded concrete overlays since 1977. Unbonded concrete overlays have performed well in Minnesota, and the design and construction of each successive project built on the experiences gained from the previous project. This report documents the design, construction, and performance of those ten projects and makes recommendations on the further use of unbonded concrete overlays in Minnesota.

Gulen, S. and A. S. Noureldin. 2000. "Evaluation of Concrete Pavement Rehabilitation Techniques on I-65 in Indiana." ***Transportation Research Record 1730.*** **Transportation Research Board, Washington, DC.**

Construction of hot mix asphalt (HMA) overlays on top of old concrete pavements is the most common concrete pavement rehabilitation strategy. These overlays, however, are usually subject to reflection cracking related to the movement of the old concrete slab. In addition, these overlays may also be vulnerable to rutting when subjected to large traffic volumes of heavy trucks. Concrete overlays have the advantage of being rut resistant compared to HMA overlays. However, the current national experience of the performance of these overlays is still, relatively, limited compared to HMA overlays. In addition, doubts are often raised about the cost effectiveness of these overlays, the ease of their rehabilitation at the end of their design life and the period of time required to close the road to traffic for ongoing and post construction operations.

This paper presents an evaluation of three concrete pavement rehabilitation techniques employed on interstate highway I-65: 1) a fiber modified HMA overlay on top of cracked and seated concrete pavement, 2) an HMA overlay on top of rubblized concrete pavement, and 3) an unbonded concrete overlay on top of 30 mm intermediate HMA layer on top of old concrete pavement. Performance of these rehabilitation techniques is also assessed in view with that of restoration (no overlay) techniques applied in 1985 on the same highway segment. It was concluded that all rehabilitation techniques performed satisfactorily. The unbonded concrete overlay segment exhibited the best performance in reflection cracks elimination, structural capacity and skid resistance. The rubblized segment exhibited the best performance in ride quality and uniformity of structural capacity. A life cycle cost analysis suggested that the unbonded concrete overlay was the most cost-effective segment, although the results were very close.

Hall, K. T., M. I. Darter, and S. H. Carpenter. 1995. ***Guidelines for Rehabilitation of Asphalt-Overlaid Concrete Pavements.*** **FHWA-IL-UI-247. Illinois Department of Transportation, Springfield, IL.**

This report presents guidelines for selection of rehabilitation strategies for asphalt-overlaid concrete (AC/PCC) pavements, summarizes the performance of second AC overlays of AC/PCC pavements on Illinois Interstates, and presents procedures for design of three types of overlays of AC/PCC pavements: second AC overlay; unbonded PCC overlay; and AC overlay of rubblized PCC with the existing AC removed. The overlay design procedures are based on the 1993 revised AASHTO overlay design procedures, customized for Illinois conditions. A practical catalog of rehabilitation designs is presented for AC/PCC pavements in various categories of condition, traffic level, and PCC pavement type.

Hall, K. T., M. I. Darter, and W. J. Seiler. 1993. "Improved Design of Unbonded Concrete Overlays." ***Proceedings, Fifth International Conference on Concrete Pavement, Design and Rehabilitation.*** **Purdue University, West Lafayette, IN.**

An unbonded concrete overlay is an excellent choice for long-lasting rehabilitation of a deteriorated concrete pavement. However, concerns over thickness and joint design of unbonded concrete overlays hinder their more widespread use. This paper reviews the existing approaches to unbonded overlay thickness design, examines actual field behavior and long-term performance of these overlays, and presents a framework for an improved design approach which relates design to performance. Four approaches to unbonded overlay design are considered: the traditional Corps of Engineers procedure, design of the overlay as a new pavement, and multilayer pavement design using either elastic layer theory or plate theory. These approaches are evaluated with respect to their theoretical bases, inherent assumptions, sensitivity to various design inputs, and compatibility with actual field performance of unbonded overlays.

Finite element analysis holds the most promise for an improved approach to bonded overlay design. Two-dimensional, plate theory-based multilayer finite element programs (or equations developed from them) are capable of realistically modeling most, but not all, of the complex factors which must be considered. Recommendations are given for further research needed to fully develop improved methods for unbonded overlay analysis and design.

Jiang, Y. 1998. "Performance of a Concrete Pavement Overlay." ***Concrete International,*** **Volume 20, No. 5. American Concrete Institute, Farmington Hills, MI.**

This study evaluated the performance of a concrete overlay project in Indiana, located on I-69 from SR-18 to 5.6 miles (9 km) north of SR-18. A 10-inch (255 mm) thick unbonded concrete overlay was placed in 1986 and 1987. The annual average daily traffic (AADT) on the four-lane highway was 18,870, with 28 percent truck volume. The original jointed concrete pavement was badly D-cracked (major surface distress due to freeze-thaw) and had been resurfaced with a bituminous overlay. The performance of the unbonded concrete overlay has been monitored since 1987. Pavement roughness, friction, and deflection were measured and visual inspections of the pavement distresses were conducted. This article presents the study results of the performance of the concrete overlay during the 8-year period after construction.

Kazmierowski, T. J. and H. J. Sturm. 1991. "Concrete Pavement Rehabilitation and Overlay: Ontario's Experience." ***Transportation Research Record 1307.*** **Transportation Research Board, Washington, DC.**

In 1989, the Ministry of Transportation of Ontario (MTO) initiated a demonstration contract to rehabilitate a freeway using various PCC pavement repair techniques in one direction and an unbonded PCC overlay in the opposing direction. The project site is a four-lane divided arterial with 30,000 vehicles per day, including 10 percent commercial vehicles. The existing pavement consisted of 230-mm JRCP with 21.3-m joint spacing constructed in 1963. The rehabilitation techniques used included full-depth repair, partial-depth repair, diamond grinding, and joint sealant replacement on the northbound lanes that had experienced moderate deterioration. The southbound lanes received a 180-mm-thick plain jointed unbonded PCC overlay to address the severe D cracking and spalling of all joints and cracks. Design and construction details and the performance of the pavement, before and after rehabilitation, are discussed in terms of load transfer efficiencies and pavement edge deflections based on falling weight deflectometer testing; roughness using the profilograph, the portable universal roughness device, and the automatic road analyzer; skid resistance using the ASTM brake force trailer; pavement condition ratings; and a crack survey. Observations of noise levels, traffic volumes, and surface texture are also presented.

Kazmierowski, T. J. and H. J. Sturm. 1993. "Performance of Ontario's First Major Rehabilitation and Overlay Project." ***Proceedings, Fifth International Conference on Concrete Pavement Design and Rehabilitation.*** **Purdue University, West Lafayette, IN.**

Currently, highway authorities are faced with the challenge of rehabilitating portland cement concrete (PCC) pavements on high-volume freeways. In 1989, the Ministry of Transportation of Ontario (MTO) initiated a demonstration contract to rehabilitate one such freeway using various PCC pavement repair techniques in one direction and an unbonded PCC overlay in the opposing direction. The rehabilitation project has now been in service for 3 years and a number of performance characteristics have been monitored. These include pavement roughness and skid resistance with testing completed on an annual basis while pavement load response, condition surveys, and noise emissions testing have been undertaken on an as-needed basis. The results of this testing and analysis are presented in this paper.

Kazmierowski, T. J. and H. Sturm. 1994. "Performance of Unbonded Concrete Overlay Project in Canada." ***Transportation Research Record 1449.*** **Transportation Research Board, Washington, DC.**

In 1989 the Ontario Ministry of Transportation constructed a project to demonstrate the feasibility of PCC rehabilitation techniques. The project incorporated the use of full-depth and partial-depth repairs, which was followed by diamond grinding and joint sealant replacement on the northbound lanes. The southbound lanes received a 180-mm-thick undoweled plain-jointed unbonded portland cement concrete overlay, which is the focus of this study. Following construction an extensive program was implemented to systematically monitor long-term performance. The program included FWD testing for corner-to-center deflection ratios and joint load transfer efficiencies, along with roughness and skid resistance testing and condition surveys. Roughness and skid resistance measurements have been completed on an annual basis, with FWD testing and condition surveys carried out as needed. The design and construction of the unbonded overlay are highlighted, and the results of the performance monitoring program carried out on this 4-year-old project are detailed.

Kazmierowski, T. J. and A. Bradbury. 1999. "Ten-Year Evaluation of a Concrete Pavement Rehabilitation Project in Ontario." ***Preprint Paper No. 99-0340.*** **78th Annual Meeting of the Transportation Research Board, Washington, DC.**

Until 1989 the traditional method of rehabilitating concrete pavements in Ontario was to remove the distressed concrete full depth or partial depth and replace it with asphaltic concrete. Although these repairs performed well in the short-term, in the long-term the hot mix patches would begin to distort and crack resulting in a rough ride. During the mid to late 1980s, significant advances in concrete pavement rehabilitation (CPR) techniques in North America were achieved allowing the restoration of rigid pavements with concrete. This maintained the integrity of the PCC slabs and the long-term benefits of concrete. A site was selected in the late 1980s to demonstrate various restoration techniques, including methods that had not previously been utilized in Ontario. At the time of construction, this project utilized the most comprehensive CPR techniques undertaken in Canada. The demonstration site is located on a 5 km urban arterial facility in London, Ontario, Highway 126. The highway was assumed by the City of London and is now known as Highbury Avenue. Rehabilitation techniques included partial and full depth repairs and diamond grinding on the northbound lanes and an unbonded overlay on the southbound lanes. Construction was completed late in the summer of 1989.

Khazanovich, L. and A. M. Ioannides. 1994. "Structural Analysis of Unbonded Concrete Overlays Under Wheel and Environmental Loads." ***Transportation Research Record 1449.*** **Transportation Research Board, Washington, DC.**

At present it is common to treat an overlaid concrete pavement system as a multilayered Kirchhoff plate. A number of finite-element computer programs use this idealization, which assumes that the original slab and the overlay have the same deflection profile, that is, that the two act as an effective, homogeneous plate. Environmental loads, however, are among the most significant factors, making the assumption of equality of the deflection profiles of slab and overlay unacceptable. A formulation proposed by Totsky is implemented into an existing finite-element program to account for the effects of layer separation and compressibility. The effects of combined traffic loading and temperature differentials on the stresses and deflections in the pavement system when the overlay curls away from the existing slab are examined. The product is a finite-element code, abbreviated ILSL2, which represents an extension of the ILLI-SLAB program. Practical illustrative examples of the use of ILSL2 pertaining to typical pavements are presented. When appropriate these results are compared with existing analytical or numerical solutions, including those obtained by using an earlier version of ILLI-SLAB. It was found that both layer compressibility and separation effects need to be accommodated in a reliable mechanistic model for unbonded concrete overlays. If these effects are neglected, significant errors may result.

Lewis, A. S. and J. P. Mohsen. 2000. "Instrumentation of an Unbonded PCC Overlay Pavement for Maturity Monitoring." ***Preprint Paper.*** **79th Annual Meeting of the Transportation Research Board, Washington, DC.**

To further investigate the use of the maturity method for opening pavement to traffic criteria, an unbonded portland cement concrete overlay pavement in Louisville, Kentucky was instrumented for maturity studies. This is an ongoing project that will also include a correlation of pulse velocity with in-place compressive strength. The pulse velocity portion of the project is not covered in this report. The purpose of this paper is to report the procedure followed in the development of the correlation curve for the concrete and the instrumentation of the pavement for measuring maturity. The sensor placement device developed at the University of Louisville worked with excellent accuracy and minimal concrete disturbance. A sandpit was used to cure the cylinders, which were used to verify the pavement strength estimations. The sandpit did not insulate the concrete in the cylinders as well as the pavement did, however, it did insulate them better than air curing. The current maturity equations appear to be very sensitive to the mix proportions of the concrete and the values of "activation energy" and "datum temperature" used. Therefore, any correlation curves used for opening pavement to traffic criteria should be developed using concrete that is as near as possible to the same concrete used in the pavement. An investigation is underway to determine the extent of the equations' sensitivity to the values of "activation energy" and "datum temperature" used. More study is needed to easily determine the appropriate values of these constants.

Minnesota Department of Transportation. 1993. *Unbonded Concrete Overlay Design Procedure.* Minnesota Department of Transportation, Maplewood, MN.

This document presents the Minnesota Department of Transportation's procedure for the design of unbonded concrete overlays of existing concrete pavements. The design procedures presented in the document are derived from previous work done by the Portland Cement Association and by the Corps of Engineers. Additional design information, such as joint design, drainage design, and traffic loading estimation, is also presented.

Owusu-Antwi, E. and L. Khazanovich. 1999. "Design and Construction Guidelines for Unbonded PCC Overlays." *Preprint Paper No. 99-0189.* 78th Annual Meeting of the Transportation Research Board, Washington, DC.

A method of rehabilitating rigid pavements is by resurfacing them with an unbonded portland cement concrete overlay. This involves the use of an interlayer to separate the action of the PCC overlay from the existing pavement. It is a cost-effective way of improving the structural strength of the existing pavement, and provides a new surface with improved riding quality as well. Other advantages of unbonded concrete overlays include their suitability for use on PCC pavements (or AC/PCC) without the need for extensive repairs to the existing pavement. The interlayer used also minimizes the occurrence of reflection cracking from discontinuities in the existing pavement. It can also serve as a leveling course that limits the chances of the resurfacing material from overrunning during construction, and it has been used successfully as a drainable layer in some States. Because unbonded overlays are relatively thick, typically ranging from 152 to 305 mm (6 to 12 in) for highway pavements, no special techniques are necessary during construction, and the overlays can usually be built with conventional paving methods and equipment. The thicker overlay also permits the application of conventional concrete pavement maintenance and rehabilitation techniques in future years to improve the performance of the pavement. With these advantages, pavement rehabilitation with unbonded concrete overlay is likely to become increasingly common in the coming years. To facilitate use of unbonded concrete overlays for rehabilitating PCC pavements, this paper presents reliable guidelines that have been developed for application by pavement engineers.

Saraf, C. L., J. C. Kennedy, K. Majidzadeh, and S. W. Dudley. 1991. "Life-Cycle Cost Analysis of Ohio Pavement Rehabilitation Demonstration Projects." *Transportation Research Record 1307.* Transportation Research Board, Washington, DC.

A suitable life cycle cost analysis procedure for comparing the economics of eight different projects that were treated with five different rehabilitation methods generally used in Ohio is described. Because the initial condition of each pavement included in the study was different, it was necessary to adjust for this condition so that different pavements could be compared on an equitable basis. Initial salvage value was used for this purpose. Also, the daily traffic was different on each project. Therefore, service cost index, defined as the ratio of daily traffic to life cycle cost (in multiples of $1,000) was used for comparing the benefits and costs of each project or rehabilitation method used in this study. Analysis of data used in this study indicated that composite overlays were more cost-effective than unbonded rigid overlays for pavements subjected to high levels of daily traffic. Unbonded rigid overlays were also relatively less cost-effective than asphalt concrete (AC) overlays for low levels of daily traffic conditions. Crack and seat with AC overlay was more cost-effective under medium-to-high levels than under medium-to-low levels of traffic. Concrete pavement restoration was most expensive. Results of the analysis indicated that the procedure described is a reasonable method of comparing the life cycle costs of rehabilitation methods used in Ohio.

Tyner, H. L., W. Gulden, and D. Brown. 1981. "Resurfacing of Plain Jointed Concrete Pavements." *Transportation Research Record 814.* Transportation Research Board, Washington, DC.

In 1975, the Georgia Department of Transportation placed a 1-mile concrete overlay test section on I-85 north of Atlanta. The test area consists of a 3-in continuously reinforced concrete pavement (CRCP) overlay, a 4.5-in CRCP overlay, a 6-in CRCP overlay, and a 6-in JPCP overlay. All overlays were placed in an unbonded condition, using a wax-based curing compound as a bond breaker. The primary objective was to determine the performance of various concrete overlay systems over a faulted jointed concrete pavement. Some 16 asphalt concrete overlay sections with various thicknesses and treatments were placed adjacent to the concrete overlays. The performance to date shows the importance of pre-overlay repair, including stabilization of slabs, full-depth repairs, and patching and spall repair. Both 6-in PCC overlays are both performing well, and it is recommended that this be considered the minimum for PCC overlays when there is heavy traffic.

Voigt, G. F., M. I. Darter, and S. H. Carpenter. 1989. "Field Performance Review of Unbonded Jointed Concrete Overlays." ***Transportation Research Record 1227.*** **Transportation Research Board, Washington, DC.**

This paper describes a nationwide pavement survey and evaluation of 14 unbonded concrete overlays. A comprehensive distress survey was performed, past traffic equivalent single-axle loads were estimated, and design, subgrade, and climatic data were obtained. The data were evaluated, and the results are summarized herein. Overall, unbonded concrete overlays have performed quite well with little deterioration to date. Specific conclusions are presented to aid in the future design of unbonded overlays.

Ultra-Thin Whitetopping Overlays

American Concrete Pavement Association (ACPA). 1999a. *Ultra-Thin Whitetopping*. Information Series IS100.02. American Concrete Pavement Association, Skokie, IL.

UTW is a relatively new process where a thin layer of concrete (2 to 4 in) is placed over a prepared surface of distressed asphalt. This special report is intended to give the state-of-the-practice on the design and construction of ultra-thin whitetopping (UTW). It presents information on the following topics: applications, history, materials, research and performance data, load-carrying capacity and mechanistic analysis, joint design, construction procedures, and repair.

American Concrete Pavement Association (ACPA). 1999b. *Construction Specification Guideline for Ultra-Thin Whitetopping*. Information Series IS120P. American Concrete Pavement Association, Skokie, IL.

This document provides guideline specifications useful for developing concrete project specifications for ultra-thin whitetopping pavement. The guidelines are written in the three-part section format of the Construction Specifications Institute, and include references to appropriate material standards, test methods, and specifications of the American Society of Testing and Materials (ASTM), the American Association of State Highway and Transportation Officials (AASHTO), and the Canadian Standards Association (CSA).

American Concrete Pavement Association (ACPA). 2000. *Repair of Ultra-Thin Whitetopping*. Publication PA397P. American Concrete Pavement Association, Skokie, IL.

Ultra-thin whitetopping (UTW), a 2 to 4 inch (50 to 100 mm) concrete overlay of existing asphalt pavements, is a viable and successful pavement rehabilitation strategy for intersections, ramps, low volume streets and highways, local roads, and light aircraft aprons. Since 1992, over 200 distresses asphalt pavements in at least 35 states have been rehabilitated with UTW. As UTW pavements age and carry traffic, repair may eventually be required. This document provides information on when UTW repairs should be conducted, and also describes the recommended step-by-step repair procedure.

Armaghani, J. M., and D. Tu. 1997. "Performance of Ultra-Thin Whitetopping in Florida." *Proceedings, Sixth International Purdue Conference on Concrete Pavement, Design and Materials for High Performance.* Indianapolis, IN.

Three experimental test tracks were constructed in Gainesville, Florida to evaluate design, construction and performance of the ultra-thin whitetopping (UTW). In Test Track 1, a 100-mm (4-in) thick UTW was placed on existing pavement composed of 38-mm (1.5-in) asphalt pavement and 150-mm (6-in) concrete base. The surface of the asphalt pavement was prepared in three ways: milling, broom cleaning, and applying a thin asphalt crack relief layer (CRL). Test Track 2 consisted of 75-mm (3-in) and 100-mm (4-in) thick sections on a 38-mm (1.5-in) asphalt pavement and a 163-mm (6.5-in) base layer of sand and Florida limerock material. The surface of the asphalt pavement was prepared in two ways: milling and broom cleaning. In Test Track 3, a 50-mm (2-in) thick concrete overlay was placed on 38-mm (1.5-in) milled asphalt pavement and 163-mm (6.5-in) base layer similar to that in Test Track 2. Joint spacings in Tracks 1 and 2 were 1.22 m x 1.22 m (4 ft x 4 ft) and 1.83 m x 1.83 m (6 ft x 6 ft). Test Track 3 had 0.92 m x 0.92 m (3 ft x 3 ft) and 1.22 m x 1.22 m (4 ft x 4 ft) joint spacings. Polypropylene fibers were used in Tracks 1 and 2. Polyolefin fibers were used in Track 3. High early strength concrete was designed for the UTW. Bond strength at the concrete-asphalt interface was generally above 1.4 MPa (200 psi). The test tracks were subjected to approximately 60,000 (18-kip) ESALs using a truck loaded with concrete blocks. Falling Weight Deflectometer (FWD) test results showed significant improvement in the structural capacity of the pavement after placement of the UTW. Frequent condition surveys indicated excellent performance of the three test tracks. The UTW is a viable option for rehabilitation and restoration of asphalt pavements.

Armaghani, J. M. and D. Tu. 1999. "Rehabilitation of Ellaville Weigh Station With Ultrathin Whitetopping." ***Transportation Research Record 1654.*** **Transportation Research Board, Washington, DC.**

The Florida Department of Transportation constructed the first ultrathin whitetopping (UTW) project at the Ellaville truck weigh station on I-10 in northwest Florida. This rehabilitation project included the placement of UTW on the existing asphalt pavement, which had experienced severe rutting problems. Layer thicknesses for the UTW were 80 mm and 100 mm. The joint spacings for the UTW panels were 1.2 m and 1.6 m. High early strength concrete was used in this project. Polypropylene fibers were included in the concrete for the sections on the west side of the weighing platform and plain concrete was used on the east sections. The joints on the east section were sealed with silicone sealant and the joints on the west section were left unsealed. Falling weight deflectometer tests and frequent condition surveys were performed on the projects. After 1 year of service and 1.1 million equivalent single-axle loads, the UTW shows good performance. Success and long-term performance of the UTW is highly dependent on the degree of bonding between the UTW and the asphalt base. During the study period, panels 1.2 m^2 and 1.6 m^2, concrete with and without fibers, sealed and unsealed joints, showed similar performance. It is predicted that the 1-year performance of the UTW at the weigh station is equivalent to 4.5 years of service at a medium traffic intersection.

Balbo, J. T. 1999. "Applications of High-Performance Concrete for Ultra-Thin Pavement Overlays (White Topping)." ***American Concrete Institute Special Report 186.*** **American Concrete Institute, Farmington Hills, MI.**

Employment of High Performance Concrete (HPC) for thin overlays construction for aged flexible pavements has become a reality during the 1990s, especially in the U.S. and some northwest European countries. While whitetopping old pavements was a technique employed from earlier decades of the twentieth century, the construction of ultra-thin concrete overlays (of 100 mm) for rehabilitation of pavements has been enhanced by the availability of technology for manufacturing HPC and the possibility of fast tracking. Ultra-thin whitetopping is a technique requiring several field conditions to be met concerning the old asphalt pavement in order to perform well as an overlay. They are full bond condition at the interface of HPC and asphalt concrete (generally provided by milling), asphalt concrete without fatigue cracking and rational joint spacing. All these factors, on the other hand, must be taken into account on the basis of the peculiar resistance of the HPC to be used. Within this context, this paper presents a study with regard to the HPC strength to be achieved for ultra-thin whitetopping purposes. It is supported by a numerical analysis based on a finite element solution for slabs-on-grade and takes into account the elastic properties for both HPC and old asphalt, as well as slab dimensions and the load critical position. An international review of HPC applied on whitetoppings around the world, that includes recent work in this field, is also presented.

Balbo, J., D. Pereira, and A. Severi. 2001. "Behavior and Performance of UTW on Thin Asphalt Pavement." ***Proceedings,*** **Seventh International Conference on Concrete Pavements, Orlando, FL.**

During the spring of 1999, two sections of ultra-thin whitetopping (UTW) were built in an urban street in São Paulo City over a thin 45 mm asphalt layer. The UTW was constructed at a bus stop within the University campus, defining two panels of squared 0.6 and 1.0 m and 95 mm slabs. A high strength concrete was applied and the test sections were fully instrumented with top and bottom thermal resistors and strain gages. The instrumentation allowed the analysis of temperatures and thermal gradients on slabs as well as the curling deformation induced by these gradients. Temperature measurements showed daytime thermal gradients up to 11.7 °C and nighttime gradients up to –4.2 °C. Concrete stresses due uniquely to curling reached very low values in all seasons and were deemed negligible. Open to traffic since November 1999, 19 months later no cracks or faulting were observed on UTW sections submitted to a daily traffic of 120 buses and trucks making it possible, on the basis of measured flexural stresses on slabs, to predict a good performance of the UTW even over a thin 45 mm asphalt layer.

Cable, J. K., J. D. Grove, and M. Heyer. 1997. "Ultrathin Pavements Making the Grade." ***Proceedings, Sixth International Purdue Conference on Concrete Pavement, Design and Materials for High Performance.*** **Indianapolis, IN.**

In 1994, the Iowa Department of Transportation (Iowa DOT), Federal Highway Administration and Iowa State University joined forces to construct an 11.6 km ultrathin portland cement concrete overlay project. This marked the beginning of the use of thin whitetopping for rehabilitation of existing asphalt roadways on primary highways in Iowa. The research was designed to evaluate long term performance of ultrathin whitetopping. Four major variables were chosen for evaluation and 41 test sections, each 213 m in length, were constructed. Some 24 additional sections were used to transition from one set of parameters to the next. Research efforts over the last two years have consisted of visual distress surveys conducted on a quarterly bases and measurement of overlay interface strains measured three times per year at some 35 sites on the project. This paper provides insight into the changes in condition of the test sections which have occurred since construction over two years ago. The overall performance to date has been very good. Isolated areas of distress have been notes and are beginning to provide some trends in the data.

Cable, J. K., J. M. Hart, and T. J. Ciha. 1999. ***Thin Bonded Overlay Evaluation.*** **Construction Report, Iowa DOT Project HR-559. Iowa Department of Transportation, Ames, IA.**

Ultra-thin whitetopping (UTW) has evolved as a viable rehabilitation technique for deteriorated asphalt concrete pavements. Numerous UTW projects have been constructed and tested, enabling researchers to identify key elements contributing to the performance of UTW pavements. This project continues the investigation of UTW performance through the construction of a field project constructed on a 7.2-mile stretch of roadway located on Highway 21 between Highway 212 and U.S. 6, near Belle Plaine. Variables investigated include asphalt concrete surface preparation, PCC thickness, synthetic fiber reinforcement usage, joint spacing, and joint sealing. This report describes the planning, equipment selection, and construction of the project that was built in 1994.

Cable, J. K., J. M. Hart, and T. J. Ciha. 2001. ***Thin Bonded Overlay Evaluation.*** **Final Report, Iowa DOT Project HR-559. Iowa Department of Transportation, Ames, IA.**

In 1994, the Iowa Department of Transportation constructed a 7.2-mi portland cement concrete overlay project in Iowa County on Iowa Highway 21. The research work was conducted in cooperation with the Department of Civil Engineering and the Federal Highway Administration under the Iowa Highway Research Board project HR-559. The project was constructed to evaluate the performance of an ultrathin concrete overlay over a 5-year period.

The experiment included variables of base surface preparation, overlay depth, joint spacing, fiber reinforcement, and sealed and nonsealed joints. The project was instrumented to measure overlay/base interface temperatures and strains. Visual distress surveys and deflection testing were also used to monitor performance. Coring and direct shear testing was conducted three times over the course of the research period.

Results of the testing and monitoring are identified in the report. The experiment was very successful and the results provide an insight into construction and design needs to be considered in tailoring a portland cement concrete overlay to a performance need. The results also indicate a method to monitor bond with nondestructive methods.

Cable, J. K., J. Hart, and T. Ciha. 2001. "The Ultrathin Whitetopping Option." ***Proceedings,*** **Seventh International Conference on Concrete Pavements, Orlando, FL.**

In 1994, Iowa built 11.6 km (7.2 mi) of ultrathin whitetopping on a segment of Iowa Highway 21. Research was carried out under Iowa Highway Research Board Project Number HR-559 for 5 years and TR-432 for an additional 5 years. When considering ultrathin as an overlay option, is how long is the performance life and what are the rehabilitation options? Some 41 sections of pavement including three overlay depths, four joint patterns, three surface preparations, and fiber usage were constructed. The methods employed in the rehabilitation and success of each are discussed. Deflection testing, visual surveys, coring, and direct shear testing have continued over the seven years. Analysis of the data have identified ways to estimate accurately the overlay performance in terms of bond strength retained.

Caestecker, C., T. Lonneux, and F. Haemels. 2001. "UTW: Test Section at Vilvoorde on the R22 in Belgium." ***Proceedings,*** **Seventh International Conference on Concrete Pavements, Orlando, FL.**

UTW (Ultra Thin Whitetopping) is a very recent and innovative application to repair a degraded asphalt pavement. The repair technique tries to bond the two layers (overlay in concrete and lower lying asphalt pavement) to obtain a composite structure. The technique consists of milling off the rutted asphalt layer in a suitable thickness and covering it with a thin cement concrete layer (< 15 cm) that bonds perfectly with the underlying asphalt layer. UTW is characterized by a good bonding between concrete and asphalt and by the short distances between the joints (between 0.60 and 1.80 m). For the first time in Flanders, a test section was carried out near the intersection of the Woluwelaan with the Houtemsesteenweg at Vilvoorde. A survey of the existing pavement, the project, the successive construction phases as well as the most important points of particular interest are described in this article.

Churilla, C. J. 1998. "Ultra-Thin Whitetopping." ***Public Roads.*** **Volume 62, Number 2. Federal Highway Administration, Washington, DC.**

Several years ago, a new technique/process in which 50-100 mm of high-strength, fiber-reinforced concrete is placed over a milled surface of distressed asphalt concrete pavement was introduced, and this process, called ultra-thin whitetopping (UTW), has proven to be a low-cost, effective, and fairly simple solution for repairing bumpy, rutted, potholed pavements. UTW is designed for low-speed traffic areas or areas with a lot of stop-and-go traffic, such as street intersections, bus stops, or toll booths. UTW requires significantly less time to construct than conventional pavement maintenance, and repairs last much longer. Given its success in these limited applications, UTW is now being considered for a range of other applications. A few states have pilot projects using UTW as an alternative to asphalt overlays for interstate roads. To help state and local highway agencies make decisions about using UTW for other applications, the Federal Highway Administration recently launched a joint UTW research effort with the American Concrete Pavement Association that will evaluate critical design factors affecting the performance of UTWs.

Cole, L. W. 1997. "Pavement Condition Surveys of Ultrathin Whitetopping Projects." ***Proceedings, Sixth International Purdue Conference on Concrete Pavement, Design and Materials for High Performance.*** **Indianapolis, IN.**

Portland cement concrete overlays of existing asphalt pavements, often called "whitetopping," is an established rehabilitation method used extensively throughout the United States. In 1994, a survey of projects documented 189 concrete resurfacings of asphalt pavement on highways, airfields, streets, and county roads. These whitetopping projects were geographically spread throughout the Untied States with projects in 33 states. With few exceptions the concrete overlay thickness ranged from 100 mm (4 in.) (city streets) to 450 mm (18 in.) (airfields).

Cole, L. W. 1999. "Performance of Ultrathin Whitetopping Roadways." ***Materials and Construction: Exploring the Connection, Proceedings of the Fifth ASCE Materials Engineering Congress.*** **American Society of Civil Engineers, Reston, VA.**

Since 1992, over 200 ultrathin whitetopping (UTW) projects have been built in the U.S. to rehabilitate distressed asphalt pavements. This paper presents the performance of 10 of the earliest UTW projects. All but one of the projects includes concrete containing synthetic fibers. Pavement condition surveys using MicroPaver protocol have been made in 1995, 1996, 1997, and 1998. The survey results are presented as well as observations on the performance of these 10 UTW projects.

Cole, L. W. 1999. "Thin Bonded Concrete Overlays of Asphalt Pavement: Experience With Ultra-Thin Concrete Overlays (Whitetopping) in the USA." ***Routes/Roads,*** **Issue No. 302. Permanent International Association of Road Congresses, La Defense Cedex , France.**

The paper discusses a technique for rehabilitating distressed asphalt pavements in concrete overlay, a techniques referred to in North America as "whitetopping." The overlay is either plain jointed concrete, with or without fibers, or continuously reinforced concrete. This technique represents one of the most recent and innovative applications in this field; it emerged in the United States; and is primarily used for low-volume roads and for municipal applications, such as bus stops, parking areas, crossings.

Crawley, A. B. and J. M. Pepper. 1999. "Application of Fiber Reinforced Concrete for Thin And Ultra-Thin Whitetopping On I-20 in Mississippi." ***Preprint Paper 99-0040.*** **78th Annual Meeting of the Transportation Research Board, Washington, DC.**

Whitetopping is a term for a pavement overlay or inlay where portland cement concrete is bonded to a hot mix asphalt (HMA) pavement. This composite pavement system relies on the strengths of both materials acting as a unit. Ultra-thin whitetopping (UTW) is a pavement rehabilitation option that has been used in recent years at intersections where the HMA pavement has experienced excessive rutting due to plastic flow under slow moving, heavy wheel loads. Fibrillated plastic fibers are usually used in the concrete. The first UTW project that established the viability of this concept was constructed in Louisville, Kentucky in 1991. The Tennessee Department of Transportation has constructed several UTWs from 1992 to 1995.

Typical thickness of UTW range from 50 mm to 125 mm and require saw cuts at short intervals to ensure curling and warping stresses do not cause debonding of the whitetopping and to reduce the amount of flexural stress. Joint spacings generally follow the ratio of 12/1 for saw cutting interval/thickness of slab. A new plastic fiber being marketed by the 3M Company is reputed to make it possible to extend the joint spacing/thickness ratio to 120/1 or more. The 3M fibers are 50 mm long and 0.63 mm in diameter (hereinafter referred to as 50/63 fiber). Since the cost of saw cuts is a significant part of the cost of a UTW, the extra cost of the 50/63 fiber may be offset by the decreased amount of sawing. Some northern states seal the joints in UTW and fewer numbers of joints will have an even greater impact under these conditions.

This paper describes a thin whitetopping inlay constructed on Interstate 20 near Jackson, Mississippi, in August 1997. This construction was done in conjunction with the U.S. Army Engineers Waterways Experiment Station (USAEWES) as a test installation for a jointly sponsored study between the Corps of Engineers and 3M. Sponsors of this thin interstate whitetopping (TIW) are the Mississippi Department of Transportation (MDOT), the Federal Highway Administration (FHWA), the USAEWES, and the Mississippi Concrete Industries Association (MCIA).

Cumberledge, G., P. King, and S. Hawk. 1996. ***Ultra-Thin Portland Cement Concrete Overlay—Construction Report.*** **Construction Report, Project 37P-0001-001. Pennsylvania Department of Transportation, Harrisburg, PA.**

Ultra-thin portland cement concrete overlays have been used in other states on full depth bituminous pavements. Areas of particular interest in using this technology are those subject to rutting and shoving, such as ramps and intersections. This report discusses the details and construction procedures used to implement this technology on a reinforced concrete ramp that was overlaid with bituminous concrete. The performance of this application of an ultra-thin portland cement concrete overlay (whitetopping) will be monitored.

Dumitru, N. I., M. Hossain, and J. Wojakowski. 2002. "Construction and Performance of Ultra-Thin White Topping in Kansas." ***Preprint Paper.*** **81st Annual Meeting of Transportation Research Board, Washington, DC.**

A suburban city street in Kansas was rehabilitated with a 50 mm (2 in) portland cement concrete overlay, commonly known as ultra-thin white topping (UTW). The construction and performance of this UTW project have been described in this paper. The project, constructed in the spring of 1995, incorporated the following design features: 0.9 m x 0.9 m (3 ft x 3 ft) panels versus 1.2 m x 1.2 m (4 ft x 4 ft) panels, plain versus fiber reinforced concrete, and sealed versus unsealed joints. The project has performed fairly well to date although some test sections needed periodic maintenance. Experience on this project shows that the UTW overlay can be easily built with conventional equipment and locally available materials. UTW permits a skid-resistant finish to be applied. Excellent smoothness can also be obtained although the slab thickness is very small. Corner cracking appears to be the most dominant distress type, though it was observed that a strong bond existed between the concrete and asphalt layers even for the cracked panels. Joint spacing has a significant effect on performance. The sections with smaller joint spacing appeared to perform better. The performance of the sections with fibers in concrete was inconclusive. Also, joint sealing did not appear to affect the performance.

Federal Highway Administration (FHWA). 1999. *Full-Scale Accelerated Testing of Ultra-Thin Whitetopping Pavements*. FHWA-RD-99-087. Federal Highway Administration, McLean, VA.

In spring 1998, under a cooperative research agreement with the Federal Highway Administration (FHWA), the American Concrete Pavement Association (ACPA) built eight full-scale lanes of Ultra-thin Whitetopping (UTW) overlay on previously tested asphalt concrete pavements. The sections were placed at the FHWA's Pavement Testing Facility in McLean, Virginia. FHWA began testing the 15-m-long by 4-m-wide sections in May 1998 with one of its two Accelerated Loading Facility (ALF) machines. The second ALF machine was incorporated into the UTW program in November 1998. The tests, which include the collection of data on UTW performance and response, are scheduled to continue through November 1999. The goals of the study are to validate design equations and mechanistic analysis models (i.e., finite element versus layer theory) recommended in ACPA's design methods for UTW and to document the performance of UTW. This TechBrief provides a summary of the design, instrumentation, and construction of the eight experimental sections of UTW and the results of the project to date.

Gucunski, N., V. Ganji, N. Vitillo, K. Tabrizi, and A. Maher. 1999. "FE Analysis Based Prediction Equations for Ultra Thin Whitetopping (UTW)." *Preprint Paper No. 99-0397*. 78th Annual Meeting of the Transportation Research Board, Washington, DC.

A three-dimensional finite element model of an AC pavement with UTW overlay is presented. In the model the pavement layers are described by several layers of isotropic solid elements, except the subgrade that is described by a set of discrete springs. Effects of possible debonding on the AC-UTW interface and cracking along slab joints are described by layers of anisotropic solid elements. A parametric study is conducted to evaluate influence of a number of parameters on maximum stresses in AC and UTW layers due to single and tandem axle and temperature loadings. The parameters include: material and geometrical properties of pavement layers, UTW-AC interface bonding, UTW joint cracking, presence of construction joints and position of loading. The single most important parameter found is the degree of AC-UTW bonding. Other important parameters include the thickness and elastic moduli of AC and UTW layers, that can be conveniently joined in the description by the corresponding flexural rigidities. The finite element analysis results are used to develop maximum stress prediction equations for both UTW and AC layers for both bonded and unbonded conditions. Use of the prediction equations in the pavement design is discussed. The presented design requires traffic data for the project, fatigue equations for UTW and AC, and the maximum stress prediction equations.

Harper, M. and C. L. Mathias. 1998. "Concrete Inlay of an Adelaide Intersection." *Transport Proceedings, Conference of the Australian Road Research Board*. Australian Road Research Board, Vermont, Australia.

In January 1998, an innovative concrete pavement surface treatment was successfully undertaken in Adelaide South Australia, as the first of six proposed trial sites. Located at the signalized intersection approach of Osmond Terrace and Magill Road, the concrete inlay replaced the asphalt surfacing layer which had developed severe shoving and rutting due to the combined effects of standing and slow moving heavy traffic during hot summers. This paper describes the design methodology and construction processes of the polypropylene fibre reinforced thin concrete inlay treatment applied to Osmond Terrace. Details of the adaptation of technology developed in the USA, commonly referred to as Ultra Thin Whitetopping (UTW) are provided, together with an evaluation of the structural and functional performance of the treatment during the initial 5 months of service. The treatment costs and the refinements to design and construction details which are proposed for future trial sites are also discussed.

Jeppson, V. R. Warren, and D. R. Pieper. 1997. "Paving Intersections for Durability." *Concrete International*, Volume 19, No. 5. American Concrete Institute, Farmington Hills, MI.

There are three methods of paving intersections in concrete: full depth repair, where the existing pavement is removed and replaced with concrete; whitetopping, where a depth of 4 to 5 in of asphalt is removed and replaced with concrete; and ultrathin whitetopping techniques may be only 2.5 in thick and still have 2-3 times the life of the best asphalt overlay. Furthermore, initial costs can be competitive with asphalt. The ultrathin techniques are still under development, but show promise for the future. An intersection rehabilitated at night during weekends with fast track techniques can be back in service for the Monday morning rush hour. Clearly, concrete is the answer for intersection pavements.

King, P.E. 1997. *Ultra-Thin Portland Cement Concrete Overlay.* Report No. FHWA-PA-96-002F. Pennsylvania Department of Transportation, Harrisburg, PA.

Ultra-thin portland cement concrete overlays have been used in other states on full depth bituminous pavements. Areas of particular interest in using this technology are those subject to rutting and shoving, such as ramps and intersections. In October of 1995, this technology was implemented on a reinforced concrete ramp that was overlaid with bituminous concrete. This report evaluates the performance of the ultra-thin portland cement concrete overlay (whitetopping) thus far. Also, an attempt is made to evaluate the cost effectiveness of this practice as an alternative for the maintenance of high volume traffic areas which are subject to excessive rutting and shoving.

Mack, J. W., L. W. Cole, and J. P. Mohsen. 1993. "Analytical Considerations for Thin Concrete Overlays on Asphalt." *Transportation Research Record 1388.* Transportation Research Board, Washington, DC.

An analytical study was undertaken to account for the performance of a thin concrete overlay on an asphalt concrete (AC) pavement built in Louisville, Kentucky, in September 1991. Initial investigations and theoretical studies based on conventional concrete theory indicated that the pavement should have failed after its first few loadings. However, this did not happen. After 11 months of use, with very heavy traffic (400 to 600 garbage trucks per day, 5 1/2 days a week), the pavement has provided much better service than was anticipated, suggesting that a bond developed between the concrete overlay and underlying AC that greatly improved the pavement's performance. Some direction for future research is provided.

Mack, J. W., L. D. Hawbaker, and L. W. Cole. 1998. "Ultrathin Whitetopping: State-Of-The-Practice for Thin Concrete Overlays of Asphalt." *Transportation Research Record 1610.* Transportation Research Board, Washington, DC.

Ultrathin whitetopping (UTW) is a concrete overlay of a distressed asphalt pavement, 50 to 100 mm thick, with close joint spacing. The overlay is specifically bonded to the existing asphalt pavement. It may or may not contain fibers. By bonding the UTW to the existing asphalt surface, the UTW forms a composite pavement section with the underlying asphalt, which reduces the stresses in the concrete layer. This composite pavement section delivers the longer life and durable performance characteristics of concrete pavement and is cost competitive with ordinary asphalt. Many areas are discussed, but not all questions about UTW are answered. A snapshot of the current state of the practice as it now stands is presented.

Mack, J. W., C. L. Wu, S. Tarr, and T. Refai. 1997. "Model Development and Interim Design Procedure Guidelines for Ultra-thin Whitetopping Pavements." *Proceedings, Sixth International Purdue Conference on Concrete Pavement, Design and Materials for High Performance.* Indianapolis, IN.

Ultra-thin whitetopping (UTW) involves placing very thin concrete slabs (50 to 100 mm (2 to 4 in. thick)) on old asphalt pavements to form bonded, or partially bonded, composite pavements. The reduction in thickness is justified by the use of high quality concrete with relatively high strength, close joint spacings, and bond between the concrete and the existing asphalt pavement. Although the use of UTW has been increasing, no structural design method is currently available. Because of this, the pavements have probably been either over-designed or under-designed. To address this deficiency, the Portland Cement Association and the American Concrete Pavement Association initiated a research project to develop an ultra-thin whitetopping design procedure. The objective of this research was to develop a mechanistic UTW pavement design procedure, the results of which are summarized herein.

Marks, V. J. and M. Heyer. 1995. *Ultra Thin PCC Overlays: Iowa 21 Whitetopping.* Final Report, Project HR-559. Iowa Department of Transportation, Ames, IA.

An 11.6 km portion of IA 21 in Iowa County was divided into 65 different test sections of a PCC overlay over an existing AC surface with thicknesses of 50 mm, 100 mm, 150 mm, and 200 mm. The joint spacings for these sections were 0.6 m, 1.2 m, 1.8 m, 3.7 m, and 4.6 m. Joints were sealed if the thickness of the pavement was over 100 mm, unless specified. Two types of polypropylene fibers, monofilament and fibrillated, were added to the conventional PCC mix for designated sections. Three additional sections consisted of an asphalt overlay for comparison with the concrete overlay. Three different base preparations were used on the project, consisting of: patching and scarifying, patching only, and cold-in-place recycling. Sensors were placed in various test sections to measure the temperature and strain during and after construction of the overlay. Pullout tests were also conducted at various locations. Beam cylinders were made for each of the PCC mixes and tested for flexural and compressive strengths. Evaluation of the performance will be conducted through December 31, 1999.

Mohsen, J. P. 1995. "Use of Concrete-Asphalt Composites in Pavements." ***Proceedings, Restructuring: America and Beyond Structures Congress.*** **American Society of Civil Engineers, New York, NY.**

Ultrathin concrete overlays' use in asphalt was examined in an experiment. The experiment included the construction of two thicknesses of concrete overlay on the access road to a waste disposal landfill. The access roads serves 400-600 trucks per day, 5½ days per week. It is viewed as an accelerated test as this rate of truck loading is 20 to 100 times greater than the number of trucks experienced by many lower volume roads in the US. This paper summarizes the experiment, subsequent analytical work, and presents directions for future research.

Nishizawa, T., Y. Murata, and T. Nakagawa. 2002. "Curling Stress In Concrete Slab Of Ultra-Thin Whitetopping Structure." ***Preprint Paper 02-2922.*** **81st Annual Meeting of Transportation Research Board, Washington, DC.**

A basic concept of design of ultra-thin whitetopping (UTW) is the fatigue analysis of concrete slabs. The analysis requires not only load stress but also curling stress due to temperature gradients in the concrete slab. In this study, the effects of temperature gradients in the concrete slab, rigidity of the asphalt layer, joint spacing and joint stiffness on the curling stress in the concrete slab were investigated using the results of temperature measurement and FWD test conducted on a test pavement and three dimensional finite element simulation. The results of the simulation showed that the composite effect between the concrete slab and the asphalt layer has a significant effect on the reduction of the curling stress at the bottom of the slab. Also, it was found that the bending stiffness at the joint increases the curling stress in the case of short joint spacing.

Petersson, O. and J. Silfwerbrand. 1993. "Thin Concrete Overlays on Old Asphalt Roads." ***Proceedings, Fifth International Conference on Concrete Pavement Design and Rehabilitation.*** **Purdue University, West Lafayette, IN.**

This paper reports on a joint venture project involving the Stockholm Regional Road Administration, Cementa AB (a Swedish cement producer), and the Swedish Cement and Concrete Research Institute (CBI). The project aims at building a test road in the Stockholm area in 1993 that incorporates a thin concrete surface over an existing asphalt surface. Various calculation models are presented to determine stresses in such a system.

Rajan, S., J. Olek, T. L. Robertson, K. Galal, T. Nantung, and W. J. Weiss. 2001. "Analysis of Performance of the Ultra-Thin Whitetopping Subjected to Slow Moving Loads in an Accelerated Pavement Testing Facility." ***Proceedings,*** **Seventh International Conference on Concrete Pavements, Orlando, FL.**

Ultra-Thin Whitetopping (UTW) is rapidly emerging as a technology that can be used for the rehabilitation of deteriorated pavements. To investigate the performance of UTW when they are placed over flexible pavements and subjected to a slow moving load, four whitetopping mixtures were placed over a milled asphalt surface in the Accelerated Pavement Testing (APT) facility of the Indiana Department of Transportation (INDOT) Research Division in West Lafayette, Indiana in the Fall of 1999. This paper presents the response of the UTW to repeated loading, including analysis of stresses and strains, with the goal of identifying the factors influencing the performance of UTW. The data was analyzed to determine the maximum strains and their location, the degree of bonding between the UTW and the existing pavement, and the pavement performance under repeated loading. The study described in this paper is a part of a larger effort to develop preliminary design guidelines for UTW construction in Indiana including development of the semi-empirical model for performance prediction of UTW.

Rasmussen, R. O., W. J. Wilde, J. M. Ruiz, J. Sherwood, and J. Mack. 2001. "An Analysis of Ride Quality of the Ultra-Thin Whitetopping Overlays at the FHWA Accelerated Loading Facility." ***Proceedings,*** **Seventh International Conference on Concrete Pavements, Orlando, FL.**

In 1998, eight test lanes of ultra-thin whitetopping (UTW) were constructed over existing hot-mix asphalt pavements at the Federal Highway Administration's Accelerated Loading Facility (ALF) located at the Turner-Fairbank Highway Research Center. Various combinations of thickness, joint spacing, fiber reinforcement, and asphalt base type were used. As part of the experimental data collection effort, pavement profiles were obtained periodically during the ALF loading history. From this information, ride statistics were calculated including Ride Number (RN) and the International Roughness Index (IRI). This paper describes a comprehensive analysis of this information from a number of perspectives.

Rasmussen, R. O., B. F. McCullough, J. M. Ruiz, J. Mack, and J. A. Sherwood. 2002. "Identification of Pavement Failure Mechanisms at the FHWA ALF UTW Project." ***Preprint Paper.*** **81st Annual Meeting of Transportation Research Board, Washington, DC.**

In 1998, eight test lanes of ultra-thin whitetopping (UTW) were constructed over existing hot-mix asphalt (HMA) pavements at the Federal Highway Administration's Accelerated Loading Facility (ALF) located at the Turner-Fairbank Highway Research Center in McLean, Virginia. Various combinations of thickness, joint spacing, fiber reinforcement, and HMA base type were used. In spring 2000, the loading experiment of these pavements was completed, and the task was started to analyze the behavior and performance. This paper serves to summarize some of the pavement distresses that were observed at the ALF, and to identify the failure mechanisms that are hypothesized. In this, this paper adds to the state-of-the-knowledge with respect to the actual life cycle of UTW pavements.

Risser, R. J., S. P. LaHue, G. F. Voigt, and J. W. Mack. 1993. "Ultra-Thin Concrete Overlays on Existing Asphalt Pavement." ***Proceedings, Fifth International Conference on Concrete Pavement Design and Rehabilitation.*** **Purdue University, West Lafayette, IN.**

In 1991, an experimental project incorporating ultra-thin whitetopping concrete overlays was built on a Louisville, Kentucky landfill access road. Two sections with different thicknesses were built, one 2 inches thick and the other 3.5 inches thick. The experiment also employed unconventional 6 foot and 2 foot joint patterns. This paper documents the design and construction of the project, and also includes a summary of the current performance of the various pavement sections. As of September 1992, both sections on the project are performing well and have carried over 585,000 equivalent single-axle load applications (ESALs). The designs have out-lived performance predictions made by conventional pavement theory and models.

Silfwerbrand, J. 1997. "Whitetoppings—Swedish Field Test and Recommendations." ***Proceedings, Sixth International Purdue Conference on Concrete Pavement, Design and Materials for High Performance.*** **Indianapolis, IN.**

In order to find methods to restore rutted asphalt pavements and increase their rutting resistance, field tests have been carried out with thin, high strength concrete overlays (whitetoppings) placed on top of the milled asphalt pavement. The concrete overlay was limited to 70 mm. Both plain and steel fibre reinforced concrete have been studied. Two different joint spacings have been investigated: 1.25 and 3.5 m. Also a test section with an overlay consisting of an open asphalt filled with high strength cementitious mortar has been included in the investigation. After three years in service, the test section performance is promising. Besides test results, the paper provides recommendation for the construction of whitetoppings and suggests needs for future research.

Speakman, J. and H. N. Scott. 1996. "Ultra-Thin, Fiber-Reinforced Concrete Overlays for Urban Intersections." ***Transportation Research Record 1532.*** **Transportation Research Board, Washington, DC.**

An industry partnership involving state and local highway officials, ready mix concrete producers, and fiber, admixture, and cement suppliers paved the way for extensive testing of a new concrete technology in Tennessee. Known as ultra-thin whitetopping (UTW), this experimental repair procedure involves placing a layer 50 to 75 mm (2 to 3 in) thick of high-strength concrete over milled asphalt. Crucial to a long life for the thin concrete is a high synthetic fiber content, normally 1.4 kg/cu m (3 lb/cu yd) with joints spaced at a minimum of 0.92 to 1.22 m (3 to 4 ft) on center. From 1992 to 1995, seven cities in Tennessee participated in UTW demonstrations. The field tests were so successful that synthetic fiber-reinforced UTW is now being bid on many state and local projects. Presently, the primary use of UTW is to rehabilitate high average daily traffic intersections. Other potential uses include overlays of residential streets, parking lots, and airport aprons.

Vandenbossche, J. M. and D. Rettner. 1998. *The Construction of US-169 and I-94 Experimental Ultra-Thin Whitetopping Sections in Minnesota.* State Project No. 7106-60. Minnesota Department of Transportation, Maplewood, MN.

Ultra-thin whitetopping refers to placing a thin concrete overlay directly on an existing distressed asphalt pavement. For long-term performance, the overlay must bond to the underlying asphalt so that the two layers respond in a monolithic manner, thereby reducing load-related distress. Although two ultra-thin whitetopping projects have been constructed by the Minnesota Department of Transportation, the purpose of this project is to further evaluate how whitetoppings perform in Minnesota and to determine what design features are desirable to optimize the life of the pavement. This study includes two projects, one constructed on US-169 in Elk River and the other on I-94 at the Minnesota Road Research (Mn/ROAD) test facility. The mix design, instrumentation, and construction of these projects are described in this paper.

Vandenbossche, J. M. and D. L. Rettner. 1999. "One-Year Performance Summary of Whitetopping Test Sections at the Mn/ROAD Test Facility." *Proceedings, 1999 Accelerated Pavement Testing International Conference*, Reno, NV.

A 345-mm (13.5-in) asphalt pavement on I-94 at the Minnesota Road Research test facility (Mn/ROAD) was rehabilitated with a fiber-reinforced, ultra-thin whitetopping (UTW) concrete overlay in October 1997. Interstate 94 is a heavily trafficked roadway with approximately 1 million ESAL applications per year. Although this is not a typical application for UTW, it provided the opportunity to monitor the performance of the overlay under accelerated loading conditions. The test section was sub-divided into six test cells with various thicknesses, joint patterns, and fiber types. The test cells were instrumented with dynamic and static strain, temperature, and moisture sensors. The purpose of this research project is to measure the static and dynamic response of the pavement under various applied and environmental loading conditions to optimize the design of UTW.

Dynamic strain was collected in conjunction with falling weight deflectometer data and traffic loading at various times throughout the year. Temperature, moisture, and static strain data have been collected continuously since construction. The collected data have been used to determine the location and to quantify the magnitude of the maximum strain produced by both environmental and applied loads for various UTW designs. The strains can be used to predict the expected mode of failure and performance life of each test cell. Based on this, an optimum UTW design can be identified.

Vandenbossche, J. M. 2001. "The Measured Response of Ultra-Thin and Thin Whitetopping to Environmental Loads." *Proceedings*, Seventh International Conference on Concrete Pavements, Orlando, FL.

The technique of whitetopping asphalt concrete pavements with a thin (102-mm to 152-mm thick) or ultra-thin (51-mm to 102-mm thick) concrete overlay is becoming more common. The increase in the use of thin whitetopping (TW) and ultra-thin whitetopping (UTW) has amplified the need for a comprehensive design procedure. The purpose of this research project is to develop a better understanding of the behavior of thin and ultra-thin whitetopping by measuring the responses of the pavements to various environmental loadings.

A 345-mm asphalt concrete pavement on I-94, at the Minnesota Road Research (Mn/ROAD) facility, was whitetopped with a fiber-reinforced concrete overlay in October 1997. The experimental design features six test cells with various thicknesses, joint patterns and types of fibers. Each cell is instrumented with dynamic and static strain, temperature and moisture sensors. An evaluation of the data over time indicates seasonal changes occur in the quality of bond between layers, which has implications for changes in load-related stress over time. Both thermal and moisture changes in the overlay have a significant effect on the shape of the slab. Strain measurements revealed the complexity of the relationship between changes in temperature and/or moisture and slab deformation. This relationship must be clearly defined before an analysis of the response of the overlay to an applied load can begin. This study helped in characterizing these relationships.

Vandenbossche, J. M. and A. J. Fagerness. 2002. "Performance and Repair of Ultra-Thin Whitetopping: The Minnesota Experience." ***Preprint Paper.*** **81st Annual Meeting of Transportation Research Board, Washington, DC.**

In 1997, the Minnesota Department of Transportation constructed several thin and ultra-thin whitetopping test cells at the Minnesota Road Research (Mn/ROAD) facility. The test cells varied in overlay thickness from 76-mm (3-in) to 152-mm (6-in). The joint spacing of these cells ranged from 1.2-m by 1.2-m (4-ft x 4-ft) to 3.1-m by 3.7-m (10-ft x 12-ft). After 3.5 years and 4.7 million ESALS, both temperature- and load-related distresses were observed on the 76-mm (3-in) and 102-mm (4-in) thick sections. There were no noticeable distresses in the 152-mm (6-in) sections. Typical distresses included corner breaks, transverse cracks, and reflective cracks. Different techniques for repairing ultra-thin whitetopping were investigated. It was determined that using a milling machine with tungsten carbide teeth to remove the concrete greatly reduced the time required per repair. Various techniques were also used to deter reflective cracking. This included the use of various bond-breaking materials and full-depth sawing at strategic locations along the longitudinal joint to prevent cracks from propagating into adjacent panels at misaligned transverse joints. Four of the six sections had PSIs greater than 3.5 before the repairs, showing a good level of performance has been maintained after 4.7 million ESALS. The two sections that exhibited the largest drop in PSI were the overlays with 1.2-m x 1.2-m (4-ft x 4-ft) panels. The repairs made in sections containing 1.2-m x 1.2-m (4-ft x 4-ft) panels have brought the PSI back up to an acceptable level (PSI>3). The thin and ultra-thin whitetopping test sections at Mn/ROAD have shown that whitetopping is a viable rehabilitation alternative for asphalt pavements. The importance of choosing an optimum panel size was exhibited. It has also been shown that, when necessary, it is easy to repair ultra-thin whitetopping sections. Techniques for repairing each type of distress are summarized.

Wu, C. L., S. M. Tarr, A. Ardani, and M. J. Sheehan. 1998. "Instrumentation and Field Testing of Ultrathin Whitetopping Pavement." ***Preprint Paper.*** **77th Annual Meeting of the Transportation Research Board, Washington, DC.**

As part of an effort for developing design guidelines for ultra-thin whitetopping pavement systems, instrumentation and field testing was conducted on three different project sites. The first test section was constructed in early 1995 and was located at a parking area in a general aviation airport, Spirit of St. Louis Airport in Missouri. Strain gages were installed both in the concrete and on concrete slab surface, and at various locations. Temperature gradients in the concrete and asphalt layers were measured using thermocouples embedded in the pavement. Different joint conditions were included as a variable in this experiment. Truck loads were applied at various locations and stresses under and around the loads were measured. Surface profiles were also measured to study slab movements induced by temperature changes in the pavement layers. In summer of 1996, through joint sponsorship of Colorado Department of Transportation (CDOT) and the Portland Cement Association (PCA), two whitetopping test sections in Colorado were instrumented and tested. Additional variables considered in this two sites included slab thickness, joint spacing and different asphalt surface preparation.

Wu, C. L., S. M. Tarr, T. M. Refai, M. A. Nagi, and M. J. Sheehan. 1997. ***Development of Ultra-Thin Whitetopping Design Procedure.*** **PCA Research and Development Serial No. 2124. Portland Cement Association, Skokie, IL.**

A mechanistic design procedure for ultra-thin whitetopping pavement was developed through a comprehensive study involving extensive field load testing and theoretical analysis of whitetopping responses. A three-dimensional finite element model was developed to calculate critical pavement stresses and strains for partially bonded asphalt and concrete layers. The 3-D model was correlated to a two-dimensional finite element model so that an extensive parametric analysis could be performed. Equations predicting critical stresses and strains were derived based on the results of the parametric study. Two types of pavement failure were considered in this procedure – portland cement concrete fatigue under corner loading and asphalt concrete fatigue under joint loading. Temperature induced stresses and strains are also included in this procedure. The design example presented shows that the technique gives reasonable results. However, this procedure is viewed as the first generation design method. It should be refined as more field performance data (especially long-term performance data) become available.

Wu, C. L., S. Tayabji, and J. Sherwood. 2001. "Repair of Ultra-Thin Whitetopping Pavements." ***Preprint Paper.*** **80th Annual Meeting of the Transportation Research Board, Washington, DC.**

Since its inception in 1991, ultra-thin whitetopping (UTW) has moved beyond the experimental stage and has become a viable pavement rehabilitation alternative. With the use of UTW rapidly increasing, the next logical step in the development of this technology is the development of UTW repair and rehabilitation techniques. The construction of the UTW test sections at the FHWA's Turner-Fairbank Highway Research Center (TFHRC), subjected to accelerated loadings, provided an excellent opportunity for the evaluation of potential UTW pavement repair methods. Panel removal and replacement was selected to repair some of the distressed panels of the test sections. This paper describes those repair activities that were conducted.

Wu, C. L., S. D. Tayabji, M. Sheehan, and J. Sherwood. 2001. "Performance and Repair of UTW Pavements." ***Proceedings,*** **Seventh International Conference on Concrete Pavements, Orlando, FL.**

Since its inception in the early 1990s, ultra-thin whitetopping (UTW) technique has rapidly developed into a viable pavement rehabilitation alternative for deteriorated asphalt pavement. The development of the mechanistic design procedure in 1997 represented a milestone in the development of this technology. Over 150 UTW pavements have been built in the last decade. As the UTW technology matures and with many existing UTW projects in service, it is essential to review the performance of existing UTW pavements to understand their behavior under traffic and to establish repair and rehabilitation techniques once distresses occur. The purpose of this paper is to present the results of studies conducted dealing with UTW pavement performance reviews and repair and rehabilitation experiences. The effectiveness of using currently available concrete pavement repair and rehabilitation procedures is explored. The paper includes detailed performance descriptions of several existing UTW pavements in Georgia and Tennessee and the UTW repair and rehabilitation projects conducted at the Federal Highway Administration's (FHWA's) Pavement Testing Facility and in several states.

Whitetopping Overlays

American Concrete Pavement Association (ACPA). 1991. *Guidelines for Concrete Overlays of Existing Asphalt Pavements (Whitetopping).* Technical Bulletin TB-007P. American Concrete Pavement Association, Arlington Heights, IL.

These guidelines cover the construction of a concrete overlay over an existing asphalt pavement. This procedure is commonly termed whitetopping. Whitetopping entails pre-overlay repair, correction of major surface profile deviations, and placement of the concrete overlay. The concrete pavement overlay is constructed directly on the existing asphalt surface. Whitetopping overlays have been built on all types of facilities dating back to 1918. Airports, interstate highways, primary roads, and even secondary roads, city streets and parking areas have been significantly improved with concrete over asphalt. Concrete pavement provides a stronger, more durable surface than asphalt. Concrete also improves surface drainage characteristics by eliminating unsafe deviations, such as asphalt rutting and shoving.

American Concrete Pavement Association (ACPA). 1998. *Whitetopping—State of the Practice.* Engineering Bulletin EB210P. American Concrete Pavement Association, Skokie, IL.

The purpose of this publication is to describe the "state-of-the-practice" for the design and construction of concrete overlays on existing asphalt pavements, commonly referred to as "whitetopping." Variations of whitetopping include:

- Conventional whitetopping, defined as a concrete overlay of 4 in or more placed directly on an old asphalt pavement.
- Concrete inlay, which is a concrete overlay placed in a trench milled out of a thick asphalt pavement.
- Ultra-thin whitetopping (UTW), a concrete overlay of thickness 4 in or less placed on an old asphalt surface that is prepared to enhance the bond between the concrete and the asphalt.

For the construction agency or owner and the public, whitetopping an existing asphalt pavement provides many benefits, including superior service, long life, low maintenance requirements, low life-cycle cost, improved safety, and environmental benefits. In this publication, design and construction guidelines and practices are covered for both conventional whitetopping and UTW overlays.

Cable, J. K. 1996. *Impact of Pavement Type on County Road Systems.* Report No. RP340.01P. Portland Cement Association, Skokie, IL.

A study of three representative counties in Iowa, of equal size and approximately equal paved mileage, was conducted to evaluate the pavement selection policies employed by each government. The sample counties utilized asphaltic concrete, portland cement concrete, or a combination of the two surface types to meet paved system needs. Visual distress surveys were conducted in each county to measure the current condition of each mile of paved roadway. The results were compared using the MICRO PAVER pavement management system to develop pavement condition index values for each mile of paved surface. Historical pavement mileage, maintenance costs, and construction cost data was collected in each county by surface type for the period of 1955 through 1993. The results of this study indicate that the decision to use portland cement concrete as the surface of choice for new pavement construction and whitetopping of existing pavements resulted in improved performance. The data indicate that this decision results in decreased unit construction and maintenance costs when placed on an annual cost basis over the life of the pavement.

Charonnat, Y., J. Chauchot, and A. Sainton. 1989. "Cement Concrete Overlays on Flexible Pavements in France." ***Proceedings, Fourth International Conference of Concrete Pavement Design and Rehabilitation.*** **Purdue University, West Lafayette, IN.**

Before 1973, pavement overlays were made for heavy traffic with 15 cm of bituminous base course or 25 cm of hydraulic gravel and 8 cm of asphalt concrete. Between 1973 and 1976, experimental sites were carried out in cement concrete laid by vibration. This deliberate government action was aimed at 1) maintaining a variety of techniques guaranteeing a minimum of competition on one hand, and the survival of a technicality on the other, 2) developing a low energy and low aggregate consumption technique, 3) restricting the use of imported material (bitumen) while using a well know industrial material (portland cement), and 4) having suitable solutions for different site conditions. Five experimental sites were constructed to evaluate the performance of concrete overlays of existing flexible pavements. This paper presents a summary of the construction activities and also describes the performance of these sections after more than 10 years of service.

Gisi, A. J. 1985. "Portland Cement Concrete Pavement Overlay Over a Bituminous Pavement." ***Transportation Research Record 1040.*** **Transportation Research Board, Washington, DC.**

The interstate pavement structure in Kansas was built using bituminous pavement in the western half of the state and portland cement concrete (PCC) pavement in the eastern half of the state. The initial stage construction completed in 1970 on Interstate 70 in western Kansas consisted of a 10-in asphaltic concrete pavement. At the time of the second stage, 10 years later, the pavement exhibited signs of load and nonload associated cracking. A 3-in second stage overlay had been planned during the initial design, but based on current conditions, traffic loadings, and distress, a structural design called for a 5-in asphaltic concrete overlay. Because overlays generally do not control reflective cracking for appreciable periods of time, several additional alternatives were considered, including an 8-in portland cement concrete overlay that was ultimately selected based on a life-cycle cost analysis. This paper presents the rehabilitation design alternatives that were considered and describes the design and construction of the new overlay.

Grove, J. D., E. J. Engle, and B. J. Skinner. 1996. ***Bond Enhancement Techniques for PCC Whitetopping.*** **Report HR-341. Iowa Department of Transportation, Ames, IA.**

This research was initiated in 1991 as a part of a whitetopping project to study the effectiveness of various techniques to enhance bond strength between a new portland cement concrete (PCC) overlay and an existing asphalt cement concrete (ACC) pavement surface. A 1,676 m (5,500 ft) section of county road R16 in Dallas County was divided into 12 test sections. The various techniques used to enhance bond were power brooming, power brooming with air blast, milling, cement and water grout, and emulsion tack coat. Also, two sections were planed to a uniform cross-section, two pavement thicknesses were placed, and two different concrete mix proportions were used. Major conclusions from the study are as follows: (1) Bond Strength Differences—Milling increased bond strength versus no milling. Tack coat showed increased bond strength versus no tack coat. Planing, air blast, and grouting did not provide noticeable improvements in bond strength; nor did different PCC types or thicknesses affect bond strength significantly. (2) Structure—Structural measurements correlated strongly with the wide variation in pavement thicknesses. They did not provide enough information to determine the strength of bonding or the level of support being provided by the ACC layer. Longitudinal cracking correlated with PCC thicknesses and with planing. (3) Bond Over Time—The bond between PCC and ACC layers is degrading over time in the outside wheel path in all of the sections except tack coat (section 12). The bond strength in the section with tack coat was lower than the others, but remained relatively steady.

Grove, J. D., G. K. Harris, and B. J. Skinner. 1992. *Bond Contribution to Whitetopping Performance on Low Volume Roads.* Report No. HR-341. Iowa Department of Transportation, Ames, IA.

This research was initiated in 1991 as a part of a whitetopping project to study the effectiveness of various techniques to enhance bond strength between a new portland cement concrete (PCC) overlay and an existing asphalt cement concrete (ACC) pavement surface. A 1,676 m (5,500 ft) section of county road R16 in Dallas County, Iowa, was divided into 12 test sections. The various techniques used to enhance bond were power brooming, power brooming with air blast, milling, cement and water grout, and emulsion tack coat. As a part of these bonding techniques, two pavement thicknesses were placed; two different concrete proportions were used; and two sections were planed to a uniform cross-slope. The research found that bond was developed in every section, regardless of the bond enhancement technique used. The underlying ACC does contribute to the composite structure and can be considered in the design. The sections where the underlying ACC contributed the greatest amount of structure to the composite pavement were not the sections with the highest bond strength.

Grove, J. D., G. K. Harris, and B. J. Skinner. 1993. "Bond Contribution to Whitetopping Performance on Low-Volume Roads." *Transportation Research Record 1382.* Transportation Research Board, Washington, DC.

Research was initiated in 1991 as a part of a whitetopping project to study the effectiveness of various techniques to enhance bond strength between a new portland cement concrete (PCC) overlay and an existing asphalt cement concrete (ACC) pavement surface. A 1676-m (5,500-ft) section of County Road R16 in Dallas County, Iowa, was divided into 12 test sections. Techniques used to enhance bond were power brooming, power brooming with air blast, milling, cement and water grout, and emulsion tack coat. As a part of these bonding techniques, two pavement thicknesses were placed, two concrete proportions were used, and two sections were planed to a uniform cross slope. The research found that bond was developed in every section, regardless of the bond enhancement technique used. The underlying ACC does contribute to the composite structure and can be considered in the design. The sections on which the underlying ACC contributed the greatest amount of structure to the composite pavement were not the sections with the highest bond strength.

Ramakrishnan, V. 1997. "A New Material (Polyolefin Fiber Reinforced Concrete) for the Construction of Pavements and White-topping of Asphalt Roads." *Proceedings, Sixth International Purdue Conference on Concrete Pavement, Design and Materials for High Performance.* Indianapolis, IN.

This paper presents the performance characteristics of a newly developed polyolefin fiber reinforced concrete. These polyolefin fibers were used for the first time in South Dakota for the construction of a full-depth pavement and whitetopping on scarified asphalt pavement. The mixture proportions used, the procedure used for mixing, transporting, placing, consolidation, finishing, tining, and curing of the concretes are described. The feasibility of using this high performance polyolefin fiber reinforced concretes in the construction of highway structures has been established and presented in this paper. These polyolefin fibers performed well in the mixing operation without causing any balling or segregation. This new polyolefin fiber reinforced concrete with enhanced fatigue, impact resistance, modulus of rupture, ductility, and toughness properties is particularly suitable for the construction of pavements and white-topping.

Ramakrishnan, V. and S. Kakodkar. 1995. *Evaluation of Non-Metallic Fiber Reinforced Concrete in PCC Pavements and Structures.* Report SD94-04. South Dakota Department of Transportation, Pierre, SD.

This interim report presents the construction and performance evaluation of four structures, namely a full-depth pavement, a thin bridge-deck overlay, a jersey barrier, and white-topping on scarified asphalt pavement, constructed with a new type non-metallic fiber reinforced concrete. This is the first time this newly developed polyolefin fiber reinforced concrete had been used in construction. The mixture proportions used, the quality control tests conducted for the evaluation of the fresh and hardened concrete properties, the procedure used for mixing, transporting, placing, consolidation, finishing, tining, and curing of the concretes are described. Periodic inspections of these structures were made and this report includes the results of these inspections. Tentative conclusions and recommendations are given.

Strand, D., C. N. MacDonald, V. Ramakrishnan, and V. N. Rajpathak. 1996. "Construction Applications of Polyolefin Fiber Reinforced Concrete." ***Proceedings, Materials for the New Millennium, 1996 Materials Engineering Conference.*** **New York, NY.**

This paper presents construction applications of a recently developed non-metallic polyolefin fiber reinforced concrete (NMFRC). This NMFRC was used for the first time in the repair and rehabilitation of bridge decks and pavements in South Dakota. It was used for construction of thin bridge deck overlay, jersey barrier, white-topping on scarified asphalt pavement, full-depth bridge deck slab, and full depth pavement. The mixture proportions used, the procedure used for mixing, transporting, placing, consolidation, finishing, tining, and curing of the concretes are described. The feasibility of using this high performance polyolefin fiber reinforced concrete in the construction of highway structures has been established and presented in this paper. This new polyolefin fiber reinforced concrete with enhanced fatigue, impact resistance, modulus of rupture, ductility and toughness properties is particularly suitable for the construction of thin bridge deck overlays, bridge deck slab and white-topping.

Tarr, S. M., M. J. Sheehan, and A. Ardani. 2000. "Mechanistic Design of Thin Whitetopping Pavements in Colorado." ***Transportation Research Record 1730.*** **Transportation Research Board, Washington, DC.**

A mechanistic design procedure was developed for the State of Colorado to determine appropriate concrete thicknesses for thin (5 to 7 in) whitetopping overlays on asphalt pavements. Field testing was conducted to evaluate critical load locations for whitetopping with joint spacing up to 12 ft. The load-induced flexural strains were used to calibrate fully bonded stresses computed using finite element analysis techniques to partially bonded stresses measured in the field. For each test section, load testing was conducted throughout the course of a day to develop a temperature correction for the critical stresses derived for zero temperature gradient (zero slab temperature curling). Equations predicting the critical concrete flexural stresses and asphalt concrete strains for use in whitetopping were developed. A mechanistic design procedure is described which allows the evaluation of trial whitetopping thicknesses and joint spacings. The procedure computes the concrete and asphalt fatigue life for specific material properties. Iterations are required to determine the appropriate parameters which provide the required design life for both concrete and asphalt layers. In addition to the design procedure, the effect of surface preparation during construction was studied by comparing identical slabs constructed on milled and unmilled asphalt. It was concluded that existing asphalt pavement should be milled and cleaned prior to concrete placement for an overall reduction of 25 percent in the critical load-induced stresses. However, new asphalt, such as that placed in repair patches, should not be milled prior to concrete placement to avoid a 50 percent increase in critical load-induced stresses.

Tarr, S. M., M. J. Sheehan, and P. A. Okamoto. 1998. ***Guidelines for the Thickness Design of Bonded Whitetopping Pavement in the State of Colorado.*** **Report CDOT-DTD-R-98-10. Colorado Department of Transportation, Denver, CO.**

This report summarizes the development of procedures for the thickness design of bonded whitetopping pavement in the state of Colorado. Included in this report is an overview of the selected sites, design parameters, instrumentation, data acquisition and analysis. The parameters studied included joint spacing, concrete flexural strength, asphalt thickness, subgrade modulus of elasticity, concrete and asphalt modulus of elasticity, design equivalent single axle load (ESAL) and temperature gradient. Equations were developed to predict the critical stresses and asphalt strains. A mechanistic design procedure is described which allows the evaluation of trial whitetopping thickness and joint spacing. A modified procedure was also developed incorporating an empirical approach based on the number of ESALs. Based on this research a minimum subgrade modulus of 150 pci is required along with an asphalt thickness of 5 in (12.7 cm). As with the AASHTO procedure, the method is not too sensitive to the number of ESALs.

Traweek, L. G. 1985. "The Concrete Resurfacing of Existing Asphalt on I-30 in Rockwall, County, Texas." ***Transportation Research Record 1040.*** **Transportation Research Board, Washington, DC.**

Interstate 30 is a controlled-access freeway that has deteriorated to such an extent that it has become a maintenance problem. Before this highway could be restored to a first class facility, however, there would be problems with the design of the pavement structure, traffic control, and construction. The existing asphalt highway was built over expansive clays and, after considering several alternatives, concrete resurfacing was selected as the rehabilitation option. The methods of offsetting the longitudinal contraction joint onto the shoulder and using concrete shoulders were incorporated to lower maintenance costs and better distribute heavy wheel loads near the pavement edges. Traffic control would be accomplished by reconstructing the south frontage road, diverting eastbound traffic onto it, and constructing the eastbound freeway. The westbound traffic could then be diverted to the newly constructed eastbound pavement and the westbound pavement could then be constructed. However, there were problems with the at-grade intersections of the frontage road, the alignment and typical section of the frontage road, and the areas of the frontage road that are prone to flooding. The 38-ft wide pavement is being constructed full width in one pass. The production is controlled by the plant capacity and not by the paving operations. The full width pass slows the speed of the concrete placement enough to cause some problems with finishing and texturing.

Webb, R. D. and N. J. Delatte. 2000. "Performance of Whitetopping Overlays." ***Preprint Paper No. 00-1068.*** **79th Annual Meeting of the Transportation Research Board, Washington, DC.**

Concrete overlays of asphalt pavements (whitetopping) have been used for pavement rehabilitation since 1918. Conventional whitetopping overlays are generally 100 mm (4 in) or more in thickness and are designed on the assumption that there is no bond between the new concrete pavement and the existing asphalt. More recently, ultrathin whitetopping overlays (UTW), less than 100 mm (4 in) thick, have been designed to bond to the asphalt. The development of this bond is critical to the performance of these overlays. In this paper, the historical development of both types of overlays is discussed, as well as factors that affect behavior and performance, and recommendations for future research. Design factors include condition of existing asphalt pavement, condition of other pavement layers, traffic and design period, concrete properties, joint spacing, and load transfer. For UTW, construction factors that affect performance include surface preparation and bond, concrete placing, finishing, and curing, joint construction, and early opening to traffic. Conventional whitetopping has provided excellent service for over eight decades. The UTW experience of the last ten years indicates that bonded whitetopping has even greater potential.

Wu, C. and M. Sheehan. 2002. ***Instrumentation and Field Testing of Whitetopping Pavements in Colorado and Revision of the TWT Design Procedure.*** **Interim Report, CDOT-DTD-R-2002-3. Colorado Department of Transportation, Denver, CO.**

Whitetopping has recently been generating considerable interest and greater acceptance as an approach to asphalt pavement rehabilitation. A number of thin whitetopping (TWT) and ultra-thin whitetopping (UTW) pavement test sections have been constructed during the past 10 years, and the overlays have demonstrated considerable advantages as a pavement rehabilitation technique. In 1996, the Colorado Department of Transportation (CDOT) sponsored a research project to develop a mechanistic design procedure for TWT pavements. Construction Technology Laboratories, Inc. (CTL) installed the instrumentation, conducted the load testing on the instrumented test sections, performed a theoretical analysis, and developed a TWT design procedure for CDOT. Many variables were considered in the construction of the test sections, including concrete overlay thickness, slab dimensions, existing asphalt layer thickness, different asphalt surface preparation techniques, and the use of dowel bars and tie bars. Based on the original design procedure development, there are several observations and conclusions regarding the use of TWT pavements for rehabilitation that should be examined more extensively with a supplemental investigation. The items include subgrade support conditions, required thickness of asphalt beneath the concrete layer, and effects of variable joint spacings.

To address this need, new TWT pavement test sections were constructed during 2001 in conjunction with a TWT project constructed by CDOT on SH 121 near Denver. This provided an opportunity to instrument and load test additional TWT test sections and use the data to calibrate and verify the existing observations and design procedure. The overall objectives of this project are to instrument, load test, and monitor the performance of the new and original TWT test sections to supplement and confirm the results of the 1996 study.

Yu, H. T., L. Khazanovich, M. I. Darter, and A. Ardani. 1998. "Analysis of Concrete Pavement Responses to Temperature and Wheel Loads Measured From Instrumented Slabs." ***Transportation Research Record 1639.*** **Transportation Research Board, Washington, DC.**

The structural response of jointed plain concrete pavement slabs was evaluated using data obtained from instrumented slabs. The instrumented slabs were a part of newly constructed jointed plain concrete overlay that was constructed on existing asphalt concrete pavement on I-70 in Colorado, near the Kansas-Colorado border. The instrumentation consisted of dial gauges for measuring curling deflections at the slab corner and longitudinal edge and surface-mounted strain gauges for measuring load strains at the longitudinal edge at midslab. The through-thickness temperature profiles in the pavement slabs were also measured at 30-min intervals during the field test. Analysis of the field data showed that the instrumented slabs had a considerable amount of built-in upward curling and that concrete slabs on a stiff base can act completely independent of the base or monolithically with the base, depending on the loading condition. The built-in upward curling of the slabs has the same effect as negative temperature gradients. These findings suggest that the effects of temperature gradients on the critical edge stresses may not be as great as previously thought and that the corner loading, in some cases, may produce more critical conditions for slab cracking. Another important finding of this study is that a physical bond between pavement layers is not required to obtain a bonded response from concrete pavements.

PCC Pavement Rehabilitation Strategy Selection

American Concrete Pavement Association. 1997. *The Concrete Pavement Restoration Guide.* Technical Bulletin TB002P. American Concrete Pavement Association, Skokie, IL.

A variety of rehabilitation strategies are available for addressing deficiencies in concrete pavements. This publication provides guidelines for selecting cost-effective rehabilitation strategies, relying upon experience gained from past projects. Emphasis is placed on a systematic approach to assessing the pavement and identifying suitable rehabilitation techniques.

Darter, M. I. and K. T. Hall. 1990. *Structural Overlay Strategies for Jointed Concrete Pavements, Volume IV—Guidelines for the Selection of Rehabilitation Alternatives.* FHWA-RD-89-147. Federal Highway Administration, McLean, VA.

A major field study and evaluation has been conducted into the effectiveness of three structural overlay types for portland cement concrete (PCC) pavements. These include sawing and sealing asphalt concrete (AC) overlays of PCC pavements, cracking and seating PCC pavements prior to AC overlay, and constructing a thin bonded PCC overlay on top of the existing PCC pavement. Condition surveys, deflection testing, and roughness measurements were performed on a total of 55 sections. The performance of these sections was evaluated and the effectiveness of each overlay type analyzed. Based on the field data, guidelines were developed for the use of structural overlays.

This volume provides detailed guidelines and case studies prepared specifically for the practicing engineer as an aid in the evaluation and rehabilitation of jointed concrete pavements. Feasibility guidelines are given for restoration, resurfacing, and reconstruction alternatives in terms of constructibility, future life, and life cycle costs. New prediction models are developed for bonded PCC overlays, sawing and sealing and AC overlays, and cracking and seating and AC overlay. The EXPEAR program was extensively modified to include the above rehabilitation alternatives and improved predictive models and to provide for much easier usage by the practicing engineer for evaluation and rehabilitation. Detailed rehabilitation case studies are presented that will be of interest to the practicing engineer.

Darter, M. I. and K. T. Hall. 1991. "Case Studies in Rehabilitation Strategy Development." *Transportation Research Record 1307.* Transportation Research Board, Washington, DC.

The performance and rehabilitation of jointed concrete pavements was recently investigated in a major national field study. Among the products of the research were detailed guidelines prepared to assist practicing engineers in determining when restoration was likely to be cost-effective, and when a structural overlay was required. The database for this project included 95 sections of joint reinforced concrete pavement and joint plain concrete pavement in their first performance period. Thirteen of these sections were selected for use as case studies in rehabilitation strategy development. The sections were selected to cover the range of four major climatic zones of the United States, as well as a range of pavement conditions from good to poor (assessed on the basis of cracking, joint deterioration, joint faulting, pumping, and serviceability). The EXPEAR computer program was used to evaluate the current condition of each pavement section, predict its future condition without rehabilitation, and compare the predicted performance and cost-effectiveness of various rehabilitation strategies. The case studies demonstrated that restoration is the most cost-effective alternative for pavements that are structurally adequate.

Hall, K. T., J. M. Conner, M. I. Darter, and S. H. Carpenter. 1989. *Rehabilitation of Concrete Pavements, Volume III—Concrete Pavement Evaluation and Rehabilitation System.* Report No. FHWA-RD-88-073. Federal Highway Administration, McLean, VA.

Extensive field, laboratory, and analytical studies were conducted into the evaluation and rehabilitation of concrete pavements. Field studies included over 350 rehabilitated pavement sections throughout the U.S., and the construction of the two field experiments. A laboratory study was conducted on anchoring dowel bars in full-depth repairs. Analyses of field and laboratory data identified performance characteristics, improved design and construction procedures, and provided deterioration models for rehabilitated pavements. A concrete pavement advisory system was developed to assist engineers in project level evaluation and rehabilitation.

This volume presents a comprehensive concrete pavement evaluation and rehabilitation advisory system for jointed plain, jointed reinforced, and continuously reinforced concrete pavements.

Hall, K. T., C. E. Correa, S. H. Carpenter, and R. P. Elliott. 2001. ***Rehabilitation Strategies for Highway Pavements.*** **Final Report, NCHRP Project 1-38. Transportation Research Board, Washington, DC.**

This project was conducted to develop a process for the selection of appropriate rehabilitation strategies for the ranges of pavement types and conditions found in the United States. A review of the pavement rehabilitation practices of the State DOTs was conducted, along with a review of the literature available on pavement evaluation, rehabilitation techniques, and rehabilitation strategy selection.

Although all State DOT agencies are engaged in pavement rehabilitation, fairly few of them have any more than the most simple and general guidelines for the selection of pavement rehabilitation strategies. The rehabilitation strategy selection procedures used by the various highway agencies differ in their details, but typically consist of 1) data collection, 2) pavement evaluation, 3) selection of rehabilitation techniques, 4) formation of rehabilitation strategies, 5) life-cycle cost analysis, and 6) selection of one pavement rehabilitation strategy from among the alternatives considered. This report provides a step-by-step process for project-level evaluation of pavements in need of rehabilitation, for the selection of rehabilitation techniques believed to be appropriate, and for the development of rehabilitation strategies expected to be feasible and cost-effective. As the pavement field's ability to predict rehabilitation performance improves, this process may be further refined and customized to the needs of the individual highway agencies.

Morian, D. A. and G. C. Cumberledge. 1997. "Techniques for Selecting Pavement Rehabilitation Strategies: Pennsylvania Case Studies." ***Transportation Research Record 1568.*** **Transportation Research Board, Washington, DC.**

The design and construction of pavement rehabilitation projects is one of the greatest challenges afforded today's pavement engineers. All project information pertinent to the performance of a pavement section is needed to select, design, and successfully construct a rehabilitation strategy. Case studies of three rehabilitation projects constructed by the Pennsylvania Department of Transportation are provided. Each case study uses a different rehabilitation strategy. The process for selecting the appropriate requirements for each one is discussed. The pavement performance for each rehabilitated pavement section is reviewed. The pavement performance information is tied back to the original criteria for project strategy selection and design to evaluate the success of the rehabilitation in addressing the issues that necessitated the rehabilitation and providing quality pavement performance.

Voigt, G. F. and M. J. Knutson. 1989. "Development and Selection of the Preferred 4R Strategy." ***Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation.*** **Purdue University, West Lafayette, IN.**

This paper reviews the major steps to be followed in selecting the proper rehabilitation activity for a particular pavement. These steps include pavement data collection, project evaluation, selection of feasible alternatives, preparation of preliminary designs, life cycle cost analysis, preparation of detailed plans, and implementation. In most cases, the choice of rehabilitation strategy, like most choices involving money, will boil down to the price tag. The key to determining the most cost-effective solution is applying a proper life-cycle cost analysis, choosing the appropriate interest rate for the funding agency, considering all 4R options, and considering non-pavement related factors.

Walls, J. and M. R. Smith. 1998. ***Life-Cycle Cost Analysis in Pavement Design—Interim Technical Bulletin.*** **FHWA-SA-98-079. Federal Highway Administration, Washington, DC.**

This document recommends procedures for conducting life-cycle cost analysis (LCCA) of pavement, provides detailed procedures to determine work zone user costs, and introduces a probabilistic approach to account for the uncertainty associated with LCCA inputs. The report begins with a discussion of the broad fundamental principles involved in an LCCA. It discusses input parameters and presents simple examples of traditional LCCA in a pavement design setting. It discusses the variability and inherent uncertainty associated with input parameters, and provides recommendations on acceptable ranges for the value of time as well as discount rates. It explores the use of sensitivity analysis in traditional LCCA approaches. User costs are a combination of delay, vehicle operating costs, and crash costs. Each of these components is explored and procedures are presented to determine their value. Given the power and sophistication of today's computers and software, simulation techniques such as Monte Carlo are recommended for incorporating variability associated with LCCA inputs into final results.

Wang, L., M. I. Darter, K. T. Hall, Y. Lu, and D. L. Lippert. 1997. "Improved Methodology for Developing a Long-Range Pavement Rehabilitation Program." ***Transportation Research Record 1568.*** **Transportation Research Board, Washington, DC.**

Current methodologies for developing, evaluating, and improving long-range rehabilitation programs may be classified as manual or analytical. The major limitation of manual methods is that although they are practical, they are not cost-effective because of the difficulties of comparing potential cost/benefit trade-offs among alternatives. Analytical methods, on the other hand, are more cost-effective but often not practical, because they oversimplify the real problem into a mathematical model and do not allow engineers and managers to be involved in the decision-making process. A new methodology and new concepts to overcome these major deficiencies were developed and implemented. The first of the new concepts is the use of graphical data interpretation and "visual thinking." The second is that pavement performance evaluations should be comprehensive and quickly accessible. The third is that human knowledge should be involved in the decision-making process through a user-friendly interface. The fourth is that the relationships among the components of the methodology should be flexible rather than rigid. Finally the process should be efficiently repeatable so that a satisfactory long-term rehabilitation program can be developed.

Wotring, D. C., G. Y. Biladi, N. Buch, and S. Bower. 1998. "Pavement Distress and Selection of Rehabilitation Alternatives: Michigan Practice." ***Transportation Research Record 1629.*** **Transportation Research Board, Washington, DC.**

The Michigan Department of Transportation (MDOT) practice regarding the preservation, rehabilitation, and preventive maintenance actions for rigid, flexible, and composite pavements is presented and discussed. For each pavement type, the causes of distress and the corresponding MDOT fix alternatives are also presented. Examples of the MDOT practice regarding the selection of maintenance and rehabilitation alternatives for rigid, flexible, and composite pavements are also presented.

APPENDIX B

GLOSSARY

APPENDIX B. GLOSSARY

Aggregate Interlock – The mechanical interactions of aggregate particles at the abutting joint faces of adjacent slabs. Sufficient aggregate interlock helps transfer traffic loading from one slab to another.

Analysis Period – The time period over which competing pavement design alternatives are compared.

Base Course – The layer of a specified material of designed thickness placed directly beneath the pavement surface to provide support to the surface course.

Bondbreaker – See separator layer.

Bonded PCC Overlay – A portland cement concrete (PCC) overlay bonded directly to an existing PCC pavement to form a monolithic structure.

Bonding Agent – A cementitious or other material placed on an existing portland cement concrete (PCC) pavement prior to bonded PCC overlay placement to promote bonding between the two layers.

Continuously Reinforced Concrete Pavements (CRCP) – A portland cement concrete (PCC) pavement system characterized by no transverse joints and containing continuous longitudinal reinforcement.

Conventional Whitetopping – The placement of a PCC overlay on an existing HMA pavement that is generally designed as a new PCC pavement structure assuming no bonding between the PCC overlay and the existing HMA pavement. Conventional whitetopping are generally constructed about 203 to 305 mm (8 to 12 in) thick.

Curling – Upward or downward deformation of a portland cement concrete (PCC) slab due to a difference in temperature between the top and bottom of the slab.

Discount Rate – The rate used to discount future expected costs to present day terms, often taken as the difference between the prevailing interest rate and the current inflation rate.

Equivalent Single Axle Loads (ESALs) – Summation of 80-kN (18 kip) single axle load applications used to combine mixed traffic to design traffic during the analysis period.

Falling Weight Deflectometer (FWD) – A device in which electronic sensors measure the deflection of the pavement under a load of known magnitude. The results can be used to determine the modulus of subgrade and pavement layers and the load transfer across joints and cracks.

Fiber Reinforcement – Any of several types of fibers added to a portland cement concrete (PCC) pavement to reduce shrinkage and increase post-cracking behavior. Common types of fibers that are used include polypropylene, polyolefin, and steel.

Joint Orientation – The alignment of transverse joints in a concrete pavement with respect to the centerline of the pavement. Transverse joints are usually placed at regular intervals perpendicular to the centerline of the pavement. Joints that are aligned such that the obtuse angle at the outside pavement edge lies ahead of the joint in the direction of traffic are called skewed joints.

Joint Sealant – Material used to minimize the infiltration of incompressibles and surface water into the joint. A liquid sealant can be either hot- or cold-poured. Common sealant materials include rubberized asphalt, silicone, and preformed compression seals.

Joint Sealant Reservoir – The channel sawed or formed at a joint that accommodates the joint sealant.

Joint Spacing – The distance between two consecutive transverse joints in a concrete pavement. In other words, referred to as slab length. Short-jointed pavements (JPCP) generally have joints spaced less than about 5 m (16 ft) apart while long-jointed pavements (JRCP) have longer joint spacings, typically about 9 m to 12 m.

Jointed Plain Concrete Pavements (JPCP) – A portland cement concrete (PCC) pavement system characterized by short joint spacings and containing no reinforcing steel distributed in the slab. The joints may or may not contain dowels.

Jointed Reinforced Concrete Pavements (JRCP) – A portland cement concrete (PCC) pavement system characterized by long joint spacings and containing reinforcing steel distributed throughout the slab to hold any cracks tightly together.

Leveling Course – The placement of a thin layer of hot-mix asphalt (HMA) or other bituminous material to produce a uniform surface for paving.

Load Transfer – The means through which wheel loads are transferred or transmitted across a joint from one slab to the next.

Life-Cycle Cost Analysis (LCCA) – An economic assessment of competing pavement design alternatives in which all significant costs over the life of each alternative are considered.

Milling – A process used to remove material from an HMA pavement and provide texture to promote bonding with a PCC overlay. Milling operations use drum-mounted carbide steel cutting bits to chip off the surface of the pavement.

Modulus of Elasticity (E) – A measure of the rigidity of a material and its ability to distribute loads, based on the relationship between stress and strain in the slab.

Modulus of Rupture – A measure of the extreme fiber stress developing under slab bending, the mode in which most concrete pavements are loaded. Also referred to as the concrete flexural strength.

Modulus of Subgrade Reaction (k-value) – A measure of the supporting capability of a soil based on its ability to resist penetration of a series of loaded stacked plates.

Partially Bonded Overlay – A portland cement concrete (PCC) overlay that is placed directly on an existing PCC pavement without any surface preparation. Consequently, some partial bonding between the two pavements is expected.

Preoverlay Repair – Any repair or renovation activity performed on an existing pavement prior to the placement of an overlay.

Roughness –Irregularities in the pavement surface that adversely affect ride quality, safety, and vehicle maintenance costs.

Separation Layer – A layer of hot-mix asphalt (HMA), bituminous material, or other stress-relieving material used at the interface between an unbonded PCC overlay and the existing PCC pavement to ensure independent behavior.

Shotblasting – A surface preparation process in which steel shot are propelled against the surface of a portland cement concrete (PCC) pavement, effectively cleaning and preparing the surface to receive a bonded PCC overlay.

Slab Fracturing – A technique in which an existing portland cement concrete (PCC) pavement is cracked or broken into smaller pieces to reduce the likelihood of reflection cracking.

Subbase Course – The layer or layers of specified material of designed thickness between the base and subgrade to support the base course or provide drainage.

Subgrade – The top of a roadbed upon which the pavement structure is constructed.

Subsurface Drainage – The inclusion of specific drainage elements in a pavement structure intended to remove excess surface infiltration water from a pavement.

Surface Preparation – Any of several methods used to prepare an existing pavement to receive a PCC overlay; commonly used surface preparation methods include cold milling, shotblasting, and waterblasting.

Surface Texture – The characteristics of the surface of a portland cement concrete (PCC) pavement that contribute to both surface friction and noise.

Swelling Soils – Soils, usually clays and shales, that are susceptible to significant volume increases in the presence of sufficient moisture.

Thin Whitetopping – A moderately thin PCC overlay (thicknesses between 102 and 203 mm [4 and 8 in]) that is placed on a milled HMA pavement. The bond between the PCC overlay and HMA pavement is relied upon in the design procedure, and short joint spacing (between 1.8 and 3.7 m [6 and 12 ft]) is used. .

Ultra-Thin Whitetopping – A thin (less than 100 mm [4 in]) portland cement concrete (PCC) overlay placed on an existing HMA pavement. The PCC overlay is bonded to the existing HMA pavement and employs short joint spacing (typically less than 1.8 m [6 ft]) to ensure performance.

Unbonded PCC Overlay – A portland cement concrete (PCC) overlay placed on an existing distressed PCC pavement; the overlay is separated from the existing pavement through a separator layer to ensure independent behavior.

User Costs – In a life-cycle cost analysis, costs incurred by the user, such as delay costs, vehicle operating costs, and crash costs.

Variable Joint Spacing – A series of different joint spacings repeated in a regular pattern intended to reduce the rhythmic response of vehicles traveling over uniformly spaced joints.

Warping – Upward or downward deformation of a portland cement concrete (PCC) slab due to a difference in the moisture levels of the top and bottom of the slab.

Whitetopping – A portland cement concrete (PCC) overlay placed on an existing HMA or bituminous pavement. Whitetopping may be used in referring to conventional whitetopping, thin whitetopping, or ultra-thin whitetopping.

Widened Slab – A concrete pavement slab that is paved wider (usually at least 450 mm (18 in) wider) than a conventional 3.66-m (12-ft) traffic lane. The purpose of a widened slab is to increase the distance between truck tires and the slab edge and thereby greatly reduce critical edge stresses due to loading.

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