PB91-100610

Structural Overlay Strategies for Jointed Concrete Pavements

Volume III, Performance Evaluation and Analysis of Thin Bonded Concrete Overlays

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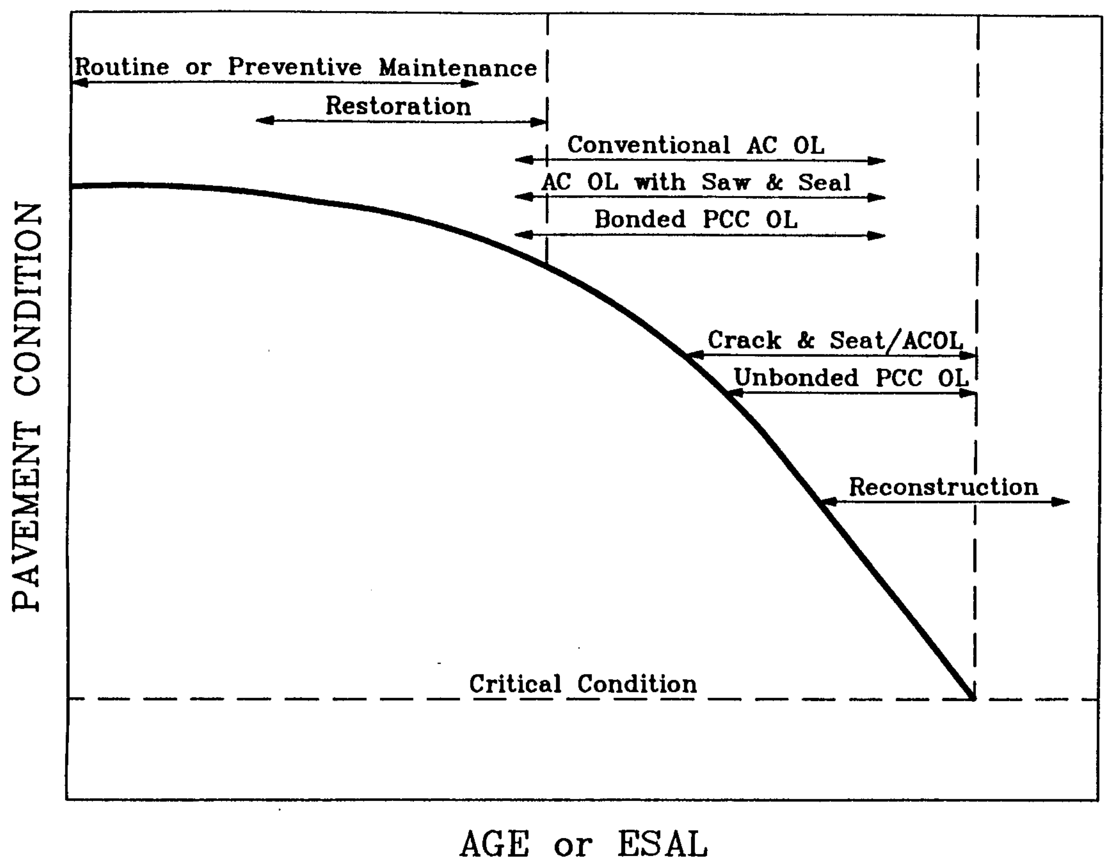

U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
Turner-Fairbank Highway Research Center
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McLean, Virginia 22101-2296

FOREWORD

This report is one volume of a four volume set of interim reports documenting a major field study and evaluation of the effectiveness of three structural overlay types for jointed portland cement concrete pavements and guidelines for their use. The three overlay types are sawing and sealing joints in 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 survey, deflection testing and roughness measurements were performed on a total of 60 sections. It should be noted that the small sample of projects and the unknown condition of the pavement prior to overlay limit the conclusions that can be drawn from the study. Volume V (Summary of Research Findings) and the technical summary will be given widespread distribution in the near future. These reports will be of interest to those involved in design, construction and rehabilitation of jointed concrete pavements.

Sufficient copies of this report are being distributed by FHWA memorandum to provide one copy to each FHWA Region and Division and two copies to each State highway agency. Direct distribution is being made to the division offices. Additional copies for the public are available from the National Technical Information Service (NTIS), U.S. Department of Commerce, 5285 Port Royal Road, Springfield, Virginia 22161. A small charge will be imposed for each copy ordered from NTIS.

Thomas J. Pasko, Jr.

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Highway Operations Research and Development

NOTICE

TECHNICAL REPORT STANDARD TITLE PAGE

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16. Abstract

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. In addition, the results of this study were used to revise and enhance the EXPEAR rehabilitation advisory system.

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 includes 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.

This volume is third in a series. The other volumes are:

FHWA No.	Vol. No.	Short Title
FHWA-RD-89-142	I	Sawing and Sealing of Joints in AC Overlays of Concrete Pavements
FHWA-RD-89-143	II	Cracking and Seating of Concrete Slabs Prior to AC Overlay
FHWA-RD-89-145	IV	Guidelines for the Selection of Rehabilitation Alternatives
FHWA-RD-89-146	V	Summary of Research Findings
FHWA-RD-89-147	VI	Appendix A - Users Manual for the EXPEAR Computer Program

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Unclassified	Unclassified	185	

Form DOT F 1700.7 (8-69)

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

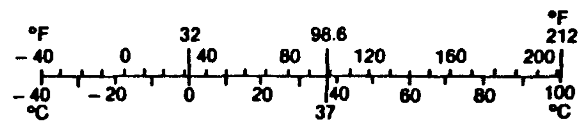
Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
	AREA			
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
	VOLUME			
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
	MASS			
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
	AREA			
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
	VOLUME			
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
	MASS			
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
	TEMPERATURE (exact)			
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F



* SI is the symbol for the International System of Measurement

(Revised April 1989)

TABLE OF CONTENTS

VOLUME I SAWING AND SEALING OF JOINTS IN AC OVERLAYS OF CONCRETE PAVEMENTS

PART I

PART II

PART A. DESIGN AND CONSTRUCTION GUIDELINES

PART B. GUIDE SPECIFICATIONS

VOLUME II CRACKING AND SEATING OF CONCRETE SLABS PRIOR TO AC OVERLAY

PART I

PART II. CRACKING, SEATING, AND OVERLAY OF PORTLAND CEMENT CONCRETE PAVEMENTS

PART A. DESIGN AND CONSTRUCTION GUIDELINES

PART B. GUIDE SPECIFICATIONS

VOLUME III PERFORMANCE EVALUATION AND ANALYSIS OF THIN BONDED CONCRETE OVERLAYS

Chapter		Page

VOLUME IV GUIDELINES FOR THE SELECTION OF REHABILITATION ALTERNATIVES

VOLUME V SUMMARY OF RESEARCH FINDINGS

VOLUME VI APPENDIX A—USERS MANUAL FOR THE EXPEAR COMPUTER PROGRAM

LIST OF FIGURES

VOLUME I SAWING AND SEALING OF JOINTS IN AC OVERLAYS OF CONCRETE PAVEMENTS

VOLUME II CRACKING AND SEATING OF CONCRETE SLABS PRIOR TO AC OVERLAY

VOLUME III PERFORMANCE EVALUATION AND ANALYSIS OF THIN BONDED CONCRETE OVERLAYS

VOLUME IV GUIDELINES FOR THE SELECTION OF REHABILITATION ALTERNATIVES

VOLUME V SUMMARY OF RESEARCH FINDINGS

LIST OF TABLES

VOLUME I SAWING AND SEALING OF JOINTS IN AC OVERLAYS OF CONCRETE PAVEMENTS

VOLUME II CRACKING AND SEATING OF CONCRETE SLABS PRIOR TO AC OVERLAY

VOLUME III PERFORMANCE EVALUATION AND ANALYSIS OF THIN BONDED CONCRETE OVERLAYS

VOLUME IV GUIDELINES FOR THE SELECTION OF REHABILITATION ALTERNATIVES

VOLUME V SUMMARY OF RESEARCH FINDINGS

VOLUME VI APPENDIX A—USERS MANUAL FOR THE EXPEAR COMPUTER PROGRAM

CHAPTER 1 INTRODUCTION

1. BACKGROUND

Portland cement concrete (PCC) pavements constitute a large percentage of those pavements that are designed to carry high volumes of heavy traffic. It is therefore not surprising that there exists a good deal of interest in the rehabilitation of PCC pavements. Pavement rehabilitation can be loosely defined "as any work performed to extend the service life of the existing pavement facility," and typically includes activities that are grouped together under the heading of "4R", or resurfacing, recycling, restoration, and reconstruction. Of the 4R activities, resurfacing (or overlays) is one of the most commonly performed methods of restoring rideability and improving structural capacity.

The most frequently constructed type of overlay is made of asphalt concrete (AC). An AC overlay can be placed fairly rapidly, at a very competitive cost, and with little shutdown time of the facility. Surface preparation may be minimal. The placement of an AC overlay will also result in dramatic initial improvement in the serviceability of the pavement. However, there are two major problems associated with AC overlays: reflection cracking and rutting. These problems contribute to a shorter service life than is desired in many cases for a rehabilitation strategy on high volume, heavily loaded pavements. Also, the commonly constructed thinner AC overlays do not provide much structural improvement; a fairly thick overlay is required to improve the structural capacity of the pavement.

An intriguing alternative to the construction of an AC overlay is the use of PCC as an overlay material. A PCC overlay holds the promise of an extended service life, increased structural capacity, and lower life cycle costs, compared to other overlay techniques. While the initial costs of a bonded PCC overlay may be much higher than those of an AC overlay, the benefit of longer life and reduced maintenance costs suggests that bonded overlays can be a viable resurfacing alternative.

2. PROBLEM STATEMENT AND RESEARCH OBJECTIVES

The research discussed in this report was performed as part of the second phase of a two phase study for the Federal Highway Administration (FHWA), entitled *Performance/Rehabilitation of Rigid Pavements*. Phase I of this project is devoted to a performance evaluation of selected PCC pavements, with the goal of improving the inputs to new pavement design.

The second phase of the project examines the rehabilitation of jointed concrete pavements. The goals of Phase II are to:

- Develop design and construction procedures for the following structural overlay techniques: crack and seat overlays, sawing and sealing joints in AC overlays over existing PCC joints, and thin bonded PCC overlays.

- Develop guidance on how to determine the most appropriate structural overlay technique(s) so that their cost-effectiveness can be compared with other strategies, such as concrete pavement restoration, construction of unbonded overlays, or reconstruction/recycling.

Separate reports are under development for each of the three strategies presented in the first goal noted above. The guidance on determination of the appropriate structural overlay techniques will also be presented in a separate document.

It is the purpose of this report to present information relevant to the design and performance of bonded concrete overlays. The specific goal of this part of Phase II is to "evaluate the performance of selected projects, including previously reviewed projects, perform additional testing and/or analysis to verify and/or improve recommended design and construction procedures, and to develop improved design and construction procedures for this technique."

3. SCOPE OF STUDY

There are three basic types of PCC overlays constructed on concrete pavements: unbonded overlays, partially bonded overlays, and bonded overlays. While they have in common the use of portland cement concrete, they differ appreciably in the warrants for their use and the appropriate construction techniques.

Bonded concrete overlays, also referred to as thin, bonded overlays (TBOL's) or thin, bonded concrete overlays (TBCO's), have been constructed in the United States for over 70 years. While there is a fairly extensive body of literature devoted to this topic (see appendix D), most of the research has consisted of individual projects or laboratory studies.

One purpose for constructing a bonded concrete overlay is to improve the structural capacity of a pavement through the construction of a thicker, monolithic cross section. This is intended to be a feasible rehabilitation alternative for pavements that require additional thickness to accommodate an increase in traffic loadings. Bonded overlays can also be constructed to correct a surficial defect that is not structural. For example, a bonded overlay will restore a worn, but structurally sound, pavement surface to a like-new condition and also improve surface friction.

Bonded overlays are currently not a widely used technique for pavement rehabilitation, except in Iowa, so there are not many candidate sections available for study. In this study, bonded overlay sections of jointed concrete pavements were evaluated in 6 States and 3 climatic zones. All of the overlay projects were constructed since 1976; the age of the original pavement varied considerably. An attempt was made to include sections on Interstates or pavements subject to heavy traffic. Most of the sections that are discussed here are also included in an earlier rehabilitation study performed for the FHWA and in other summary evaluations performed by the FHWA.[1,2]

4. RESEARCH APPROACH

For this study, an extensive performance evaluation of 16 different bonded overlay designs at 10 locations in 6 States was carried out during 1987-8. A database was formed and the pertinent elements summarized in tabular form. The general research approach can be summarized as follows:

1. Field performance surveys were conducted to determine the current condition of the pavement sections. Because of the overlap of many of the sections with the research presented in reference 1, follow-up comparisons of performance can be made.

2. Original pavement design and overlay design parameters were determined from research reports, construction specifications, and correspondence with engineers associated with the projects.

3. Historical traffic volumes were provided by the State highway agencies. Classifications and W-4 Loadometer data were provided by the FHWA.

4. Environmental data were summarized from National Oceanic and Atmospheric Administration publications and other sources of climatic information.

After all of the data were assembled, they were reduced and entered into a comprehensive PC database that was developed exclusively for this study. Extensive error-checking was performed on the data to ensure its accuracy. Summary tables were then produced, which compile the essential design, construction, and performance data in a format that is convenient for referencing. These tables, found in appendix B, allow quick performance comparisons between different designs.

Using the data from the summary tables, detailed summary reports were prepared which closely examine and analyze the performance of each project. From these reports, an evaluation of the impact of the different design factors was performed. Information gathered throughout the study was used to revise design and construction guidelines for bonded concrete overlays.

5. SEQUENCE OF REPORT

The data collection procedures followed in this study are described in chapter 2. The field performance of each pavement project is presented in chapter 3. An analysis of this data and the current literature on thin bonded overlays was used to evaluate several of the design models currently available for bonded concrete overlays, as presented in chapter 4. The performance data are also used to complete a performance evaluation of design factors, found in chapter 5. Revised design and construction guidelines for bonded concrete overlays are presented in appendix A. The summary tables in appendix B provide a more complete breakdown of the data which are available for each section. This information is supplemented by the core log found in appendix C. Appendix D consists of an annotated bibliography of reports that concern projects included in this study and other current resources on bonded concrete overlays.

CHAPTER 2 DATA COLLECTION PROCEDURES

1. INTRODUCTION

The data collection procedures presented here represent an exhaustive effort to obtain as complete and uniform a set of information for each project site as possible. This information was obtained from a number of sources and verified for completeness and accuracy. The sources include: field performance surveys; field testing; lab testing; original pavement design plans and specifications; overlay design plans and specifications; published and unpublished research reports; traffic data; environmental data; and conversations with State highway personnel. The procedures followed to obtain the distress information basically followed those presented in the *Data Collection Guide for the Long-Term Pavement Performance (LTPP) Studies*, which is sponsored by the Strategic Highway Research Program (SHRP).[3] It is anticipated that the database formed for this project will be compatible with that developed under LTPP.

2. SELECTION OF THIN BONDED OVERLAY PROJECT SECTIONS

The major effort to identify candidate sections for the evaluation of the thin bonded overlay technique was carried out under a previous study for the FHWA.[1] In that study, thin bonded overlay projects were identified and surveyed at 11 locations in Iowa, Louisiana, New York, South Dakota, and Wyoming. Of these 11 locations, 9 were selected for inclusion in this study and a tenth project in California was added.

The section selection criteria were rather limited. The controlling factor was the availability of actual sections. It was only in Iowa that additional sections were identified that were not surveyed. There were several other types of bonded overlays identified, including fiber-reinforced concrete overlays, bonded overlays of continuously reinforced concrete pavements (CRCP), and bonded CRCP overlays, that were intentionally not included. It was felt that too many dissimilarities between these and the more conventional design would be involved to allow for valid comparisons. Because of the limited availability of sections, one goal, to identify sections in the four different climatic zones, could not be met. No bonded concrete overlays were identified in the Dry Non-Freeze zone.

Another important criterion was the selection of projects for which there would be available all of the necessary data for analysis. Again, the earlier FHWA study had already accomplished much of this data collection. A very valuable additional benefit of this fact was that comparisons would be possible between the field performance of the sections surveyed in 1985-1986 and those surveyed in 1987-1988. This would provide an indication of the rate of deterioration of these sections, since the survey methods were fairly similar.

It was hoped that projects could be identified that included rigorous experimental designs. Unfortunately, only one project (IA 3) included different designs which could be used for comparison of the design features. That project did not include a factorial experimental design which would have facilitated the comparison of the design factors. Table 1 presents the pavement sections which were included in this study. Their location is shown on the map in figure 1.

3. FIELD DATA COLLECTION

Surface Distress

A comprehensive field survey was conducted at each project site. These were performed over a period from July 1987 to November 1987 (New York, Louisiana, and California) and from May 1988 to June 1988 (Iowa, Wyoming, and South Dakota). The field surveys were conducted using procedures similar to those in use for the SHRP LTPP program. Data items collected include transverse and longitudinal cracking, joint spalling, joint and crack faulting, and sealant condition. The data collection sheets used are shown in figures 2, 3, and 4. The *Distress Identification Manual for the LTPP Studies* was used as a guideline for the identification of the type, extent, and severity of the noted distresses.[4]

Debonding

A major goal in the construction of a bonded concrete overlay is the creation of a monolithic pavement cross section. Debonding is said to exist if the overlay is not completely bonded to the existing slab. Debonding may result in cracking of the overlay from loading and curling, since the overlays are quite thin. In the absence of adequate bond, the thin overlay cannot withstand the heavy traffic loadings and will rapidly fail in fatigue.

There are several ways of measuring bonding, including mechanical devices, the use of a chain drag, and "sounding." After considering several approaches, it was decided to follow a technique based on sounding of the pavement and limited coring. The survey crew consisted of two people, one to tap the pavement using a 4-lb (9 kg) hammer, and the other to record whether the sound represented a bonded or debonded layer. A debonded area was said to be present if the pavement gave off a hollow sound, or one of "low frequency." When a debonded area was located, its extent was identified by tapping with the hammer and establishing a contour of the debonded area on the pavement surface.

It should be noted that it is probably impossible to distinguish between debonding as described above and delamination or other deterioration of the concrete below the bond line. Since this distinction can not be made, it is very possible that some of the debonding identified herein is actually deteriorated original concrete.

This testing was performed at every fifth slab, starting at the transverse joint, as the joints typically experienced more debonding than the slab centers. The extent of debonding was calculated several ways: the percent of the slab area debonded, the percent of the slab corners debonded, and the percent of debonded wheelpath area. On some of the sections with widespread cracking, the extent of debonded cracks was also

Table 1. Bonded overlay sections included in study.

Project	Route	Location	Original Construction Date	Overlay Construction Date
NY 6	I-81	Syracuse, NY	1957	1981
IA 1	I-80	Grinnell, IA	1964	1984
IA 2	I-80	Avoca, IA	1966	1979
IA 3	C 17	Clayton County, IA	1968	1977
IA 4	SR 12	Sioux City, IA	1954	1978
IA 5	US 20	Waterloo, IA	1958	1976
CA 13	I-80	Truckee, CA	1964	1984
SD 1	SR 38A	Sioux Falls, SD	1950	1985
WY 1	I-25	Douglas, WY	1969	1983
LA 1	US 61	Baton Rouge, LA	1959	1981

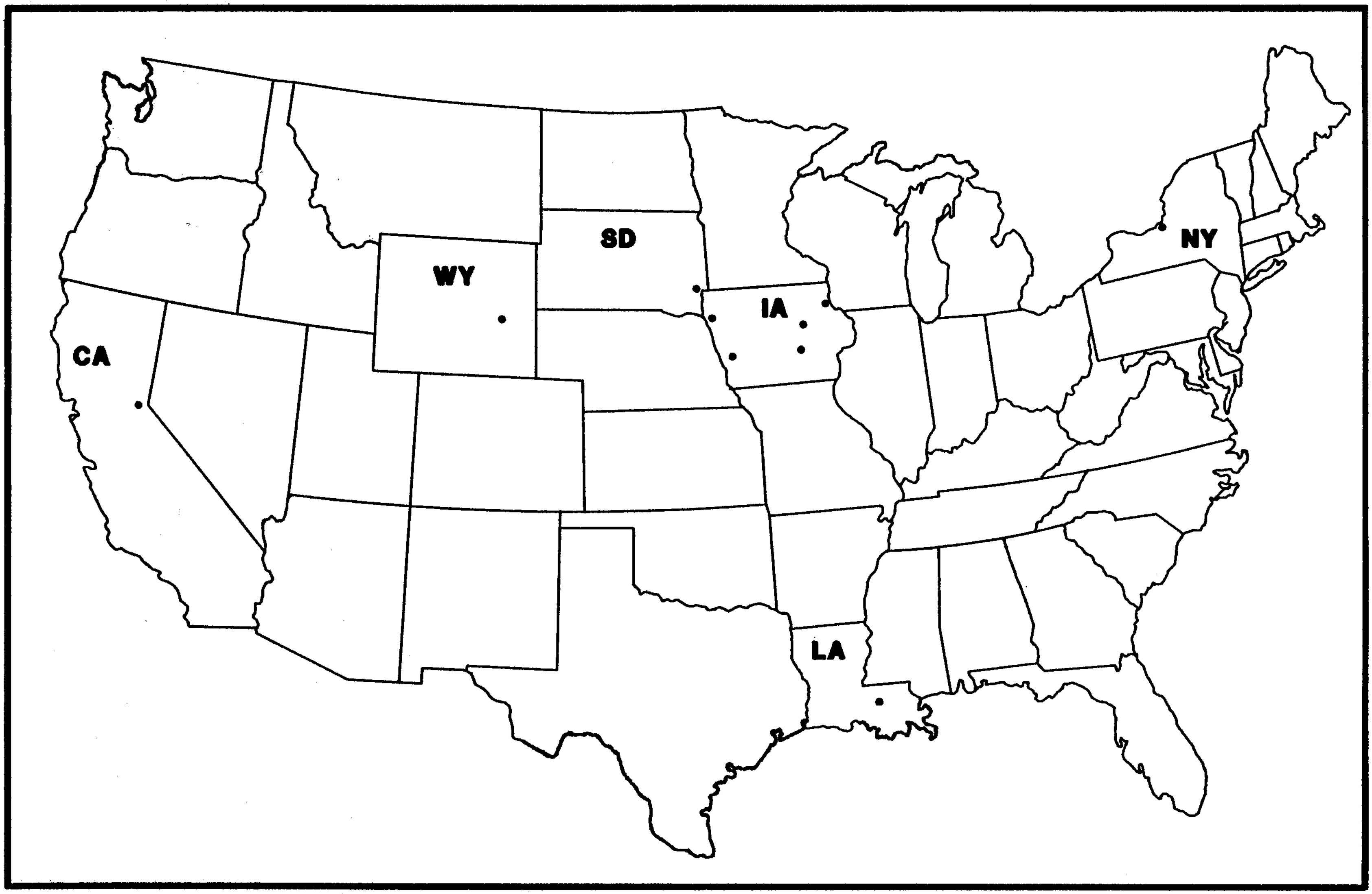


Figure 1. Location of bonded overlay projects included in study.

FIELD SURVEY: GENERAL INFORMATION

STATE CODE __ __
PROJECT ID __/__ __ __/__
ID __ __ __ __-__ __

DATE OF FIELD SURVEY (MONTH/DAY/YR) __ __/__ __/__ __
SURVEYORS' INITIALS ____/____/____

TEST SECTION LOCATION:

START POINT MILEMARK __ __ __.__ __
END POINT MILEMARK __ __ __.__ __

START POINT STATION NUMBER __ __ __ __+__ __.__ __
END POINT STATION NUMBER __ __ __ __+__ __.__ __

LENGTH OF SECTION (FEET) __ __ __

IF NO MP OR STN, DISTANCE FROM NEAREST STRUCTURE/ INTERCHANGE/CROSSROAD (FEET) __ __ __ __

TYPE/NAME OF STRUCTURE/INTERCHANGE/CROSSROAD ____________

NUMBER OF THROUGH LANES IN DIRECTION OF SURVEY ___

OUTSIDE SHOULDER WIDTH (FEET) __ __.__

INSIDE SHOULDER WIDTH (FEET) __ __.__

SHOULDER SURFACE TYPE
OUTSIDE SHOULDER ___
INSIDE SHOULDER ___

TURF...............1	CONCRETE............4
GRANULAR...........2	SURFACE TREATMENT...5
ASPHALT CONCRETE...3	OTHER__________6

AVERAGE CONTRACTION JOINT SPACING (FEET) __ __ __.__
RANDOM JOINT SPACING (IF APPLICABLE) __ __-__ __-__ __-__ __
TRANSVERSE JOINT SKEWNESS Y / N
FT/LANE ___

ROUGHNESS AND SERVICEABILITY:

		LANE NUMBER*		
		1	2	3
ROUGHNESS INDEX	(TRIAL 1)	__ __ __	__ __ __	__ __ __
	(TRIAL 2)	__ __ __	__ __ __	__ __ __
ROUGHNESS MEASUREMENT SPEED (MPH)		__ __	__ __	__ __
PRESENT SERVICEABILITY RATING (MEAN)		__.__	__.__	__.__

*LANE 1 IS OUTER LANE, LANE 2 IS NEXT TO LANE 1, ETC.

Figure 2. General field survey sheet.

FIELD SURVEY: DRAINAGE INFORMATION

STATE CODE __ __
PROJECT ID __/__ __ __/__
ID __ __ __ __-__ __

	STATION	SLOPE INNER	SLOPE OUTER
LONGITUDINAL SLOPE (NEAREST 1/16")	__ __ __ __+__ __	__/__	__/__
(3 MEASUREMENTS, EQUALLY	__ __ __ __+__ __	__/__	__/__
SPACED ALONG PROJECT)	__ __ __ __+__ __	__/__	__/__
TRANSVERSE SLOPE (NEAREST 1/16")	__ __ __ __+__ __	__/__	__/__
(3 MEASUREMENTS, EQUALLY	__ __ __ __+__ __	__/__	__/__
SPACED ALONG PROJECT)	__ __ __ __+__ __	__/__	__/__
SHOULDER SLOPE (NEAREST 1/16")	__ __ __ __+__ __	__/__	__/__
(3 MEASUREMENTS, EQUALLY	__ __ __ __+__ __	__/__	__/__
SPACED ALONG PROJECT)	__ __ __ __+__ __	__/__	__/__

CUT OR FILL DEPTH (GROUND LEVEL TO PAVEMENT SURFACE ELEVATION) ____

FILL > 40 FT......................1
FILL 16 - 40 FT...................2
FILL 6 - 15 FT....................3
AT GRADE (5 FT FILL TO 5 FT CUT)...4
CUT 6 - 15 FT.....................5
CUT 16 - 40 FT....................6
CUT > 40 FT.......................7

DEPTH OF DITCH LINE (FROM PAVEMENT SURFACE, FEET) __ __

LANE/SHOULDER JOINT INTEGRITY:

	OUTER SHOULDER	INNER SHOULDER
SEALANT DAMAGE	N L M H	N L M H
BLOWHOLES	N L M H	N L M H

TYPE OF SUBSURFACE DRAINAGE SYSTEM PRESENT (VISUAL) ____

NONE........................1
LONGITUDINAL DRAINS..........2
TRANSVERSE DRAINS............3
OTHER________________________7

INDICATORS OF POOR DRAINAGE:

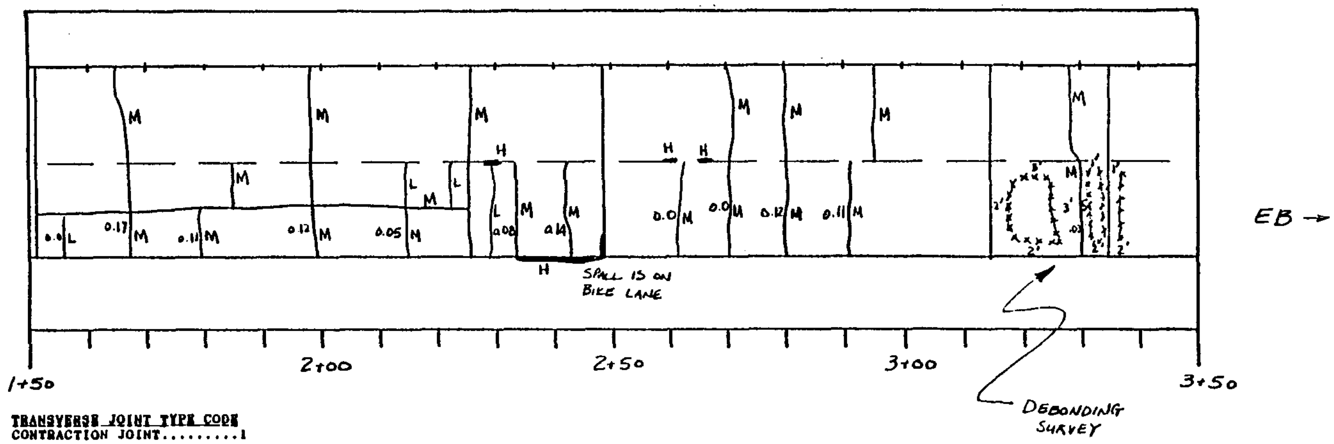
CATTAILS OR WILLOWS GROWING IN DITCH	Y / N
DRAINAGE OUTLETS CLOGGED	Y / N
DRAINAGE OUTLETS BELOW DITCHLINE	Y / N
NON-CONTINUOUS CROSS SECTION, CROWN TO DRAINAGE DITCH	Y / N
PUMPING	L M H
OTHER______________________________	Y / N

Figure 3. Drainage field survey sheet.

TO BE SKETCHED	NOTE ON SKETCH	
ALL CRACKING	LIN FT	L M H
LONGITUDINAL JOINT SPALLING		L M H
LONGITUDINAL FAULTING	INCHES	
CRACK FAULTING	INCHES	
SCALING/MAP CRACKING	AREA	L M H
PATCHES/REPLACED SLABS	DIMENSIONS	
IMPROPER JOINT CONSTRUCTION CRACKS	LIN FT	
MISSAWED JOINTS (BONDED PCOL)	LIN FT	
BLOWUPS	BU	L M H
"D" CRACKING	D	L M H
REACTIVE AGGREGATE	RA	L M H
ALL SHOULDER DISTRESSES		

STATE
PROJECT ID _/_ _ _/_
ID _ 1 4 3 - 1 _
PAGE # _ 2 OF 1 9

FIELD DATA COLLECTION FORM



TRANSVERSE JOINT TYPE CODE
CONTRACTION JOINT.........1
CONSTRUCTION JOINT........2
PATCH APPROACH JOINT......3
PATCH LEAVE JOINT.........4
PRESSURE RELIEF JOINT.....5

	EB OUTER LANE						WB INNER LANE					
	STATION 1+52	STATION 2+48	STATION 3+14	STATION 3+34	STATION	STATION	STATION 1+52	STATION 2+48	STATION 3+14	STATION 3+34	STATION	STATION
TRANSVERSE JOINT TYPE (USE CODE)	1	1	3	1			1	1	3	1		
SPALLING	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
JOINT SEAL DAMAGE	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
LONGITUDINAL JOINT SEAL DAMAGE	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
PATCH/SLAB REPLACEMENT DETERIORATION	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
SLAB DETERIORATION ADJACENT TO PATCH	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
LANE TO SHOULDER DROPOFF (INCHES)	0.27											
LANE/SHOULDER SEPARATION (INCHES)	2.50											
FAULTING (INCHES)	0.10	0.04	0.07	0.01								
JOINT WIDTH (INCHES, OUTER LANE ONLY)	0.30											

Figure 4. Field data collection form.

noted. It must be stressed that the debonding recorded for these sections is the result of limited testing using a partially subjective technique. In order to accurately characterize the extent of debonding it would be necessary to survey a larger area of the project with a laboratory-tested method.

Roughness and PSR

Pavement surface roughness was collected for all of the sections with the aid of a Mays Ride Meter installed on the rear axle of a 1985 Buick Le Sabre. This test was performed a minimum of two times per section, at a constant speed of 50 mi/h (80 km/h). For several sections it was not possible to perform the test, due either to lane closures, a steep longitudinal grade, or large volumes of slow moving traffic.

During the first pass over the section, the survey crew also gave a subjective rating of the rideability of the pavement section, which is presented as an average Present Serviceability Rating (PSR).

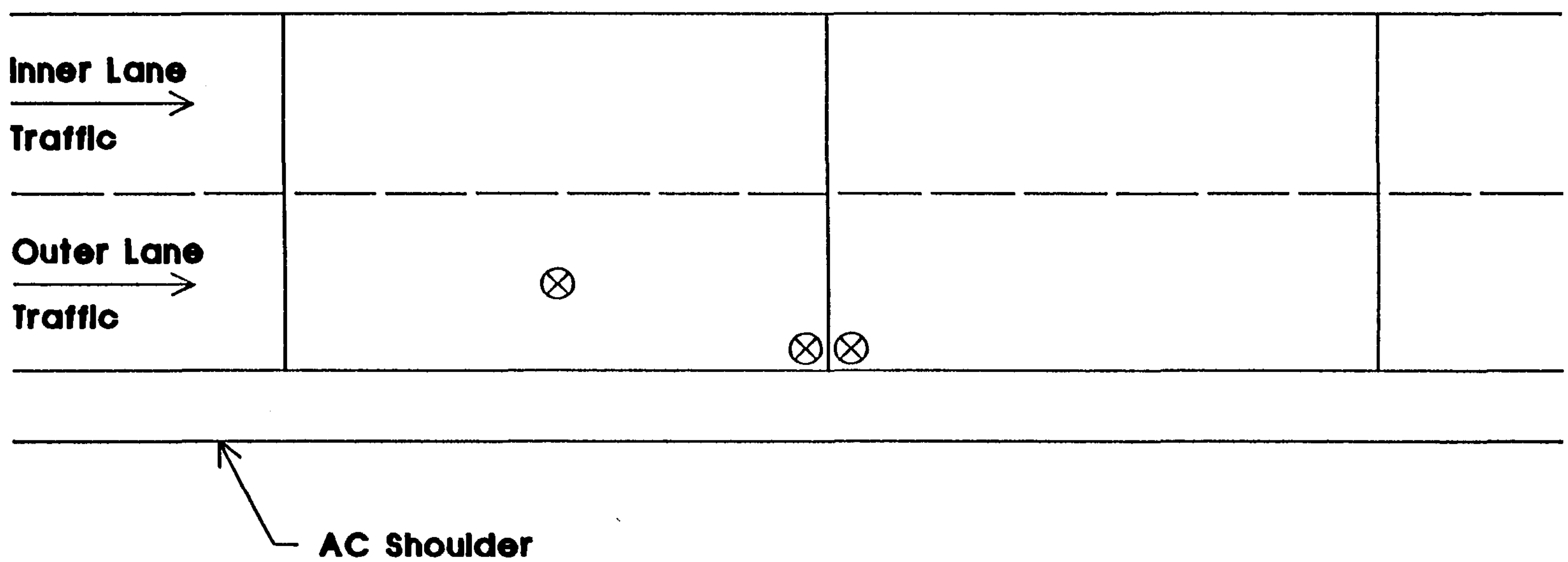
Deflection Testing

Deflection testing was performed with the use of a Dynatest Model 8002 Falling Weight Deflectometer (FWD). The testing pattern used on each section is shown in figure 5.

At all but one section, the FWD testing was performed while the slab temperature was below 80 °F (27 °C). Testing at lower temperatures helps to minimize the effect of slab expansion on the determination of load transfer and void detection. The deflections obtained at the slab centers were used to backcalculate the modulus of elasticity of the monolithic slab, and the dynamic modulus of subgrade reaction on top of the base.[5,6] Load transfer at the transverse joints was obtained by calculating the ratio of the deflection of the unloaded side to the deflection of the loaded side. Joint corner deflections were used to detect the presence of voids under the slab corners, using procedures identified in NCHRP 1-21.[7] These values are reported for each section, but it is not believed that the procedures are applicable for the detection of voids in the case where a complete bond does not exist between the overlay and the existing slab.

Material Testing

Coring and boring were performed at each project location. The retrieved cores were subjected to a visual inspection and a verification of thickness. The bond between the overlay and the existing pavement was tested by applying a shearing force on the monolithic core using a specially-constructed apparatus and a compression testing device. The core from the original pavement, when separated from the overlay, was subjected to a split tensile test in accordance with AASHTO T-98.



Required: 10 slab centers and 20 corners spread out over 1000' test sections (or length of test section, if shorter).

Figure 5. Layout for FWD testing for thin bonded overlays.

Subsurface material was retrieved from beneath the concrete. Gradation curves and moisture contents were calculated for all granular materials. These data aided in the identification of the granular material, and in the classification of the subgrade.

Traffic

The calculation of historical traffic loadings is a very important part of the performance evaluation, as this data is used to establish the relationship between loading and observed distress. The Average Daily Traffic (ADT) and percent of that traffic that was heavy trucks was provided by the State highway agencies for each year that the pavement was in service or for each available year. Data compiled by the States and submitted to FHWA on truck types and axle load distributions (W-4 tables) were used to calculate truck factors by State and to calculate the number of 18-kip (80 kN) Equivalent Single Axle Loads (ESAL's) applied to each section. This information was used to estimate the number of ESAL's which were applied prior to the overlay and the number of ESAL's that had been applied since placement of the overlay.

The general form of the equation that was used to calculate ESAL's is shown below. This equation provides an estimate of the number of ESAL's sustained by lane *i* of the pavement in year *j*. To obtain the cumulative ESAL applications for lane *i*, this calculation is performed for every year that the pavement is in service and those totals are summed. There are at least two problems with this approach. The truck factors used in the calculation are those obtained from W-4 data. The truck factors calculated from W-4 tables have been shown to underrepresent the actual truck factors of today's highway vehicles.[8] Also, the use of an average truck factor for all trucks instead of specific truck factors for each truck type is an approximation. For this study, no attempt was made to account for these discrepancies, which most likely will result in an underestimation of the cumulative ESAL's.

$$ESAL_{ij} = ADT_j * TKS_j * DD_j * LD_{ij} * TF_j * 365 \qquad (1)$$

Where:

$ESAL_{ij}$ = Number of 18-kip (80 kN) ESAL applications sustained by lane *i* in year *j*,
ADT_j = Two-way Average Daily Traffic for year *j*,
TKS_j = Percent heavy trucks for year *j*,
DD_j = Directional Distribution of traffic for year *j*,
LD_{ij} = Lane Distribution of trucks in lane *i* for year *j*, and
TF_j = Average Truck Factor for year *j*.

Environment

Environmental data were collected in order to describe the nature of the environmental forces to which the pavement is subjected. This information varies significantly from project to project. The monthly temperatures and precipitation were obtained from the data summarized by the National Oceanic and Atmospheric Administration.[9] Information on the other climatic indices, such as the Thornthwaite Moisture Index, the number of freeze-thaw cycles, and the Freezing Index, were obtained from other sources.[3,10,11]

Photographic Survey

A 35 mm photographic survey of each section was made at the same time as the distress survey was performed. This photo survey helps to visualize the pavement's condition when examined in conjunction with the distress surveys. It also is very useful in resolving anomalies uncovered in the field.

CHAPTER 3 FIELD PERFORMANCE AND EVALUATION

1. INTRODUCTION

This section presents summary information about each of the thin bonded overlay sections surveyed for this project. Location and environmental data are briefly summarized. There is also a brief description of the original pavement design and the condition of the pavement prior to the construction of the overlay. Various pertinent features regarding the construction of the overlay are discussed, including preparation of the pavement, the type of adhesive and concrete used, and joint construction features.

Where available, previously published observations of the performance of these sections are summarized. Most of the sections were surveyed during 1985-6 as part of a rehabilitation study performed for the FHWA.[1] These sections are identified and the results of those surveys are provided. Some of these sections were also reviewed by the FHWA as part of their Experimental Projects Program and, where applicable, comments from a report prepared for that program are also summarized.[2] It must be noted that the extent of overlap between the exact locations in those previous surveys and the surveys conducted for this study is not known. Therefore, comparisons between these data cannot be rigorously pursued. Special attention should be paid to the tabulation of the longitudinal cracking; this survey counted the longitudinal crack that formed when a centerline joint was not sawed in the overlay as a longitudinal crack. This is not the case in at least one of the aforementioned studies.

The most detailed performance results were obtained during the field surveys conducted for this project. The data obtained from the lab testing and distress and debonding surveys are discussed. Preliminary conclusions regarding the performance of each project are also made.

2. INTERSTATE 81—SYRACUSE, NEW YORK (NY 6)

This section is located on I-81, near Syracuse, New York. It is in the wet-freeze environmental zone, with an average of 39 in (991 mm) of precipitation annually and a Freezing Index of 990. The original pavement was constructed in 1957 as a 9-in (229 mm) dowelled, jointed reinforced concrete pavement (JRCP) with 43-ft (13.1 m) joint spacing, placed on a 12-in (305 mm) aggregate base.

Preoverlay Pavement Condition

The original JRCP pavement displayed extensive longitudinal and transverse crack deterioration. This was most likely a result of the use of coarse aggregate susceptible to freeze-thaw deterioration in the original concrete. Freeze-thaw cycling caused pop-outs in the pavement surface and disintegration beneath the surface similar to D-cracking. The areas that showed the most deterioration were those surface areas exposed to water and areas where water could be held and trapped, such as the pavement edges and joint faces. Many of the deteriorated areas had been repaired with asphalt patches.

Pavement blowups had occurred during the life of this pavement. In 1972, dowelled, full depth repairs were placed at as many transverse joints as funding permitted. In 1980, many of these repairs were showing deterioration also. Slab cracking was also present on this section, particularly over existing culverts. Before placement of the bonded overlay in 1981, it is estimated that the outer lane of this pavement had sustained 3,350,000 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications, the middle lane 1,260,000 ESAL applications, and the inner lane 350,000 ESAL applications.

Overlay

In 1981, a 3-in (76 mm) bonded PCC overlay was placed on the existing pavement after extensive surface preparation. The deteriorated slab at the joints was milled to a depth of 3 in (76 mm). Almost all of the transverse joints required treatment in this manner. The milling was generally extended about 2 ft (0.6 m) on either side of the joint. About 90 percent of the length of the longitudinal joints required the same milling. These depressed areas were paved over at the same time as the overlay was placed. Pressure relief joints were placed at blowup locations and at either end of northbound and southbound mainline structures over NY 31. Wire mesh was placed over areas of existing cracking where it was felt that the existing mesh was no longer functioning. The surface of the existing pavement was milled to a depth between 0.25 and 0.50 in (6 and 13 mm). The pavement was then sandblasted to remove any remaining loose material or contaminants.

Adhesive

A cement-sand grout was used which consisted of a mix of one part cement to one part sand, with water added to achieve the desired fluidity. The grout was spread by hand and broomed onto the pavement. It was placed shortly ahead of the paver.

Concrete

The concrete used was a modified New York State Department of Transportation (NYSDOT) Class D mix. The coarse aggregate had a maximum size of 1.0 in (25 mm), with most stone being a nominal 0.375 in (9.5 mm) size. The concrete mix had a 2.5 in (64 mm) slump and an average entrained air content of 7 percent. Temperatures at the time of placement ranged from about 50 °F to near 90 °F (10 °C to 32 °C).

Curing

The pavement was textured by transverse tining and immediately after texturing the curing compound was applied. It consisted of a white pigmented compound applied at a rate of 0.12 gallon/yd^2 (0.38 l/m^2).

Joints

Immediately after paving, the plastic concrete was scored with a straight edge and an edging tool directly over the transverse joints, using previously marked guide locations. Within 5 to 6 hours after placement of the concrete overlay, the transverse joints were sawed to a depth of 5 in (127 mm), rather than 3 in (76 mm). This compensated for the additional thickness of the overlay at the transverse joints from the additional milling performed there. The sealant reservoir was sawcut later. The longitudinal joint between lanes was sawed to a depth of 2 in (51 mm).

Early Performance Observations

This project was the subject of a report by the NYSDOT in 1982.[12] That report notes the development of narrow transverse cracks during construction of the overlay. These cracks occurred at intervals of as little as 1.5 ft (0.46 m), and averaged about 5 per slab. They were probably shrinkage cracks which formed during curing. Two blowups were noted in June 1984. These were accompanied by areas of debonding adjacent to the blowup. However, examination of the pieces of pavement broken out in the blowup showed that a good bond still existed between the original pavement and the overlay. Performance observations from 1985 showed that while the overlay exhibited shrinkage cracking, it had not delaminated. The longitudinal joints showed some minor spalling. The transverse joints were in good condition with the exception of two joints, which exhibited delamination. Some spalling occurred at the intersection of the transverse and longitudinal joints. The transverse cracking had reflected through the overlay, but had not appeared to cause delamination. The AC pressure relief joints had been heated and shaved at least once in an attempt to reduce the bumps.

These results from NYSDOT in 1985 are consistent with those in the earlier FHWA report, based on a 1985 survey. The researchers also observed shrinkage cracking, very little deteriorated transverse cracking, and no joint related distresses. The pressure relief joints were shoving and contributing to opening of the contraction joints.

Physical Testing

The PCC overlay appeared to be completely bonded to the existing slab. The existing concrete was subjected to a split tensile test and the corresponding third point modulus of rupture was estimated to be 776 psi (5.35 MPa). A core was also retrieved from a slab corner. Good bond existed between the overlay and the original slab, but the core was not recovered in one piece, having disintegrated from the level of the original slab reinforcement and below.

The average deflection at the mid-slab location was 3.8 mils (0.10 mm). The loaded corner deflections were quite high, averaging 18.8 mils (0.48 mm). The load transfer efficiency was 29 percent. This may in part be explained by the deterioration occurring in the original slab at each joint and the slab movement afforded by the pressure relief joints.

Deterioration of the Pavement Section

Since construction of the overlay in 1981, this pavement has experienced 2,360,000 ESAL's in the outer lane. This is over 70 percent of the total estimated traffic carried on the original pavement from 1957-1981. As an examination of table 2 shows, there is some cracking and faulting present and it has increased from 1985 to 1987. There is more transverse cracking in the second lane. The ride is fairly rough (PSR = 3.2), indicating that some problems are developing. The results of the bonding survey are shown in table 3. Almost all of the corners tested showed debonding, but none was detected in the wheelpaths. The average size of the debonded area at each corner was about 2 ft^2 (0.19 m^2).

Summary and Conclusions

The reports indicate that the original pavement exhibited extensive distresses, especially at the joints. There were a large number of deteriorated longitudinal and transverse cracks. Pressure relief joints had been constructed in response to a blow-up problem, but the pavement had continued to experience blow-ups. Many full-depth repairs had been constructed, starting about 15 years after the original construction and continuing until the overlay was built. The overall performance of this section, surveyed 6 years after construction, is not very good. It is very likely that there was too much deterioration present on the original pavement to warrant the construction of a bonded overlay.

3. INTERSTATE 80—GRINNELL, IOWA (IA 1)

This project on I-80 is located in central Iowa, near Grinnell. All of the sections in Iowa are in the wet-freeze environmental zone. There is an average annual precipitation of 35 in (889 mm), and a Freezing Index of 625. The original pavement was constructed in 1964 as a 10-in (225 mm) dowelled JRCP on a 4-in (102 mm) aggregate base. The transverse joint spacing was 76.5 ft (23.3 m).

Preoverlay Pavement Condition

At the time of placement of the overlay, the original pavement exhibited extensive distress. There were 110 broken interior corners noted in the plans. Other preoverlay condition data were not available.

Overlay

In 1984, a 4-in (102 mm) thick, bonded concrete overlay was constructed on this pavement. Prior to construction of the overlay, however, extensive full depth repairs were placed; 458 areas were noted as being already in place and the construction of an additional 260 patches was required, again according to the plans. Epoxy-coated tie bars were placed on chairs above full-depth concrete repair joints that did not constitute a pavement joint prior to placement of the overlay. Longitudinal subdrains with transverse outlets were added to the project. At areas of broken interior corners, a depressed area of 4 in (102 mm) was to be created by milling.

Table 2. NY 6 performance summary.

	1985 FHWA SURVEY	FIELD SURVEY 1987		
	OUTER LANE	OUTER LANE	LANE #2	LANE #3
Average PSR	N/A	3.2	N/A	N/A
Mays Roughness, IN/MI	N/A	135	N/A	N/A
Transverse Faulting, IN	0.05	0.07	N/A	0.11
Transverse Cracks/MI L	231	152	300	76
M	5	20	5	0
H	0	0	0	0
Long. Crk., LIN FT/MI L	0	20	0	0
M	0	0	0	0
H	0	0	0	0
% Joints Spalled	0	0	0	0
ESAL's on Overlay (millions)	1.10	2.36	1.01	0.26

Table 3. NY 6 bonding survey summary.

% DEBONDED JOINT CORNERS	95.0
% DEBONDED AREA OF WHEELPATH	0
% TOTAL AREA DEBONDED	3.0

The initial surface preparation consisted of shotblasting, followed by air blasting. Prior to construction of the overlay, it is estimated that the outer lane had sustained 11,800,000 ESAL's and the inner lane 2,230,000 ESAL's.

Adhesive

A cement-water bonding grout was sprayed on the cleaned surface just prior to the application of the overlay.

Concrete

The concrete used in the overlay was a standard Iowa type "C" mix.

Curing

No information was available on the procedure followed for curing the concrete surfacing.

Joints

The transverse joints were sawed the full depth of the overlay and sealed with joint sealant material and backer rope. Joints at full depth patches were not sawed. The longitudinal joint was sawed to a depth of 1.5 in (38 mm).

Early Performance Observations

This project was surveyed in 1985 under the FHWA study. At the time, it had been open to traffic for about 1 year. Several full-depth repairs were noted and where these did not extend across both lanes a crack had propagated to the adjacent lane. This had also occurred at mismatched joints. These results are summarized in table 4.

Physical Testing

Cores were retrieved from the pavement at a center slab and corner location. The center slab core was in excellent condition, with no distresses noted. The core from the corner, however, was not recovered intact. Extensive horizontal cracking passed through the aggregate and the mortar, starting about 3 in (76 mm) below the surface of the original slab. Several inches of the bottom of the core were disintegrated. There may be some microcracking in the large aggregate of the overlay also. The bond shear strength between the concrete layers, as measured at the corner, was 714 psi (4.92 MPa). The split tensile test was used to estimate a modulus of rupture of the existing concrete of 667 psi (4.60 MPa). Testing with the FWD showed an average mid-slab deflection of 2.4 mils (0.06 mm) and transverse joint load transfer efficiency of 85 percent. The average loaded corner deflection was 7.6 mils (0.19 mm).

Table 4. IA 1 performance summary.

	1985 FHWA SURVEY	FIELD SURVEY 1988	
	OUTER LANE	OUTER LANE	LANE #2
Average PSR	N/A	4.2	4.2
Mays Roughness, IN/MI	N/A	69	44
Transverse Faulting, IN	0.05	0.02	N/A
Transverse Cracks/MI L	222	210	225
M	11	0*	0
H	0	0	0
Long. Crk., LIN FT/MI L	21	0	5
M	0	0	0
H	0	0	0
% Joints Spalled	0	4.8	6.7
ESAL's on Overlay (millions)	1.87	6.31	1.41

* Several Cracks were sealed and counted as low severity.

Table 5. IA 1 bonding survey summary.

% DEBONDED JOINT CORNERS	22.2
% DEBONDED CRACK CORNERS	27.1
% DEBONDED AREA OF WHEELPATH	0
% TOTAL AREA DEBONDED	5.8

Deterioration of the Pavement Section

The results of the field survey from this section are found in table 4. These show that after 4 years of service and 6,310,000 ESAL's, the pavement was still performing satisfactorily. The amount of traffic carried by the overlay in 4 years was over 50 percent of the total traffic carried by the original pavement in 20 years. The bonding survey results are summarized in table 5. Debonding was occurring at both joints and cracks, although none was noted in the wheelpaths. Crack sealing had changed the rating of some transverse cracks noted in 1985 as medium to low severity in 1988.

Summary and Conclusions

This pavement exhibited extensive distress prior to construction of the overlay. After three years of heavy traffic, the only distresses noted were transverse cracking and some joint spalling. A comparison between the 1985 and 1988 surveys shows no indication of progressive deterioration, however. The amount of debonding suggests that further deterioration of this section may occur.

4. INTERSTATE 80—AVOCA, IOWA (IA 2)

This project is located in west central Iowa, on I-80 near Avoca. It experiences an average of 32 in (813 mm) of precipitation annually and has a Freezing Index of 688. The original pavement consists of a short section of 10 in (254 mm) dowelled JRCP on a 4 in (102 mm) aggregate subbase, with 76.5 ft (23.3 m) transverse joint spacing. It was constructed in 1966. There is a longer section, consisting of 8 in (203 mm) of CRCP, that was overlaid at the same time that was not surveyed as part of this project. Prior to construction of the overlay, it is estimated that the pavement had sustained 5,410,000 ESAL's in the outer lane and 810,000 in the inner lane.

Preoverlay Pavement Condition

The original pavement exhibited D-cracking at the transverse and longitudinal joints prior to construction of the overlay. The severity and extent of this distress were not noted, but probably consisted of low to medium severity D-cracking, occurring at most of the joints.

Overlay

In 1979 a 3-in (76 mm) bonded overlay was constructed on this pavement. Approximately 400 yd^2 (330 m^2) of partial depth repairs and 153 yd^2 (130 m^2) of full-depth repairs were specified for severely deteriorated areas, prior to placement of the overlay. The continuity of the steel was not maintained in these patches. Pressure relief joints were constructed on an average of every 800 ft (240 m). These were sawed in the overlay approximately 4 in (102 mm) wide within 48 hours and sealed with a preformed urethane foam.

The pavement was milled to a depth of 0.25 in (6 mm) prior to resurfacing. In areas where there was deteriorated D-cracked pavement the milling extended to a depth of 1 in (25 mm). Final surface preparation consisted of sandblasting and airblasting.

Before placement of the overlay, longitudinal edge drains were installed. These consisted of 4-in (102 mm) diameter, slotted polyethylene pipes, placed in a 10-in (254 mm) wide and 30-in (762 mm) deep trench located at the edge of the pavement. A porous backfill was used. Transverse outlets were placed at 1000-ft (305 m) intervals.

Adhesive

The grout used to bond the overlay consisted of a cement and water mixture.

Concrete

The concrete mix was Iowa's conventional mix with a water reducing agent added (C-4WR).

Curing

The standard curing compound was applied at a rate of one and a half times the normal rate, in two applications.

Joints

No longitudinal joint was sawed on this project. The transverse joint, however, was sawed the full depth of the overlay.

Early Performance Observations

In the FHWA's 1986 report on bonded overlays, areas of instability related to distresses in the existing pavement were identified. Near the centerline of the pavement, a longitudinal crack had formed. No debonding was noted, even in areas adjacent to the pressure relief joints, although these had continued to close and were showing some faulting. Iowa obtained PSI data in 1979, 1980, 1983, and 1986. Cracking and patching surveys have been conducted annually since construction. The Delamtect and infrared techniques were used to detect delaminations in 1980 and 1982.

This project was surveyed in 1985. The distresses noted during that survey are shown in table 6.

Physical Testing

The core retrieved from the slab center was indicative of a well-bonded overlay in good condition. No distresses were noted. The slab corner core retrieved was also

Table 6. IA 2 performance summary.

	1985 FHWA SURVEY	FIELD SURVEY 1988	
	OUTER LANE	OUTER LANE	LANE #2
Average PSR	N/A	N/A	3.7
Mays Roughness, IN/MI	N/A	N/A	173
Transverse Faulting, IN	0.06	0.10	N/A
Transverse Cracks/MI L	164	211	162
M	26	6 *	0
H	0	0	0
Long. Crk., LIN FT/MI L	53	0	0
M	0	5280 **	0
H	0	0	0
% Joints Spalled	0	25.0	23.1
ESAL's on Overlay (millions)	4.82	7.93	1.39

* Centerline joint not sawed.
** Sealing may have changed M-sev. cracks to L-sev.

Table 7. IA 2 bonding survey summary.

% DEBONDED JOINT CORNERS	30.0
% DEBONDED CRACK CORNERS	11.4
% DEBONDED AREA OF WHEELPATH	0
% TOTAL AREA DEBONDED	1.7

showed good bonding. However, the original slab was totally deteriorated, with extensive cracking and disintegration. The slab appeared to have had a bituminous subsealing, as asphaltic material had infiltrated cracks at the bottom of the slab. The shear test performed on the center slab core showed a bond of 756 psi (5.21 MPa). It was not possible to perform a shear test on the deteriorated corner core. Split tensile testing was performed to estimate a modulus of rupture of the original concrete of 854 psi (5.89 MPa). Deflection testing with the FWD showed a load transfer efficiency of 64 percent and an average mid-slab deflection of 3.3 mils (0.08 mm).

Deterioration of the Pavement Section

The 1988 survey results are found in table 6. These show some continued deterioration, with 0.1 in (2.5 mm) of faulting and deteriorated transverse cracks in the outer lane. There was also a large amount of transverse joint spalling present. Results from the bonding test are found in table 7. Debonding was present at both joint and crack corners, although it was more prevalent at joint corners. At the time of the survey, the overlay had sustained approximately 7.9 million ESAL's in the outer lane and 1.4 million ESAL's in the inner lane, which is approximately 50 percent more than the original pavement had carried.

Summary and Conclusions

This bonded overlay has performed very well and is not showing significant deterioration of the surface. It was placed over a pavement which probably showed distress at every transverse joint. While the overlay does not exhibit D-cracking at the joints, there were a large number of spalled joints and transverse cracks. Deteriorated cracks and transverse joints are showing signs of debonding.

5. COUNTY ROUTE C-17—CLAYTON COUNTY, IOWA (IA 3)

This experimental project is located in east central Iowa, near the Mississippi River. This area receives an average annual precipitation of 32 in (813 mm) and has a Freezing Index of 875. The original pavement was constructed in 1968 as a two-lane, 22-ft (6.7 m) wide and 6-in (152 mm) thick JPCP, with 40 ft (12.2 m) joint spacing. This pavement was constructed on a granular surfaced secondary roadway that had been shaped to the required cross-section. Approximately 0.6 miles (1 km) of this project is on an 11 percent grade. The existing pavement had experienced 350,000 ESAL's in the eastbound direction and 150,000 ESAL's in the westbound direction prior to placement of the overlay.

Preoverlay Pavement Condition

There was a lot of cracking and faulting of up to 0.5 in (13 mm) on some joints. This is referred to as "an extensive amount of slab cracking" in the earlier FHWA report. The 40-ft (12.2 m) joint spacing without reinforcing mesh no doubt contributed to the cracking. No further information was available concerning the existing condition of the pavement.

Experimental Variables

A total of seven different sections, designated IA 3-1 through 3-7, were evaluated. These included variations in the following: overlay thickness; surface preparation; concrete water reducing admixtures; reinforcement; and sawing of joints. These sections were constructed over a length of 0.9 miles (1.5 km). The existing full-depth repairs were also removed. Partial depth repairs were made on 27 transverse joints, of which 13 joints were repaired full-width and 14 joints were repaired one-half width. Several different surface preparation techniques were employed. One procedure removed the top 0.25 in (6 mm) of the existing concrete pavement by milling. This was followed by vacuuming or air blasting to ensure a clean slab surface. Joints which were faulted over 0.25 in (6 mm) were also milled and trimmed to match the existing slab surface. An alternate technique was to sandblast the slab surface, followed by air blasting. A third technique, water blasting, was used in this project, although no sections with water blasting were included in this evaluation. The overlay thicknesses ranged from 2 in to 5 in (51 to 127 mm).

Adhesive

The grout used was a mix of cement and sand, with the exception of the last 475 ft (145 m) of the project, in which a cement-water grout was used. The grout was hauled from a central mixing plant and was spread using brooms and squeegees. Some of the grout stayed in ready-mix trucks for up to 5 hours before placement. It was found that bond strengths decreased when using grout which had been in the truck longer than 3 hours.

Research reported herein considered the variations in surface preparation, reinforcement, and thickness. These sections are shown in table 8.

Reinforcement

There were two different types of reinforcement used. Number 4 reinforcing bars were placed on 30-in (762 mm) centers in 3-, 4-, and 5-in (76, 102, and 127 mm) thick sections. A section was constructed with chain link fence reinforcement, but this proved unsuccessful and was abandoned after 50 ft (15 m) of paving.

Concrete

The concrete used was a conventional C-4 mix and a modified C-4 mix, with super water reducers added. Two different water reducing agents were evaluated.

Curing

The curing compound applied to the entire project was a white pigmented liquid membrane curing compound. It was applied at the rate of 0.13 gallon/yd^2

Table 8. IA 3 (C-17) experimental variables.

PCC MIX	OVERLAY THICKNESS, IN	SURFACE PREP	OVERLAY REINFORCEMENT	JOINT SAWING	PROJECT ID
C-4 SWR	3	SAND BL.[1]	CHAIN LINK	NONE	NONE
C-4 SWR	3	SAND BL.	# 4 REINF	TR. JTS.	3-2
C-4 SWR	3	SAND BL.	NONE	NONE	NONE
C-4 SWR	3	SAND BL.	NONE	TR. JTS.	3-1
C-4 SWR	3	SAND BL.	NONE	ALL	NONE
C-4 SWR	3	WATER BL.[2]	NONE	ALL	NONE
C-4 SWR	5	MILL	NONE	TR. JTS.	3-3
C-4 WR	5	MILL	# 4 REINF	TR. JTS.	3-4
C-4 WR	4	SAND BL.	NONE	TR. JTS.	3-5
C-4 WR	4	MILL	NONE	TR. JTS.	NONE
C-4 WR	4	MILL	# 4 REINF	TR. JTS.	3-6
C-4 WR	2	SAND BL.	NONE	TR. JTS.	3-7
C-4 WR	2	WATER BL.	NONE	TR. JTS.	NONE
C-4 SWR	2	WATER BL.	NONE	TR. JTS.	NONE
C-4 SWR	2	SAND BL.	NONE	TR. JTS.	NONE
C-4 SWR	2	SAND BL.	NONE	NONE	NONE
C-4 SWR	2	MILL	NONE	NONE	NONE
C-4 SWR	2	MILL	NONE	TR. JTS.	NONE
C-4 SWR	2	WATER BL.	NONE	TR. JTS.	NONE
C-4 WR	2	SAND BL.	NONE	NONE	NONE
C-4 WR	2	MILL & SAND BL.	NONE	TR. JTS.	NONE

[1] Sand Blasting

[2] Water Blasting

(0.4 l/m^2), which is twice the minimum specified rate for paving in Iowa. This was applied in two coats to avoid runoff of the curing compound.

Joints

Transverse joints were marked with nails and resawed following paving. These were cut to a depth of 1.5 in (38 mm), with the exception of the 2-in (51 mm) thick slab, which had a 1-in (25 mm) sawcut. A longitudinal joint was only sawed on 300 ft (91 m) of the project. Following this project, it was recommended that the transverse joints be sawed full-depth.

Early Performance Observations

A 1979 survey showed that all of the original cracks had reflected through the overlay.[13] The reflected cracks were tighter in the thicker overlays. At the time of the 1980 final report, there were areas of debonding identified. The extent was not specified. However, no areas of debonding other than those identified at the start of the project had been further identified.[14]

This project was surveyed under an earlier FHWA study in July 1985, after approximately 7 years of service. Because there is such a significant difference in the number of ESAL's applied in each direction, the results are separated by lane. These results are presented in table 9. No areas of debonding were noted, although there were several patched areas in place which had been constructed where delaminated pieces of overlay had broken out.

Physical Testing

Pavement cores were retrieved from sections 3-1, 3-3, 3-5, 3-6, and 3-7. All of these cores showed the pavement sections to be in good condition, with no visible deterioration. Results of the shear and split tensile testing are given in the summary tables (appendix B), as are the results of the deflection testing. The average shear strengths obtained from tests run on corner cores ranged from 310 psi (2.14 MPa) to 586 psi (4.04 MPa). The average modulus of rupture of the original concrete was 620 psi (4.27 MPa). The average center slab deflections were fairly high for all of the sections, ranging from 4.7 to 6.6 mils (0.12 to 0.17 mm). The average loaded corner deflections ranged from 15.1 to 29.6 mils (0.38 to 0.75 mm). Load transfer efficiencies were low, ranging from 14 percent to 74 percent. It should be noted, however, that these sections were quite short, with the exception of 3-1 and 3-7, so sufficient test points were not available. Three of the sections were so short that only one mid-slab deflection was obtained: no averages are calculated for those.

Deterioration of the Pavement Sections

The field survey results are presented in table 10. This 10-year-old project displayed significant distress over most of its length. These are briefly analyzed in terms of the different design variables in the following section. It is estimated that

Table 9. Performance data for IA 3 from 1985 survey.

	3-1		3-2		3-3		3-4		3-5		3-6		3-7	
	3-in OL SANDBLAST		3-in OL SANDBLAST REINFORCED		5-in OL MILLED		5-in OL MILLED REINFORCED		4-in OL SANDBLAST		4-in OL MILLED REINFORCED		2-in OL SANDBLAST	
	EB	WB	EB	WB	EB	WB	EB	WB	EB	WB	EB	WB	EB	WB
Transverse Faulting, IN	0.07	0.06	0.07	0.04	0.11	0.11	0.11	0.04	0.09	0.03	0.07	0.12	0.03	0.04
Transverse Crks./MI L	56	19	11	55	0	0	132	229	0	0	60	66	246	106
M	188	143	275	231	317	158	317	70	132	79	24	24	79	31
H	15	11	0	0	0	0	53	0	0	0	0	0	0	0
Long. Crk., LIN FT/MI L	26	0	0	0	0	0	238	0	290	90	26	0	211	280
M	269	53	528	0	42	0	0	502	211	0	0	53	106	106
H	0	0	0	0	0	0	0	0	0	0	0	0	0	0
% Joints Spalled	44.4	22.2	33.3	0	0	0	0	0	0	0	0	0	5.0	5.0
ESAL's on Overlay (in millions)	0.72	0.31	0.72	0.31	0.72	0.31	0.72	0.31	0.72	0.31	0.72	0.31	0.72	0.31

Table 10. Performance data for IA 3 from 1988 survey.

	3-1		3-2		3-3		3-4		3-5		3-6		3-7	
	3-in OL SANDBLAST		3-in OL SANDBLAST REINFORCED		5-in OL MILLED		5-in OL MILLED REINFORCED		4-in OL SANDBLAST		4-in OL MILLED REINFORCED		2-in OL SANDBLAST	
	EB	WB	EB	WB	EB	WB	EB	WB	EB	WB	EB	WB	EB	WB
Transverse Faulting, IN	0.07	N/A	0.08	N/A	0.16	N/A	0.11	N/A	0.17	N/A	0.11	N/A	0.06	N/A
Transverse Crks./MI L	14	14	0	0	132	0	0	66	0	0	0	0	16	4
M	284	185	264	106	330	132	330	132	236	177	462	396	20	24
H	14	28	0	0	0	0	66	66	15	0	264	0	0	0
Long. Crk., LIN FT/MI L	0	0	634	0	0	0	0	0	0	0	0	0	63	0
M	5834	0	4963	0	10098	5280	9834	2574	7065	1755	7920	4554	246	214
H	114	0	0	0	462	0	0	0	15	15	660	0	0	0
% Joints Spalled	66.7	22.2	0	0	0	33.3	0	33.3	54.6	27.3	33.3	0	44.5	27.8
ESAL's on Overlay (in millions)	1.03	0.44	1.03	0.44	1.03	0.44	1.03	0.44	1.03	0.44	1.03	0.44	1.03	0.44

at the time of the field survey, the eastbound lane had experienced 1.0 million ESAL's and the westbound lane 0.4 million ESAL's.

The results of the bonding survey are shown in table 11. Debonding was widespread on all of the sections, especially at cracks, but also at most of the transverse joints. The section with the least overall debonding was the 2-in (51 mm) overlay. The 3-in and 5-in (76 mm and 127 mm) overlays had the most debonding.

Paved bicycle lanes were added to this project in 1981. Also, all random cracks were filled in the summer of 1987, and joints were routed and filled. Two partial depth repairs have been placed on this project.

A comparison of the two surveys from 1985 and 1988, represented by tables 9 and 10, show a continued deterioration of all sections. Most notable are the increases in faulting, spalling, and transverse cracking. By the time of the 1988 survey, the overlay had carried approximately 40 percent more ESAL's than it had in 1985.

Design Features

A strict evaluation of the different design features is not entirely valid, as their effects are confounded. For example, a comparison between milling and sandblasting is confounded by changes in overlay thickness; a comparison between reinforced and nonreinforced sections can be made for 3-in (76 mm) and 5-in (127 mm) sections, but is confounded for 4-in (102 mm) sections by surface preparation. With that in mind, the following comparisons are made. These may also be confounded by different amounts of preoverlay repair and greatly different amounts of preexisting distress.

Reinforcement

A comparison of reinforced versus nonreinforced sections is made in table 12. When the 3-, 4-, and 5-in (76, 102, and 127-mm) overlays with and without reinforcement are compared, no clear trend emerges. There is less joint spalling in the reinforced sections and possibly less longitudinal cracking.

Surface Preparation

Two surface preparation techniques, milling and sandblasting, were evaluated. All of the sections received a final surface preparation of airblasting. A direct comparison can be made for the 4-in (102 mm) overlay. A comparison is also made with the averages of the distresses from the different thicknesses. The results are shown in table 13. In the direct comparison, there is less faulting and spalling in the milled sections and more cracking. In the overall comparison, sandblasted sections had less deterioration except for joint spalling. As table 11 shows, there are not sufficient results to draw conclusions about the effect of surface preparation on bonding.

Table 11. Results of debonding survey for IA 3.

	3-1	3-2	3-3	3-4	3-5	3-6	3-7
	3-in OL SANDBLAST	3-in OL SANDBLAST REINFORCED	5-in OL MILLED	5-in OL MILLED REINFORCED	4-in OL SANDBLAST	4-in OL MILLED REINFORCED	2-in OL SANDBLAST
% DEBONDED JOINT CORNERS	100	50.0	100	N/A	77.8	N/A	92.9
% DEBONDED CRACK CORNERS	100	100	100	N/A	100	N/A	91.7
% DEBONDED AREA OF WHEELPATH	0	0	0	N/A	0	N/A	0
% TOTAL AREA DEBONDED	69.6	72.3	77.9	N/A	46.5	N/A	26.2

Table 12. Comparison of distresses on reinforced and nonreinforced sections on IA 3.

	THICKNESS, IN	FAULTING, IN	TR CRK/ MILE	LONG CRK, LIN FT/MI	% JTS SPALLED
	2	0.06	36	309	45
NON-REINFORCED[1]	3	0.07	312	667	67
	4	0.17	251	1799	55
	5	0.16	462	5280	0
	3	0.08	264	1373	0
REINFORCED[1]	4	0.11	726	3300	33
	5	0.11	396	4554	33

Table 13. Comparison of distresses on milled and sandblasted sections (averaged).

SURFACE PREP.	THICKNESS, IN	FAULTING, IN	TR CRK/ MILE	LONG. CRK, LIN FT/MI	% JTS SPALLED
MILLING	(ALL)	0.13	528	4378	22
	4	0.11	726	3300	33
SAND BLASTING	(ALL)	0.10	216	1037	42
	4	0.17	251	1799	55

[1] The original pavement consisted of 6-in (152 mm) nondowelled JPCP on a granular base.

Thickness

The concrete overlay thicknesses ranged from 2 in to 5 in (51 mm to 127 mm). As table 14 shows, the variation in distresses is rather interesting. As the slabs get thicker, the distresses generally increase. This may well be explained by the placement of thinner overlays on less deteriorated pavements.

Summary and Conclusions

This is the only project which allows for a comparison between some of the different design variables which influence the performance of a bonded overlay. However, these apparent effects are confounded by other variables, particularly cracking of the original pavement before the overlay was constructed.

It should be noted that sections 3-2, 3-3, 3-4, and 3-6 were very short and that extrapolation of the observed distresses has most likely skewed the results. Sections 3-1 and 3-7 were much longer.

The amount of patching and the description of the cracking of the original pavement suggest that the pavement had an extensive amount of deterioration. This pavement receives much more heavy truck traffic in the eastbound lane than in the westbound lane. In general, the distresses were considerably less severe in the westbound lane.

6. S.R. 12—SIOUX CITY, IOWA (IA 4)

This bonded overlay project was constructed in the western part of Iowa, on S.R. 12 in Sioux City. The average annual precipitation in this area is 25 in (635 mm) and it has a Freezing Index of 875. The original pavement, constructed in 1954, consisted of a 9 in (229 mm) nondoweled JPCP, with 20 ft (6.1 m) transverse joint spacing. The pavement had sustained approximately 1,660,000 ESAL's in the outer lane and 180,000 ESAL's in the inner lane prior to placement of the overlay.

Preoverlay Pavement Condition

In 1978, a 3-in (76 mm) bonded concrete overlay was constructed on this pavement. Prior to construction of the concrete overlay, the original pavement had been overlaid several times with asphalt concrete. Because part of the section was located just before a traffic light where heavy decelerating traffic had caused shoving and washboarding, it was decided to construct a bonded concrete overlay.

Overlay

Once the AC overlay had been removed, the surface preparation for the construction of the concrete overlay consisted of milling. Partial depth repairs were also carried out prior to placement of the overlay. These consisted of milling the pavement at deteriorated areas until sound concrete was reached. The extent of deteriorated areas or type of deterioration is not known.

Table 14. Effect of overlay thickness on pavement deterioration.

OVERLAY THICKNESS, IN	FAULTING, IN	TRANSVERSE CRACKS/MI	LONGITUDINAL CRACKING, LIN FT/MI	% JTS SPALLED
2	0.06	36	309	45
3	0.08	288	1020	34
4	0.14	489	2550	44
5	0.14	429	4917	17

Adhesive

A cement-sand grout in a 1:1 ratio was used to bond the overlay to the existing pavement. No further information is available concerning placement of the grout.

Concrete

The standard Iowa C-4 concrete mix was used, with either a water reducing agent (WR) or a super water reducing agent (SWR).

Curing

No information was available on the procedure followed for curing the concrete surfacing.

Joints

The joints were sawed directly over existing joints. The transverse joints were sawed through the overlay and the longitudinal joints were sawed to a depth of 1 in (25 mm). Because the existing joints were not well aligned, it was sometimes difficult to follow the underlying joint pattern.

Early Performance Observations

This section was surveyed in July 1985 under an earlier FHWA research project. The results from that survey are presented in table 15. At that time the observed distresses were minimal.

Physical Testing

Cores were retrieved from both center slab and slab corner locations. These were both in excellent condition, with no noticeable deterioration. Shear tests performed on the corner core indicated a bond of 537 psi (3.70 MPa). Split tensile testing of the original pavement was used to calculate a modulus of rupture of 833 psi (5.74 MPa). The deflections also are indicative of a pavement in good condition, with 100 percent load transfer efficiency at the transverse joints and no voids detected under slab corners. The average mid-slab deflection was 4.5 mils (0.11 mm) and the average loaded corner deflection was a very low 4.1 mils (0.10 mm).

Deterioration of the Pavement Section

The distresses noted during the field survey are shown in table 15. These are representative of a pavement in fairly good condition, although there is a large amount of longitudinal cracking. The distresses in the inner lane are higher than those in the outer lane. It was estimated that at the time of the survey the outer lane had sustained 1.3 million ESAL's and the inner lane had sustained 0.2 million ESAL's. This is about the same amount of traffic carried by the original pavement

Table 15. Performance data for IA 4.

	1985 FHWA SURVEY	FIELD SURVEY 1988	
	OUTER LANE	OUTER LANE	LANE #2
Average PSR	N/A	N/A	2.4
Mays Roughness, IN/MI	N/A	N/A	163
Transverse Faulting, IN	0.04	0.07	N/A
Transverse Cracks/MI L	4	10	5
M	13	55	50
H	0	5	15
Long. Crk., LIN FT/MI L	40	0	0
M	84	100	5260
H	0	0	0
% Joints Spalled	10.0	4.4	29.6
ESAL's on Overlay (millions)	0.87	1.31	0.15

Table 16. Results of debonding survey for IA 4.

% DEBONDED JOINT CORNERS	3.3
% DEBONDED CRACK CORNERS	8.3
% DEBONDED AREA OF WHEELPATH	0
% TOTAL AREA DEBONDED	5.5

prior to construction of the overlay. Results from the debonding survey are presented in table 16. Debonding has begun to develop along cracks and joints, and 5.5 percent of the total slab area showed signs of debonding. This data indicates a progressive deterioration of the overlay, especially in the amount of medium to high severity cracks and faulting.

Summary and Conclusions

At the time that this project was evaluated it had been open to traffic for 10 years. For a project of this age, it was performing well. The excessive longitudinal cracking may be due to the insufficient depth of cut of the longitudinal joint. A small proportion of the slab area showed debonding, which could be a cause for concern in the future.

7. U.S. 20—WATERLOO, IOWA (IA 5)

This bonded concrete overlay section is located in east central Iowa on U.S. 20. This area has a Freezing Index of 868 and an average of 33 in (838 mm) of precipitation annually. The original pavement was constructed in 1958. It consisted of 10 in (254 mm) of nondoweled JPCP on an aggregate base. The transverse joint spacing was 20 ft (6.1 m). Approximately 1,190,000 ESAL's had been applied to the outer lane and 95,000 ESAL's to the inner lane prior to construction of the overlay.

Preoverlay Pavement Condition

The original 10-in (254 mm) slab exhibited extensive D-cracking. There was considerable spalling of the transverse joints, especially near the intersection with the longitudinal joints. Some of these areas had been repaired with bituminous patches.

Overlay

In 1976, a 2-in (51 mm) thick bonded overlay was placed on this pavement. However, extensive work was completed on the pavement prior to construction of the overlay. Partial depth repairs at the joints consisted of additional milling of the deteriorated pavement, sandblasting, grouting, and filling of the patched area with new concrete. This work was performed at 30 joints for the full width of the pavement, and at 5 joints for one-half the width. This additional milling was approximately 2 in (51 mm) deep. Also, full depth repairs were constructed at four locations. Finally, 4-in (102 mm) pressure relief joints were sawed at either end of the project prior to the placement of the overlay and also were constructed in the overlay.

In preparation for placement of the overlay, the entire top 0.25 in (6 mm) of the pavement was milled off. This was followed by sandblasting. Just prior to placement of the overlay, the surface of the pavement was cleaned by airblasting.

Adhesive

The grout used was a 50/50 mix of cement and sand, with enough water added to produce a creamy consistency. The grout was spread on the dry pavement surface with brooms immediately prior to application of the concrete.

Concrete

The concrete was a low slump mix with super water reducers added. Two different water reducing agents were tried. Also, two mixes were used; one had 823 lb of cement/yd^3 (433 kg/m^3) and one had 626 lb/yd^3 (371 kg/m^3). The top size for the coarse aggregate was 0.5 in (13 mm). Paving operations necessitated some hand finishing of surface irregularities.

Curing

Two curing methods, a white pigmented liquid membrane curing compound and wet burlap, were tried. The curing compound was applied at the rate of 0.13 gallon/yd^2 (0.4 l/m^2), or twice the minimum rate specified for Iowa's concrete pavements.

Joints

Existing transverse joints were marked with nails on the shoulder. The resurfacing was then sawed a minimum of 1 in (25 mm) deep over approximately 20 percent of the existing transverse joints. Of the 38 joints where no partial depth patching was done, 7 were sawed after resurfacing. No centerline joint was sawcut in the pavement overlay. After 2 or 3 months, most of the transverse joints had reflected through the resurfacing.

Early Performance Observations

Bonding was checked twice on this section. The first time was shortly after the project was completed, when it was tested in the four wheelpaths. The second time was in the Spring of 1977, when it was checked in the wheelpaths, along the outside edge, at both sides of the centerline joint, and at both sides of random and sawed transverse joints. There was only one small area of debonding discovered at a transverse joint.

The results of the earlier FHWA-sponsored survey, conducted during July 1985, are summarized in table 17. These results show extensive deteriorated cracking.

Physical Testing

Sample cores were obtained from a representative slab corner and center slab location. The center slab core was in good condition, with no noted distresses. The corner core, however, was in poor condition. It had primarily horizontal cracking throughout the aggregate and mortar, extending to within

Table 17. Performance data for IA 5.

	1985 FHWA SURVEY	FIELD SURVEY 1988	
	OUTER LANE	OUTER LANE	LANE #2
Average PSR	N/A	2.4	2.6
Mays Roughness, IN/MI	N/A	174	201
Transverse Faulting, IN	0.07	0.12	N/A
Transverse Cracks/MI L	40	198	168
M	216	228	228
H	0	18	6
Long. Crk., LIN FT/MI L	216	601*	222
M	0	5334*	0
H	0	0	0
% Joints Spalled	0	40.0	50.0
ESAL's on Overlay (millions)	1.07	1.32	0.11

* Centerline joint not sawed.

Table 18. Results of debonding survey for IA 5.

% DEBONDED JOINT CORNERS	83.3
% DEBONDED CRACK CORNERS	76.9
% DEBONDED AREA OF WHEELPATH	0
% TOTAL AREA DEBONDED	45.7

0.25 in (6 mm) of the original surface. A split tensile test performed on the original concrete yielded a modulus of rupture of 676 psi (4.66 MPa). The shear test performed on the center slab core showed a bond of 706 psi (4.87 MPa). However, the shear strength from the corner slab was only 160 psi (1.10 MPa).

The average mid-slab deflection was 2.8 mils (0.07 mm) and the average loaded corner deflection was 13.6 mils (0.35 mm). Load transfer efficiency was 45 percent, and voids were detected under 35 percent of the corners tested.

Deterioration of the Pavement Section

The results from the field survey are summarized in table 17 and show that this 12-year old overlay is in very poor condition. It exhibits extensive cracking and joint spalling, and has noticeable transverse joint faulting. The condition of the inner lane is very similar, and it also had signs of D-cracking in the overlay. There were approximately 1.3 million ESAL's applied on the overlay in the outer lane and 0.1 million applied in the inner lane. This is about 110 percent of the traffic applied on the original pavement before rehabilitation.

A bonding survey was conducted and the results are shown in table 18. It suggests that there is a serious loss of bond developing between the overlay and the original pavement. The debonding is primarily associated with distresses occurring at the joints and cracks, and not in the wheelpath.

Summary and Conclusions

As is noted previously, the original pavement was severely distressed by D-cracking at the time of the overlay construction. D-cracking and spalling had evolved into severe joint deterioration, necessitating widespread repair prior to placement of both the overlay and as preoverlay repair. This pavement is now approaching a failed condition. The original pavement was most likely not a good candidate for the selection of a bonded overlay as the appropriate rehabilitation strategy because of the extensive D-cracking.

8. INTERSTATE 80—TRUCKEE, CALIFORNIA (CA 13)

This thin bonded concrete overlay project is located in a mountainous region of central California on I-80. This area is part of the wet-freeze environmental zone, receives approximately 31 in (787 mm) of rainfall annually, and has a Freezing Index of 1000. The original pavement was constructed in 1964 and consisted of 8 in (203 mm) of nondoweled JPCP on a 4-in (102 mm) cement-treated base (CTB) and a 12-in (305 mm) aggregate subbase. The transverse joint spacing was a random pattern of 12-13-19-18 ft (3.7-4.0-5.8-5.5 m).

Preoverlay Pavement Condition

The original pavement was exhibiting some random cracking when the overlay was constructed. However, the extent and severity are not known. Also, much of I-80 in this mountainous region had experienced a severe loss of wearing surface in the wheelpaths due to the use of chains on tires during periods of inclement weather. Prior to the placement of the overlay in 1984, the outer lane of the pavement had experienced approximately 5,900,000 ESAL's. The inner lane had experienced approximately 860,000 ESAL's.

Overlay

A bonded concrete overlay was constructed on this section in 1984. It was 2 in (51 mm) thick for a distance of 750 ft (229 m) and 4 in (102 mm) thick for 300 ft (91 m). Prior to placement of the overlay, the sealant material in the random cracks was removed by impact hammers. Contraction joints were also cleaned out. The initial surface preparation for the existing concrete pavement consisted of cleaning by shot blasting. The final surface preparation was by airblasting.

Adhesive

An epoxy was used as the bonding agent. It was applied to the existing pavement just prior to the overlay, which was placed within 36 hours of shot blasting. The epoxy consisted of two parts conforming to Section 95 of the CALTRANS Standard Specs and the Special Provisions. The epoxy was hand mixed in 5 gallon (19 l) buckets. It had a pot life of approximately 1 hour and, when sprayed on the pavement, retained adhesive properties for approximately 10 to 15 minutes.

Reinforcement

Reinforcement was placed in the overlay in the inner, or second, lane. This reinforcement consisted of both No. 4 rebar and welded wire. There was no reinforcement used in the outer lane.

Concrete

A Type II modified portland cement concrete was used for the overlay construction. The aggregate in this mix had a maximum top size of 1.5 in (38 mm). Placement temperatures ranged from 46 °F to 86 °F (8 °C to 30 °C).

Curing

Lane 1 was cured with pigmented chlorinated rubber base and pigmented hydrocarbon resin base curing compounds. Lane 2 had either a wet burlap cure on a pigmented chlorinated rubber base or just a pigmented chlorinated rubber base compound.

Joints

The transverse joints were sawed directly over the joints in the existing pavement to the full depth of the overlay and sealed with silicone. The two lanes were paved separately, so no special provisions were made for a longitudinal joint. The specifications required sawing of the transverse joints within 12 hours of paving.

Early Performance Observations

The pavement was chained at least three times prior to October 1984 (after construction and prior to opening) to test for debonding. Prior to acceptance, one panel was found to be debonded. This area was removed and replaced with SET 45 concrete. There is no further performance information available and this section is not one of those surveyed under the earlier FHWA project.

Physical Testing

A core from a representative slab center was retrieved and examined. No notable deterioration was present. Shear testing was not performed; from the split tensile testing a modulus of rupture for the original pavement of 900 psi (6.21 MPa) was obtained. The deflection testing yielded results indicative of a pavement in excellent condition. The average mid-slab deflection was 4.8 mils (0.12 mm) and the average loaded corner deflection was a very low 3.5 mils (0.09 mm). Transverse joint load transfer efficiency was 100 percent and no voids were detected under the slab corners.

Deterioration of the Pavement Section

The distresses measured on this section are summarized in table 19. These indicate a pavement that is performing well, although there are transverse and longitudinal cracks present. It is not known how many of these are reflected cracks. The bonding survey results are in table 20. These results show that an extraordinarily large area of this pavement appears to be debonded and that the apparent debonding is not restricted to joints or the wheelpath area, but covers substantial portions of the entire slab area. An informal survey of this project by Caltrans in 1986 revealed minimal debonding. It is estimated that the overlay had sustained 3.1 million ESAL's in the outer lane and 0.5 million ESAL's in the inner lane at the time of the survey. This is slightly more than 50 percent of the traffic carried by the original pavement until construction of the overlay.

Summary and Conclusions

This pavement has very good serviceability and no faulting. However, there is a fairly large amount of low severity transverse and longitudinal cracking that has not yet deteriorated. Given the extent of the debonding, some type of further deterioration is likely. The cause of the apparent debonding may be related to the performance of the epoxy grout or environmental conditions at the time of paving. An earlier bonded overlay (1981) in the same area failed to develop bond, although in that instance a cement grout was used and the debonding occurred almost immediately.

Table 19. Performance data for CA 13.

	FIELD SURVEY 1987	
	OUTER LANE	LANE #2
Average PSR	4.2	N/A
Mays Roughness, IN/MI	134	N/A
Transverse Faulting, IN	0.0	N/A
Transverse Cracks/MI L	245	136
M	0	0
H	0	0
Long. Crk., LIN FT/MI L	1002	543
M	0	0
H	0	0
% Joints Spalled	3.0	1.5
ESAL's on Overlay (millions)	3.09	0.53

Table 20. Results of debonding survey for CA 13.

% DEBONDED JOINT CORNERS	75.0
% DEBONDED AREA OF WHEELPATH	19.0
% TOTAL AREA DEBONDED	56.8

9. S.R. 38A—SIOUX FALLS, SOUTH DAKOTA (SD 1)

This project is located in the extreme southeastern portion of South Dakota, on State Route 38A. This is a dry-freeze climatic zone, with approximately 25 in (635 mm) of rainfall annually and a Freezing Index of 1000. The original pavement, constructed in 1950, consisted of 8 in (203 mm) of nondoweled JPCP constructed on a 6-in (152 mm) aggregate base. The transverse joint spacing was 15 ft (4.6 m). The project is a two-lane State highway running east-west.

Preoverlay Pavement Condition

A survey conducted prior to construction of the overlay showed that 4 percent of the pavement area required full-depth patching. There were 1,100 linear ft (335 m) of longitudinal cracking recorded, and 60 percent of the transverse joints were spalled. Corner breaks were also noted, as were transverse cracking, delamination, and large areas of asphalt overlay and patching already in place. Several blowups had occurred on the pavement; these had been repaired either with AC patches or full-depth concrete repairs. The pavement had sustained approximately 1,130,000 ESAL's in each direction by the time the overlay was constructed.

Overlay

The 3-in and 4-in (76 and 102 mm) thick, bonded concrete overlay was constructed in 1985. Only the 3-in (76 mm) section was evaluated for this project. Extensive repairs were performed prior to placement of the overlay. These repairs included 51 full-depth patches and additional partial-depth patching. The partial-depth patches were prepared and then filled as part of the overlay paving operation. Four different methods of reinforcing the longitudinal cracks were tried. These included placing tie bars on chairs, placing bent tie bars in predrilled holes on either side of the crack, tying tie bars to reinforcing steel rails running parallel along either side of a crack, and placing the tie bars in sawed slots.

After all of the patching was completed and the undesirable deteriorated material was removed, the pavement was shotblasted. The surface was subjected to two passes of the shotblasting equipment, with the final pass occurring immediately prior to paving.

Adhesive

A cement-water grout was used to adhere the overlay. It was sprayed onto the surface immediately prior to application of the overlay. A 500-ft (152 m) section was placed without grout.

Concrete

The concrete used was a standard class A mix. The aggregate top size was specified to be 0.75 in (19 mm), but 1.5-in (38 mm) aggregate was used instead.

The average slump was slightly over 1.75 in (44.5 mm) and the average entrained air content was 6.25 percent.

Ambient temperatures during placement ranged from 57 °F to 80 °F (19 °C to 27 °C). Three transverse cracks occurred during rapid cooling of the pavement from a severe thunderstorm which produced a cold rain.

Curing

A white pigmented curing compound was applied at the rate of 0.06 gallon/yd^2 (0.2 l/m^2). This was applied in two passes in order to minimize runoff of the compound.

Joints

The original transverse joints on the pavement were extremely crooked, having been formed with redwood inserts. The transverse joints were sawcut the full depth of the overlay as soon as possible after placement of the overlay. The cuts were made across a single lane, guided by two sets of reference pins, in an attempt to compensate for the nonuniformity of the joints. The longitudinal joint was specified to be cut within 48 hours of paving, but was actually sawed at the same time as the transverse joints. In addition, seven 4-in (102 mm) wide pressure relief joints were constructed over the project.

Early Performance Observations

Soon after the sawing was completed, twenty random transverse cracks, most very short, developed. These cracks were routed and sealed with epoxy. During the first day of paving, a random centerline crack occurred on 720 ft (219 m) of overlay prior to sawing of the longitudinal joint. The sawing time was adjusted, but the overlay still developed another 5100 ft (1550 m) of random centerline cracking. These cracks were also sealed, with the sealant material determined by the crack's location. No delamination of the overlay was detected on the project after construction.

A survey made a year later indicated a small amount of reflection cracking. The only cracks that weren't reflection cracks were located along the boundary of full depth repairs in the overlay, or in the next slab. No mention is made of debonding in the reports.

A field survey was performed on this project in July 1985 under an earlier FHWA project, shortly after the pavement was opened to traffic. The results of that survey are summarized in table 21, and show that no distresses were present.

Physical Testing

Cores were taken from the slab center and corner. They were both in excellent condition, with no signs of deterioration or distress. Shear testing was performed and the shear strength between the overlay and the pavement,

Table 21. Performance data for SD 1.

	1985 FHWA SURVEY	FIELD SURVEY 1988	
	EB LANE	EB LANE	WB LANE
Average PSR	N/A	4.2	4.0
Mays Roughness, IN/MI	N/A	59	72
Transverse Faulting, IN	0.02	0.03	N/A
Transverse Cracks/MI L	0	0	0
M	0	0	0
H	0	0	0
Long. Crk., LIN FT/MI L	0	63	0
M	0	73	259
H	0	0	0
% Joints Spalled	0	1.3	6.3
ESAL's on Overlay (millions)	0.11	0.71	0.71

Table 22. Results of debonding survey for SD 1.

% DEBONDED JOINT CORNERS	9.2
% DEBONDED AREA OF WHEELPATH	0
% TOTAL AREA DEBONDED	0.1

measured from a corner core, was 675 psi (4.65 MPa). This was higher than the shear strength obtained from the centerslab core. Split tensile tests were used to obtain a modulus of rupture for the existing pavement of 749 psi (5.16 MPa).

The deflection testing shows a fairly high corner deflection of 15.0 mils (0.38 mm) and an average mid-slab deflection of 4.7 mils (0.12 mm). The transverse joint load transfer efficiency was 46 percent. Voids were detected under 33 percent of the corners tested.

Deterioration of the Pavement Section

A field survey was conducted on this pavement in May 1988, after nearly 3 years of service and 0.71 million ESAL's in each lane. This is over 60 percent of the traffic carried by the pavement in the 25 years of service prior to construction of the overlay. The distresses were nominal, consisting of some longitudinal cracking and slight transverse joint spalling. These results are shown in table 21.

A bonding survey was also performed and showed that very little of the pavement was debonded. The only debonded areas consisted of 9 percent of the slab corners tested. These results are shown in table 22.

Summary and Conclusions

This bonded overlay is performing very well after 3 years of service and approximately 710,000 ESAL's. This good performance is occurring despite the pre-existing distresses and the need for widespread pre-overlay repairs. It is not known whether the good performance is a result of some design factor or whether the low number of applied ESAL's has kept down the presence of deterioration.

10. INTERSTATE 25—DOUGLAS, WYOMING (WY 1)

This project is located on I-25, in the southeastern corner of Wyoming. This is a dry-freeze area that has an average annual rainfall of approximately 13 in (330 m) and a Freezing Index of 750. The original pavement, constructed in 1969, was an 8-in (203 mm) thick nondoweled JPCP on an aggregate base, with 20-ft (6.1 m) transverse joint spacing. It is estimated that the outer lane of the pavement had sustained 1,970,000 ESAL's prior to construction of the bonded overlay.

Preoverlay Pavement Condition

The original pavement was relatively sound, with some transverse and longitudinal cracking and corner breaks evident in limited areas. Minor pumping and faulting were observed throughout the project. Maintenance operations up to the time of the 1983 overlay construction included sporadic joint resealing and maintenance patching with a cold mix bituminous mix, according to the Wyoming State Highway Department.

In 1983, many different repairs were carried out on this section. These included: subsealing in the outer lane for the entire length of the project with a cement-pozzolan grout, full and partial depth repairs, and joint and crack resealing.

Overlay

In 1983, a 3-in (76 mm) thick, bonded concrete overlay was constructed. Prior to placement of the overlay, the pavement was milled to a depth of 0.5 in (13 mm). This was followed by sandblasting. The final surface preparation consisted of airblasting immediately prior to the application of the bonding agent.

Adhesive

A cement-water mixture was used as the bonding agent, with a maximum water-cement ratio of 0.62. This was sprayed on the pavement at a maximum distance of 8 ft (2.4 m) ahead of the paver.

Concrete

The concrete was a standard, low-slump portland cement mixture. A Type II cement was used in a 6.25 sack mix. The design water-cement ratio was 0.446, the air content was 5.4 percent, and the slump was 1.75 in (44 mm). The temperature range during the paving period was from 65 °F to 86 °F(18 to 30 °C).

Curing

The pavement was cured with a white pigmented curing compound. No information was available about the rate or method of application.

Joints

The transverse joints were sawed full depth above the marked joints of the original pavement, except for at several locations. They were then sealed with a silicone sealant before the pavement was opened to traffic. The longitudinal joint was sawed along the middle of the overlay slab, to an initial depth of 2 in (51 mm). However, the original longitudinal joint was formed with an insert, which may have made it difficult to locate and follow the longitudinal joint.

Early Performance Observations

This pavement was evaluated several times by the Wyoming State Highway Department. Deflection testing taken shortly after construction showed that one area appeared not to have been successfully undersealed and that there were 3 slabs that appeared not to be bonded. Cores taken less than a year after construction showed shear strengths ranging from 223 to 360 psi (1.54 to 2.48 MPa). A year after construction, there were a few interior corners that had become debonded and broken out. The extent of this problem was considered minimal.

A subsequent survey in 1985 also revealed very little deterioration. However, a survey conducted by the State in the spring of 1986 showed extensive deterioration, consisting of fine transverse cracks spaced 6 in to 9 in apart (152 to 229 mm) over the entire project. There were many areas of the project that were experiencing more severe cracking and several areas that had broken up.

This pavement was next evaluated by the FHWA in 1986. Debonding was noted at the intersection of reflected transverse and longitudinal cracks. There were also about a dozen instances of small corner breaks with associated debonding. Some transverse cracks occurred in relation to missawed joints. There was minor cracking attributable to reflection of underlying cracks or joints. Some of the cracks had developed minor spalling. According to the Wyoming State Highway Department, this cracking had begun to develop during the winter and spring of 1985-6.

This project was also surveyed in June 1986. The distresses noted during that survey are summarized in table 23. At that time, researchers observed transverse cracks and shrinkage cracking on nearly 90 percent of the survey section, which was attributed to increased ambient temperatures during the curing period. No visible debonding was noted.

Physical Testing

A joint core was retrieved from a representative slab corner. There was no bond between the existing pavement and the overlay, as the overlay section was totally separated from the underlying pavement. Approximately 0.5 in (13 mm) of cement grout was recovered from the bottom of the core, no doubt from the previous subsealing operation.

Since there was no bond, shear testing was not performed. Split tensile testing was performed and showed a modulus of rupture for the original concrete of 805 psi (5.55 MPa). The deflection tests gave a transverse joint load transfer efficiency of 52 percent. Voids were detected under 20 percent of the corners tested. The average mid-slab deflection was 4.5 mils (0.11 mm) and the average loaded corner deflection was 11.0 mils (0.28 mm).

Deterioration of the Pavement Section

Table 23 presents the results of the 1988 field survey. Overall, the pavement is showing significant deterioration, with a large number of medium severity transverse cracks and extensive longitudinal cracking. The distresses are notably higher in the outer lane than in the inner, less travelled lane. The 1988 survey shows increased deterioration from the 1985 survey in terms of all distresses, and especially cracking and joint spalling.

The bonding test results are summarized in table 24. There appears to be a debonding problem at over one-half of the corners and in 1 percent of the wheelpaths. This is a large amount of debonding for a 4-year-old project. The loads applied to the overlay at the time of the survey were 1.9 million ESAL's in

Table 23. Performance data for WY 1.

	1986 FHWA SURVEY	FIELD SURVEY 1988	
	OUTER LANE	OUTER LANE	LANE #2
Average PSR	N/A	4.2	4.0
Mays Roughness, IN/MI	N/A	82	113
Transverse Faulting, IN	0.01	0.04	N/A
Transverse Cracks/MI L	21	158	0
M	0	42	0
H	0	0	0
Long. Crk., LIN FT/MI L	42	232	0
M	0	2165	528
H	0	296	11
% Joints Spalled	8.3	23.0	3.8
ESAL's on Overlay (millions)	1.12	1.90	0.13

Table 24. Results of debonding survey for WY 1.

% DEBONDED JOINT CORNERS	52.9
% DEBONDED AREA OF WHEELPATH	1.0
% TOTAL AREA DEBONDED	3.6

the outer lane and 0.1 million ESAL's in the inner lane. This is over 95 percent of the traffic carried by the original pavement prior to construction of the overlay. The amount of longitudinal cracking appears to be the most serious problem, as much of it is deteriorated. The transverse cracking could also further deteriorate and develop into excessive roughness.

Summary and Conclusions

This project is in fairly good condition, with a high PSR, but the debonding observed suggests that further cracking may occur. The original pavement required extensive rehabilitation prior to the overlay construction. This may be an indication that a bonded overlay might not have been the most appropriate rehabilitation strategy. Plans are underway to perform extensive rehabilitation on this project during the 1989 season. This rehabilitation is to include crack repair, spall repair, and sawing and resealing of all joints.

11. U.S. 61—BATON ROUGE, LOUISIANA (LA 1)

This project is located on U.S. 61, in eastern Louisiana, near Baton Rouge. This is classified as a wet non-freeze environmental zone, with an annual average precipitation of 56 in (1422 mm) and a Freezing Index of 0. The original pavement was constructed in 1959, and consisted of a 9-in (229 mm) JPCP slab with 1.13-in (28.6 mm) diameter dowels, constructed over 6 in (152 mm) of sand on a heavy clay embankment. Joint spacing was 20 ft (6.1 m) and steel dowel bars were used. Prior to placement of the overlay, the pavement had sustained approximately 2,690,000 ESAL's in the outer lane and 280,000 ESAL's in the inner lane.

Preoverlay Pavement Condition

Embankment settlement and erosion of the sand subbase had caused vertical displacement of some of the slabs, although transverse joint faulting was minimal. Some slab cracking was caused by misaligned dowel bars. The pavement was structurally sound, but had a rough ride and a PSI of 2.3.

Overlay

The 4-in (102 mm) thick, bonded concrete overlay was constructed in 1981. Before the overlay could be placed, there were minor areas of the pavement which required repair. The areas with cracking caused by misaligned dowels were repaired using partial depth patching prior to surface preparation. Three 4-in (102-mm) wide pressure relief joints were constructed approximately 2000 ft (610 m) apart. The transverse contraction joints were sawed out and then cleaned with a rotary wire brush. The cleaned joints were temporarily sealed with compression sealants during the surface preparation and then permanently filled with cotton rope prior to placement of the overlay. Deformed tie-bars were placed over some areas of cracking to serve as reinforcement. Longitudinal edge drains were also constructed prior to placement of the overlay. These consisted of a 4-in (102 mm) diameter slotted pipe placed in a filter-lined, gravel filled trench which was excavated at the pavement-shoulder interface.

Because of the hard aggregate used on this project, the surface was shotblasted. This method removed approximately 0.13 in (3 mm) of pavement surface. Immediately prior to placement of the overlay the surface was cleaned with compressed air.

Adhesive

A water-cement grout was used in a ratio of approximately 0.62. It was applied to the dry pavement using a sweeping operation. Delays between application of the grout and placement of the overlay were restricted to less than 90 minutes.

Concrete

The concrete used was Louisiana's standard slip-form mix, with the substitution of crushed limestone for the normal aggregate, and a change in the gradation. The coarse aggregate had a maximum size of 1.0 in (25 mm). The specifications called for an entrained air content of 5 $\pm$ 2 percent and a slump of 1 to 2.5 in (25 to 64 mm).

Curing

A white pigmented curing compound was applied following the surface finishing operations. The application rate was one and a half times the normal rate in use in Louisiana.

Joints

The location of the transverse joints was marked in the AC shoulder. After cutting the pressure relief joints, these needed to be re-marked because of movement of the pavement. The transverse joints were sawed 0.5 in (13 mm) wide for the full depth of the overlay. The pressure relief joints were sawed 2 in (51 mm) wide, as the underlying 4 in (102 mm) joints had closed to 2 in (51 mm) by the time of paving. The longitudinal joint was sawed to a depth of 1 in (25 mm) on half of the project, and left unsawed on the other half.

Early Performance Observations

Several cracks occurred in the pavement adjacent to one of the PRJ's about 6 months after construction. Extensive debonding was found on the slabs on either side of the joint. These slabs were rebonded to the existing slab with epoxy. A debonding survey was performed 6 months after construction on the entire pavement. It showed that 8 percent of the exterior slab corners were debonded. This was monitored for 4 years, and at the time of the FHWA report had increased to 16 percent of the exterior slab corners, of which 12 percent had cracked on the diagonal. In 1988, the bonding survey showed approximately 10.3

percent of the surface area of the overlay and 93 percent of the corners had apparently debonded.

This section was also surveyed in June 1986 under an FHWA research project. The distresses from that survey are summarized in table 25. The researchers found very little reflective transverse cracking and no longitudinal cracking, except for the reflected centerline joint. Transverse cracks were found near 12 percent of the transverse joints. Some surface scaling was observed. A number of possible causes are proposed for these, including spalling of the original pavement, inadequate bond, or excessive air content in the concrete mix.

Physical Testing

Cores were recovered from this project at both a slab center and a transverse joint. Complete bonding was observed on both; in the joint core, the original pavement was separated horizontally at its mid-depth. Split tensile testing performed on the original concrete gave a modulus of rupture of 929 psi (641 MPa). The FWD testing showed a very low mid-slab deflection of 2.8 mils (0.07 mm) and a low average corner deflection of 6.4 mils (0.16 mm). The load transfer efficiency was a fairly high 88 percent.

Deterioration of the Pavement Section

By 1988, it is estimated that the outer lane had experienced 2.1 million ESAL's and the inner lane had experienced 0.3 million ESAL's. The 1988 distress survey was conducted at the wrong site, although the physical testing and debonding was done on the overlay. The results of the bonding survey are shown in table 26.

Summary and Conclusions

This pavement has carried as much traffic since the overlay was constructed as it had before. The apparent debonding is a major problem that requires further investigation.

Table 25. Performance data for LA 1.

	1986 FHWA SURVEY	FIELD SURVEY 1987*	
	OUTER LANE	OUTER LANE	LANE #2
Average PSR	3.7	N/A	N/A
Mays Roughness, IN/MI	N/A	N/A	N/A
Transverse Faulting, IN	0.02	N/A	N/A
Transverse Cracks/MI L	15	N/A	N/A
M	0	N/A	N/A
H	0	N/A	N/A
Long. CRk., LIN FT/MI L	0	N/A	N/A
M	0	N/A	N/A
H	0	N/A	N/A
% Joints Spalled	N/A	N/A	N/A
ESAL's on Overlay (millions)	1.47	2.09	0.32

Table 26. Results of debonding survey for LA 1.

% DEBONDED JOINT CORNERS	93.1
% DEBONDED AREA OF WHEELPATH	0
% TOTAL AREA DEBONDED	10.3

CHAPTER 4 EVALUATION OF SELECTED DESIGN AND ANALYSIS MODELS

1. INTRODUCTION

Several design procedures and predictive models have been developed over the last several years to aid in the design and evaluation of bonded concrete overlays. Typically, the design procedures provide the engineer with a design thickness for the bonded concrete overlay. The engineer can then evaluate the use of a bonded overlay as a rehabilitation alternative on the basis of cost and projected performance period. The predictive models provide an indication of the performance of the concrete overlay in terms of the expected distresses.

In this chapter, the conceptual basis of several models and their inherent limitations are presented in terms of the underlying theory and assumptions supporting these models. The design models which are discussed include: American Association of State Highway and Transportation Officials (AASHTO)—Guide for the Design of Pavement Structures (1986 Revision); University of Texas—Thin Bonded Concrete Overlay 1 (TBCO1); the Federal Aviation Administration (FAA) procedure; and the Construction Technology Laboratory (CTL) and Portland Cement Association (PCA) method, as presented to the Iowa Department of Transportation. The predictive models considered are those developed under a previous FHWA project.[1]

Where applicable, these models will be used to compare predicted and actual results. For the design procedures, a comparison of a design thickness to the actual constructed thickness will be conducted. For the predictive models, the predicted level of distresses for the estimated traffic level will be compared to the actual observed distresses.

2. CONCEPTUAL BASIS OF MODELS

AASHTO

The AASHTO design procedure, as presented in the *AASHTO Guide for Design of Pavement Structures—1986*, is an empirical approach based on the concept of structural deficiency.[15] The general form of the design equation is as follows:

$$SC_{OL}^{n} = SC_{y}^{n} - F_{RL}(SC_{xeff})^{n} \qquad (2)$$

Where:

SC_{OL} = Structural capacity required for the overlay
SC_{y} = Structural capacity required for new pavement
F_{RL} = Remaining life factor or pavement condition factor for both the original pavement and the desired degree of damage for the overlay

SC_{xeff} = Structural capacity of the original pavement
n = Bonding factor (1.0 for bonded overlays)

This general approach "is applicable to all types of overlay placed on any type of pavement structure."[15] The structural capacity notation was adapted to keep the notation and procedure consistent for the design of either flexible or rigid overlays. The implicit assumption is that many of the basic steps in the design of overlays are identical regardless of pavement type. The structural capacity of a flexible pavement is a function of the structural number of the pavement system, whereas in rigid pavements the structural capacity is a function of the thickness of the slab. To apply equation 2 to the design of a rigid overlay, the structural capacity of the overlay and the new pavement are equal to the thickness of those layers. The effective structural capacity of the existing slab is the thickness of the slab adjusted for the pavement's current condition. The AASHTO procedure for the design of rigid overlays is based on figure 6.[16]

The thickness required for a new pavement can be determined using the AASHTO design method for new pavements, or any other appropriate new design method. The steps involved in the AASHTO procedure for rigid overlay design are listed below:

1. Determine the total traffic, including up to the time of the overlay and the expected traffic for the overlay.

2. Calculate the elastic modulus of each layer.

3. Compute the composite *k*-value on top of the base.

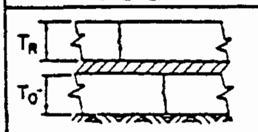
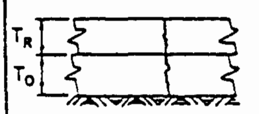
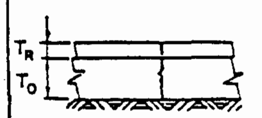
4. Obtain the design properties of the overlay materials.

5. Calculate the effective thickness of the existing pavement.

6. Determine the thickness of a new pavement required to carry the traffic calculated for the overlay to the design terminal serviceability.

7. Calculate the remaining life factor, which is based on both the remaining life of the existing pavement and the remaining life of the overlay pavement after it has reached its terminal design serviceability.

The determination of the effective thickness of the existing pavement is based on the condition of the pavement at the time of overlay. Four approaches to determine the effective thickness are given in the *Guide* and are discussed below. It is assumed, although not implicitly stated, that these are equivalent methods.

TYPE OF OVERLAY	UNBONDED OR SEPARATED OVERLAY	PARTIALLY BONDED OR DIRECT OVERLAY	BONDED OR MONOLITHIC OVERLAY
			
PROCEDURE	CLEAN SURFACE DEBRIS AND EXCESS JOINT SEAL PLACE SEPARATION COURSE-PLACE OVERLAY CONCRETE.	CLEAN SURFACE DEBRIS AND EXCESS JOINT SEAL AND REMOVE EXCESSIVE OIL AND RUBBER-PLACE OVER-LAY CONCRETE	SCARIFY ALL LOOSE CON-CRETE, CLEAN JOINTS, CLEAN AND ACID ETCH SURFACE - PLACE BONDING GROUT AND OVERLAY CONCRETE.
MATCHING OF JOINTS IN OVER-LAY & PAVEMENT — LOCATION	NOT NECESSARY	REQUIRED	REQUIRED
MATCHING OF JOINTS IN OVER-LAY & PAVEMENT — TYPE	NOT NECESSARY	NOT NECESSARY	REQUIRED
REFLECTION OF UNDERLYING CRACKS TO BE EXPECTED	NOT NORMALLY	USUALLY	YES
REQUIREMENT FOR STEEL REINFORCE-MENT	REQUIREMENT IS INDEPENDENT OF THE STEEL IN EXISTING PAVEMENT OR CONDITION OF EXISTING PAVEMENT.	REQUIREMENT IS INDEPENDENT OF THE STEEL IN EXISTING PAVEMENT. STEEL MAY BE USED TO CONTROL CRACKING WHICH MAY BE CAUSED BY LIMITED NON-STRUCTURAL DEFECTS IN PAVEMENT.	NORMALLY NOT USED IN THIN OVERLAYS. IN THICKER OVERLAY STEEL MAY BE USED TO SUPPLEMENT STEEL IN EXISTING PAVEMENT.
T_R SHOULD BE BASED ON THE FLEXURAL STRENGTH OF	OVERLAY CONCRETE	OVERLAY CONCRETE	EXISTING CONCRETE
MINIMUM THICKNESS	6"	5"	1"
APPLICABILITY OF VARIOUS OVERLAY TYPES — STRUCTURAL CONDITION OF EXISTING PAVE-MENT — NO STRUCTU-RAL DEFECTS C = 1.0*	YES	YES	YES
LIMITED STR-UCT. DEFECTS C = 0.75*	YES	ONLY IF DEFECTS CAN BE REPAIRED	ONLY IF DEFECTS CAN BE REPAIRED
SEVERE STR-UCT. DEFECTS C = 0.35*	YES	NO	NO
SURFACE CRACKS, SCALING, SPALL-ING AND SHRINK-AGE CRACKS — NEGLIGIBLE	YES	YES	YES
LIMITED	YES	YES	YES
EXTENSIVE	YES	NO	YES

* C VALUES APPLY TO STRUCTURAL CONDITION ONLY, AND SHOULD NOT BE INFLUENCED BY SURFACE DEFECTS.

Figure 6. Summary of concrete overlays on existing concrete pavements. (15)

Nondestructive Testing Method (NDT)

The *Guide* recommends the use of NDT to determine the in situ material properties of the pavement system. For rigid pavements, the *Guide* suggests using elastic layer theory to backcalculate the modulus values for the pavement layers. However, the *Guide* gives no recommendations on testing location, which is critical in the determination of in-situ material properties. A computerized solution is recommended due to the mathematical complexities of the solution. Once the modulus values for all layers are determined, figure 7 is used to determine the effective thickness of the pavement. However, it should be noted that several of the assumptions of elastic theory are violated when they are applied to rigid pavements; therefore, it is not generally recommended for this application. [6,17,18]

Visual Condition Factor Method

A visual condition factor is determined based on the condition of the existing slab. A relationship between the modulus of the cracked slab and visual condition factor is presented in the *Guide*. The modulus of the existing slab is determined as a percentage of the slab's original modulus (see figure 8). Figure 7 is then entered with the slab modulus value to determine the effective thickness.

Nominal Size of PCC Fragments

A relationship between the nominal size of the slab fragments and the modulus of a cracked slab (as a function of its original modulus) is also presented (see figure 9). The modulus of the cracked slab is used with figure 7 to determine the effective thickness.

Remaining Life Approach

The remaining life of the pavement may be estimated using several different techniques, described below. A relationship is presented between the remaining life value and the pavement condition factor, C_x, as shown in figure 10. This is an estimate of the amount of structural integrity remaining in the slab. The designer may use this factor to determine the effective thickness by direct multiplication of the structural condition factor by the existing thickness.

The *Guide* states that performing extensive rehabilitation prior to placement of the overlay will improve the pavement's structural capacity. This will increase the remaining life factor and reduce the required thickness of the overlay. It is not clear if this approach applies to bonded overlays, however, as it is usually assumed that they are constructed on structurally sound pavements. Preoverlay rehabilitation on these pavements is intended to restore the deteriorated areas to a condition where the overlay will not fail, and not to increase the structural capacity of the pavement.

The NDT method is the only one of the four methods which allows a truly analytical approach to the determination of the effective slab thickness. The other

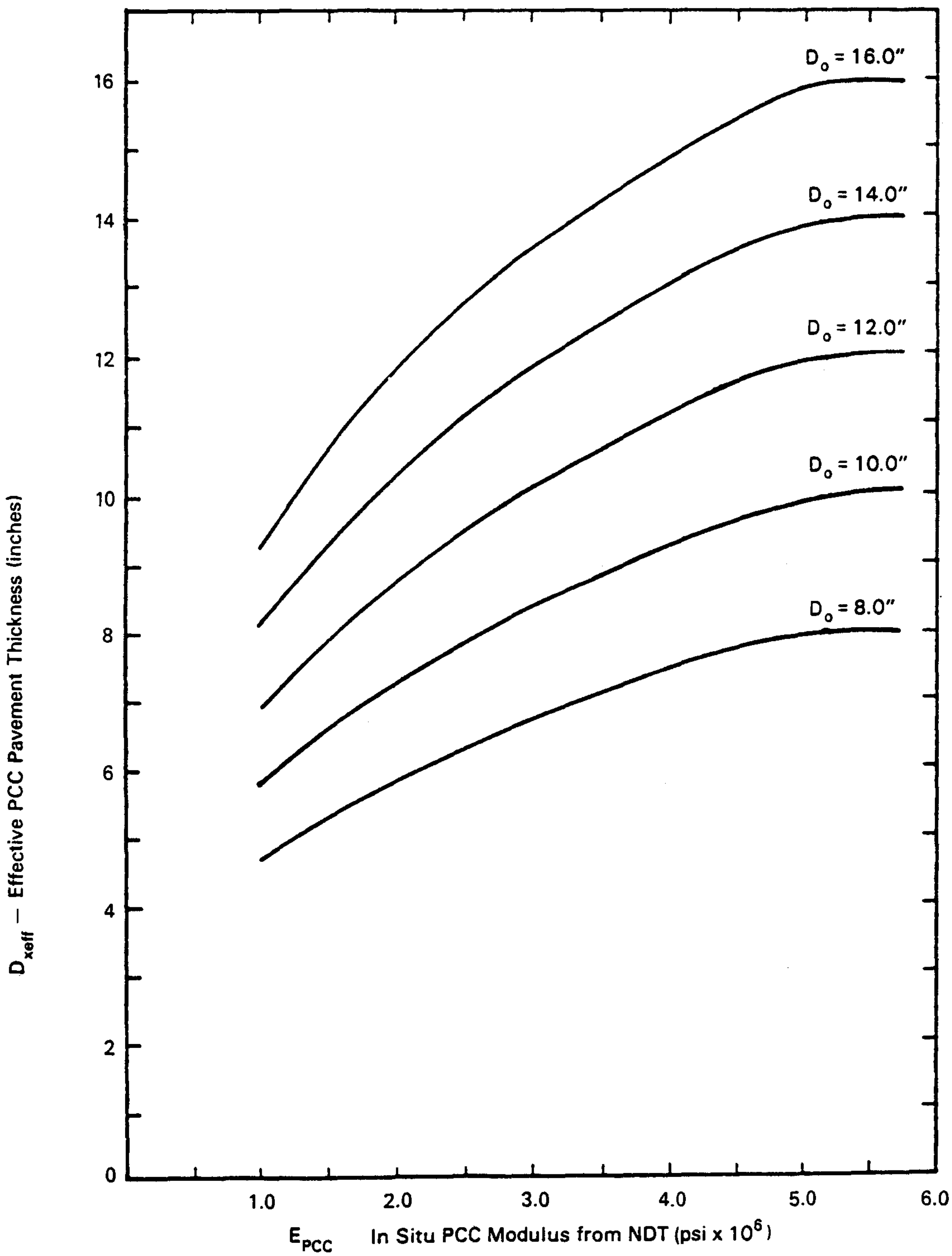


Figure 7. Determination of effective structural capacity (thickness) from NDT-derived modulus values. (15)

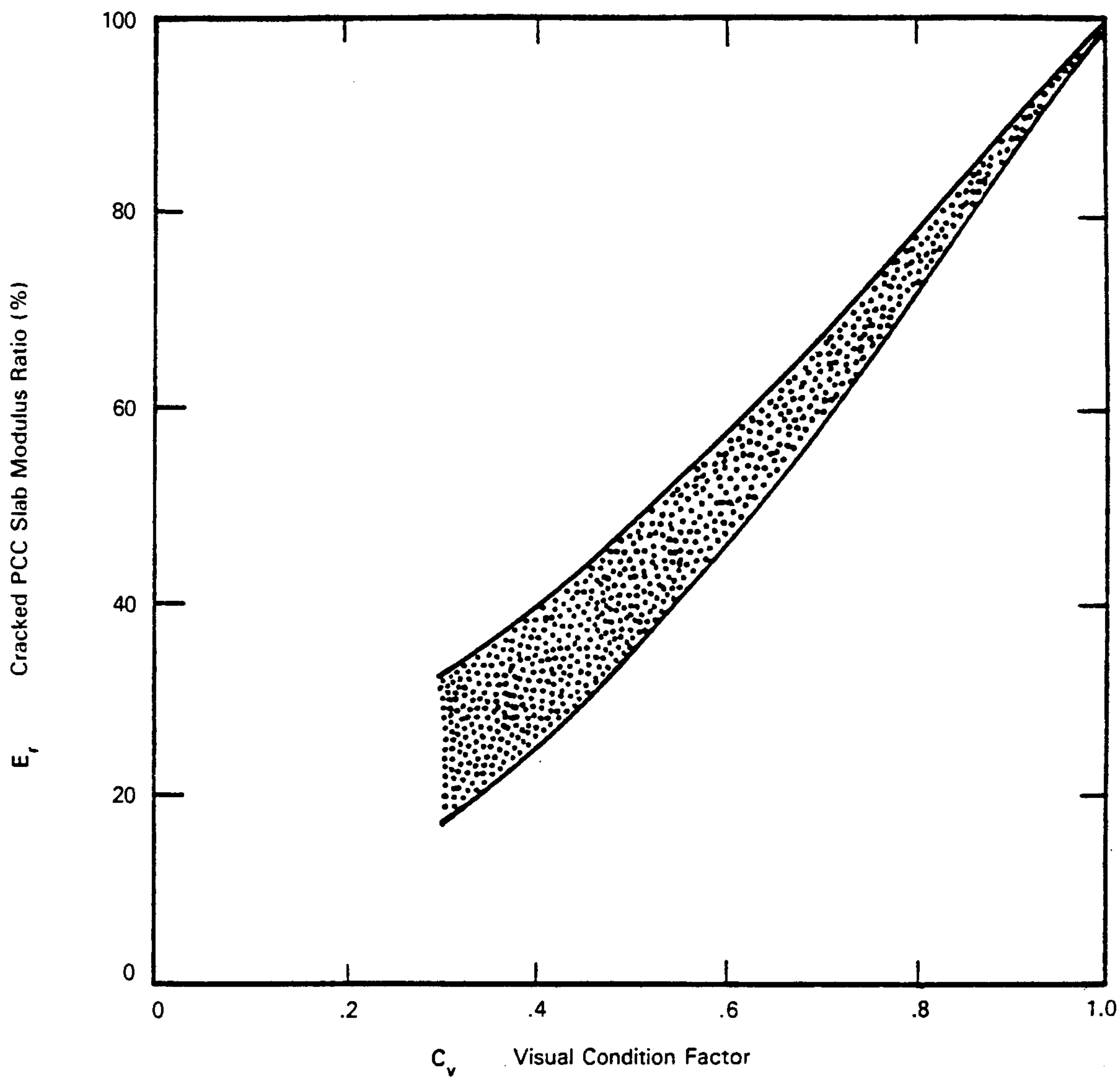


Figure 8. Relationship of visual condition factor to modulus of a cracked rigid pavement.(15)

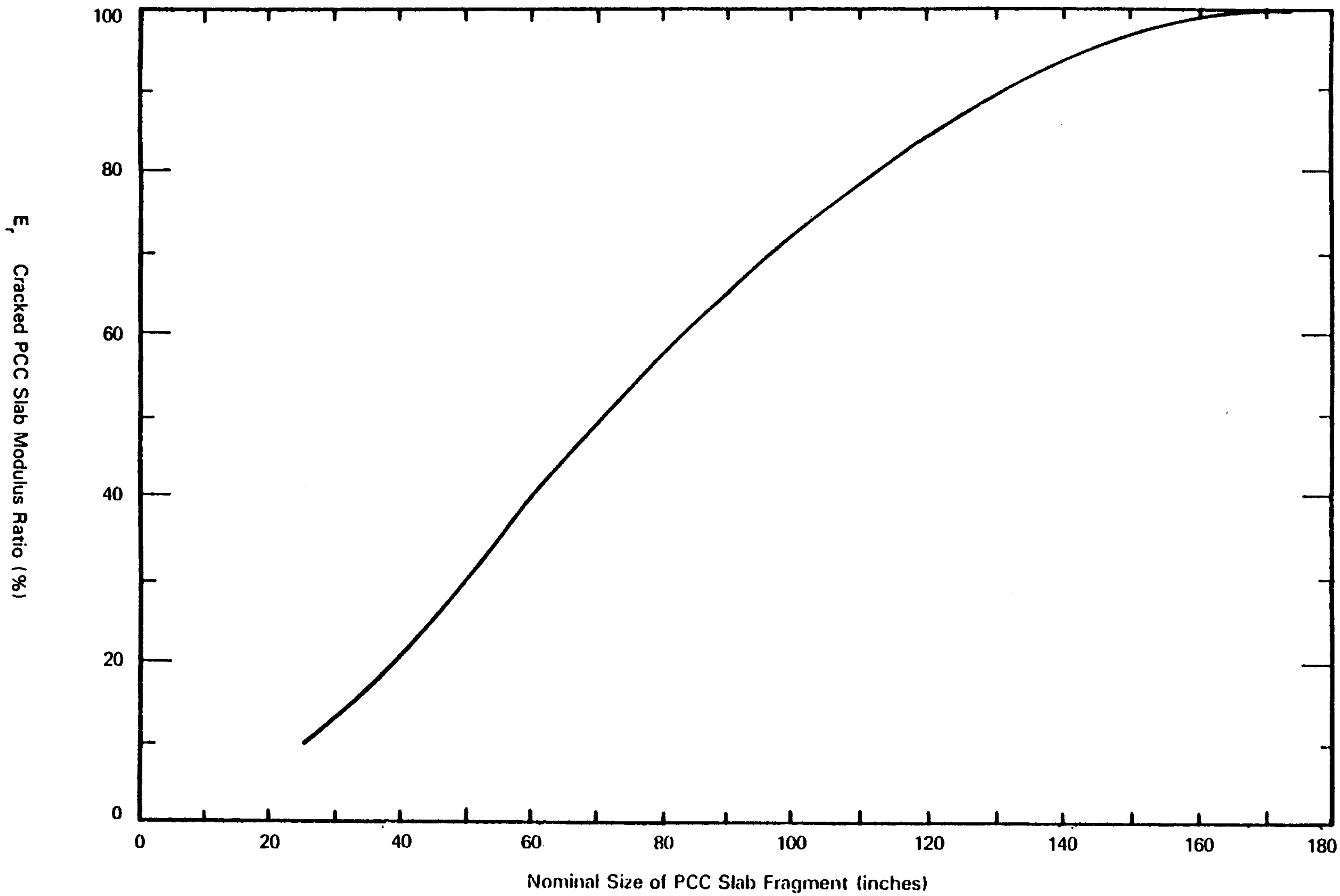


Figure 9. Relationship of slab fragment size to modulus of a cracked rigid pavement.(15)

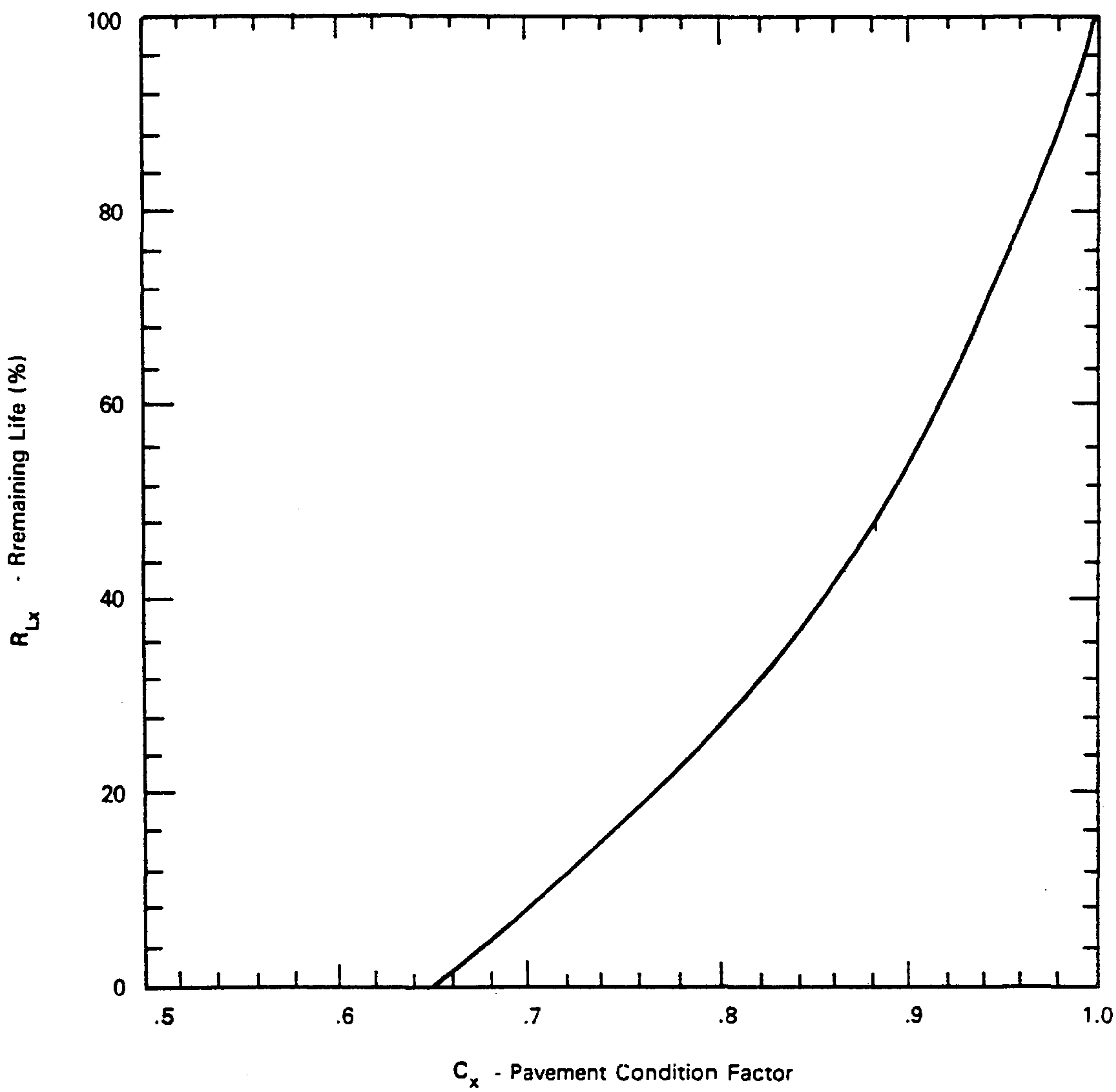


Figure 10. Remaining life estimate predicted from the pavement condition factor.(15)

methods rely on the pavement's visual condition and engineering judgment to determine the effective slab thickness. All of the methods rely on tables and figures which have not been substantiated through field verification.

If substantial surface preparation (e.g, grinding) reduces the pavement thickness then the existing structural capacity must be correspondingly reduced.

Selection of the Remaining Life Factor

One of the controversies associated with this design method surrounds the selection of the remaining life factor. It is this factor which reduces the thickness of the existing pavement to reflect its remaining life. As the allowable values for F_{RL} range from 0.57 to 1.0, the effect on the structural capacity of the existing pavement, and hence the required overlay thickness, is dramatic. The remaining life factor is dependent upon the percent remaining life of the existing pavement prior to overlay (R_{Lx}), and the percent remaining life of the overlay pavement after its terminal serviceability is reached (R_{Ly}). This concept is shown in figure 11.

Five methods are presented in the *Guide* which allow for the determination of the remaining life value of the existing pavement (R_{Lx}). Again, it is suggested that they are theoretically equivalent methods. The five methods are based on: nondestructive deflection testing; traffic; time; serviceability; and visual condition.

<u>Nondestructive Deflection Testing Approach</u>

If the effective thickness of the existing slab has been determined through the NDT method presented above, then the structural condition factor is determined by dividing the effective thickness by the original thickness. A relationship between the structural condition factor and the R_{Lx} value is presented in the Guide.

<u>Traffic Approach</u>

The *Guide* suggests that if accurate traffic information is available regarding the traffic history of the existing pavement, then estimates of R_{Lx} can be found easily, using the following equation:

$$R_{Lx} = (N_{fx} - x)/N_{fx} \tag{3}$$

Where:

N_{fx} = Number of design 18-kip (80 kN) Equivalent Single Axle Loads (ESAL's) for the existing pavement
x = Number of accumulated ESAL's on existing pavement

<u>Time Approach</u>

A relationship is presented between the time a highway section has been in service prior to overlay, the best estimate of the probable time that the particular

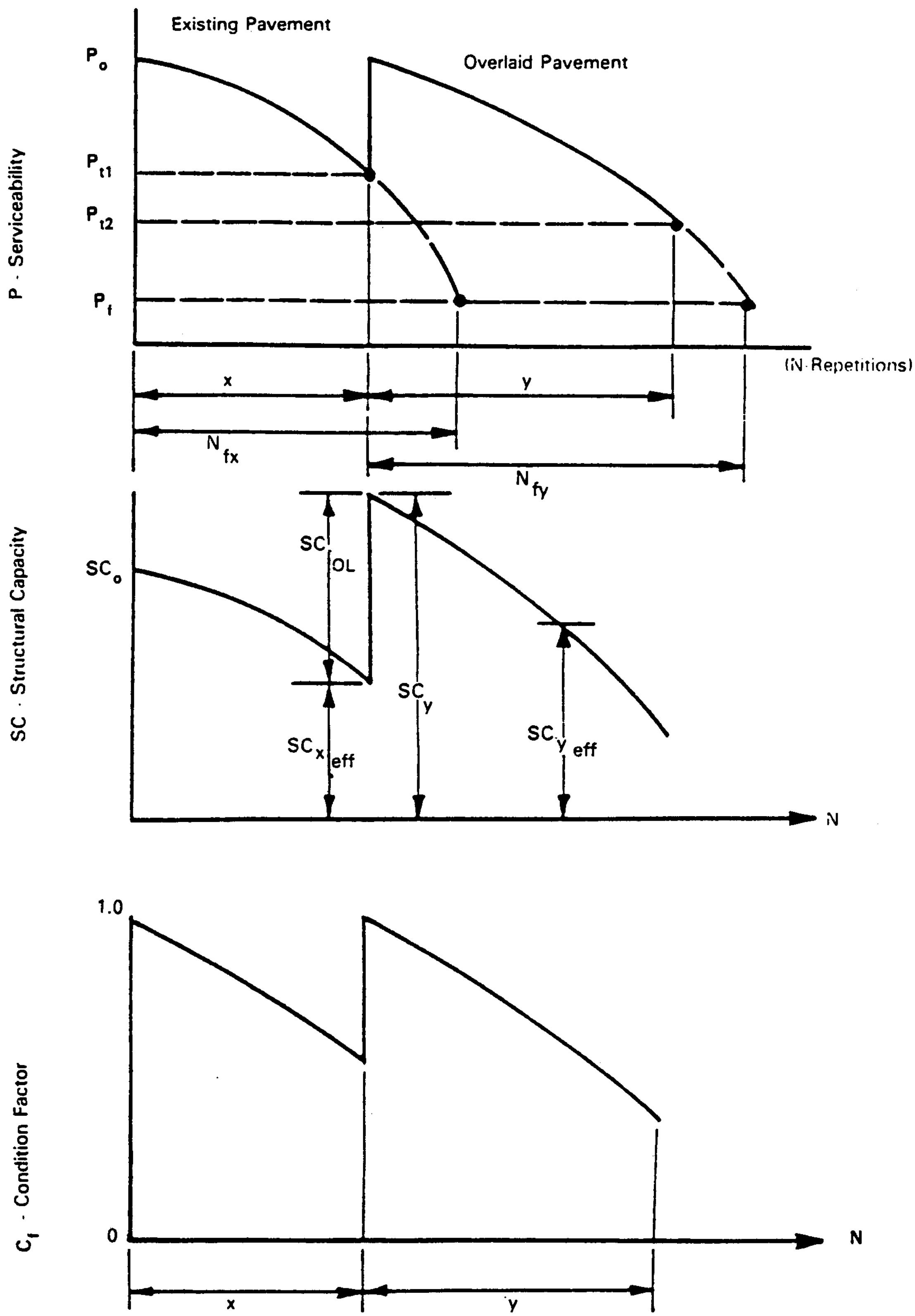


Figure 11. Relationship between serviceability, structural capacity, condition factor, and traffic.(15)

pavement lasts before an overlay is required, the annual traffic growth rate, and the R_{Lx} factor.

Serviceability Approach

The *Guide* presents a relationship between the serviceability rating at the time of the overlay, the thickness of the existing pavement, and the R_{Lx} value.

Visual Condition Survey Approach

An overall visual condition factor is determined based on the condition of the existing pavement. This factor is related to the R_{Lx} value through a graphical relationship.

It has not been shown that these are equivalent approaches and, in fact, Figure 5-16 of the *AASHTO Guide* shows the general applicability of each method for different levels of pavement damage or remaining life. The Guide recommends that several of the procedures be utilized to arrive at a better estimate of R_{Lx}.

The determination of R_{Ly}, the percent life remaining in the overlay after it has reached its terminal serviceability, is based on the agency's predetermined terminal serviceability. The *Guide* recommends terminal serviceability levels based on functional classification.

Once R_{Lx} and R_{Ly} have been determined, figure 12 is entered to obtain the remaining life factor, F_{RL}. This factor further reduces the effective thickness of the existing pavement.

Uncertainties in the procedure arise from the fact that a number of the recommended steps have never been validated through field use. For example, there is a good deal of controversy surrounding figure 12, by which the selection of F_{RL} is made.[19] Of particular concern is the notion that the remaining life factor actually increases for pavements which are getting older in a certain portion of the graph. This occurs where the remaining life of the overlay pavement is less than 30 percent and the remaining life of the existing pavement is between 20 percent and 0 percent. Since this approach has not been verified, caution must be observed in its use.

University of Texas—TBCO1

A concrete overlay design method is presented in reference 20. This design procedure was based on a study of a thin, bonded, continuously reinforced concrete pavement (CRCP) overlay of a CRC pavement on South Loop 610, near Houston, Texas. The structural design procedure is incorporated within a computer program entitled TBCO1. The procedure and program were developed for the design of CRCP overlays over CRC pavements. It has been expanded for use with jointed plain concrete pavement (JPCP) and jointed reinforced concrete pavement (JRCP). It is pointed out in the report, however, that no data from thin

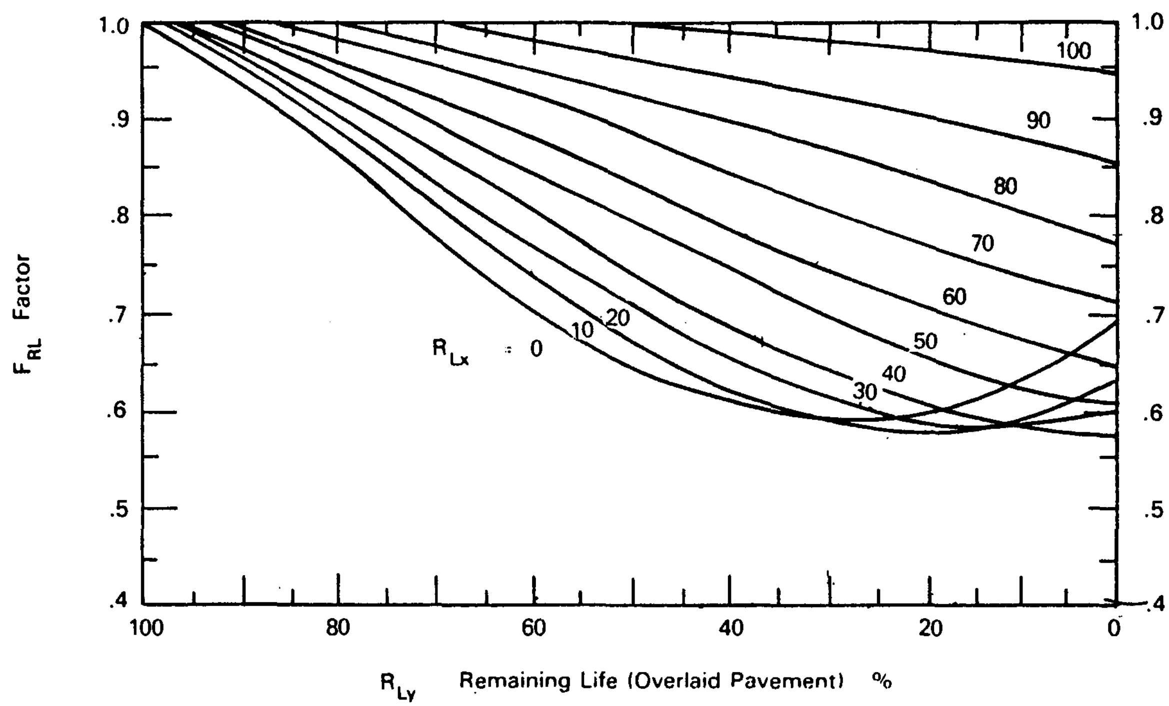


Figure 12. Remaining life factor as a function of the remaining life of the existing and overlaid pavements.(15)

bonded overlays of JPCP or JRCP were used to develop this procedure or program.

The TBCO1 program is based on the Finite Element Modeling (FEM) capabilities of the JSLAB program as well as results from NDT using a Dynaflect deflection testing device.[21] In the development of the program, the approach outlined below was used:

1. Using elastic theory and the Dynaflect deflections, the in situ material properties of the CRCP pavement are backcalculated for the midslab condition and at cracks.

2. The *k*-value on top of the base is determined based on a relationship between the modulus of the base (E_2) and the modulus of the subgrade (E_3). However, this relationship was developed using the BISAR program, which is based on elastic layer theory.[22,23]

3. The midslab modulus value is refined and determined based on matching of the deflection beneath Sensor 1 (directly beneath the load).

4. Using the ratio of the concrete modulus, as calculated by elastic theory, and the adjusted concrete modulus (from the previous step), the modulus of the concrete at the cracks is adjusted.

5. The adjusted moduli at the midslab, "hard elements", and the cracks, "soft elements", are used to model the material properties in the JSLAB program, which runs as a subroutine of TBCO1. The Dynaflect deflection basin is matched by increasing or decreasing the "soft elements" until the calculated deflection basin using the program overlaps the measured deflection basin from deflection testing.

6. The width of the soft elements is plotted versus the maximum deflection to determine the crack "zone of influence."

7. The crack modeling scheme, as applied to South Loop 610, showed the following:

   The crack could be modeled using soft elements (reduction of modulus values) and the reduction was approximately 53 percent.

   The influence of the transverse cracks extends to about 9 in (229 mm) on either side of the crack (1.5 ft [0.45 m] total).

All of these findings are built into the TBCO1 computer program.

Since no field data from Jointed Plain Concrete Pavement (JPCP) and Jointed Reinforced Concrete Pavement (JRCP) were used to develop the procedure, no attempt was made to model the effect of transverse cracks on such pavements within TBCO1.

The procedure assumes various critical loading conditions for the different pavement types. The corner condition is considered critical for JPCP because this pavement type is typically unprotected at the corner. The edge loading condition is considered critical for JRCP, while loading between cracks is considered the critical condition for CRCP. The maximum tensile stress of the original pavement is calculated for the critical loading condition. The maximum tensile stress is then used in a fatigue analysis.

The fatigue equation built into the program was developed from original AASHO road test data for asphalt concrete overlays of CRCP.[24] However, no CRCP pavements were built at the AASHO road test. The procedure assumes that this equation is applicable for use with rigid overlays. The number of 18-kip (80 kN) Equivalent Single Axle Loads (ESAL) is determined based on a failure criterion of 50 ft (15 m) of cracking per 1000 ft^2 (93 m^2) of pavement. Using the forecasted traffic, the designer may increase or decrease the thickness or modulus (to model the use of various material types) of the overlay to attain the required level of traffic. Thus, a final design thickness is obtained through an iterative approach.

TBCO1 Drawbacks

There are several problems with this method of overlay design; the following points out some of them. The use of the Dynaflect to measure deflections on heavily loaded concrete pavements is considered a questionable practice by a number of researchers.[25,26,27,28] It has been shown that the response of paving materials to a load, as well as the pavement system as a whole, is nonlinear. The use of the Dynaflect to determine the in situ properties may lead to erroneous results, since it induces very light loads on the pavement, and is being used to predict the effect of heavy loads.

Because of the assumptions of elastic theory, the backcalculation of rigid pavement materials properties based on midslab deflections has been questioned by several researchers.[6,17,18] Although the modulus as calculated at the midslab is adjusted by finite element analysis, the adjustment is made on one sensor only. The entire deflection basin has been shown to be very important in the determination of the in-situ materials properties.(See references 25,26,27,28) Furthermore, the use of elastic theory at cracks violates several basic assumptions of the theory. This violation of theory is noted, as is the difficulty in matching the deflection basins (using elastic theory) at the cracks. It is true that the "soft element" modulus values are adjusted by a ratio of the modulus as calculated using elastic theory and the modulus adjusted though finite element. It is assumed that this ratio is linear; however, it has not been shown that it is linear.

A relationship is developed between E_2, E_3, and the *k*-value, again using elastic theory. This ignores the fact the *k*-value is not an intrinsic soil property, as it depends not only on the load and load radius, but the stiffness of the overlaying layers.[29,30] It is assumed that the use of elastic theory in the determination of the *k*-value is acceptable, and then that *k*-value is used without further alteration throughout the procedure.

The extension of the design procedure to JPCP and JRCP is questionable. Since the procedure was developed for a single CRCP bonded overlay under a given traffic loading and exposed to a specific set of climatic conditions, its widespread applicability, even to CRCP, is limited.

No guidance is given to aid the designer in the determination of the in-situ properties of the existing pavement. These properties play a large role in the determination of the tensile stress and, therefore, the final overlay thickness.

The fatigue equation was developed for asphalt overlays of CRCP. There are numerous and widely accepted differences between asphalt concrete and portland cement concrete as overlay materials. The use of this fatigue equation for bonded concrete overlays of CRC pavements is highly questionable, as is the extension of this equation for use on JRCP and JPCP.

It is acknowledged in the report that the procedure was developed based on very limited data. The validity of the extrapolation of the crack modeling scheme (53% reduction of modulus at the cracks and a 1.5-ft [0.46 m] area of influence) to other CRCP's is debatable. In fact, in the "*Implementation Statement*" of the report, use of the procedure is qualified by stating that the procedure is applicable only to situations similar to those on South Loop 610 in Houston, Texas.

Performance Factors not Incorporated into the TBCO1 Model

While the report incorporates a discussion of factors known to have an effect on the performance of bonded overlays, no means of considering these factors in the design process is presented. These factors are discussed in the following paragraphs.

Horizontal movement occurs in the original pavement and in the overlay pavement. This movement can contribute to the occurrence of reflection cracking. Guidelines are given for the measurement of the strains in the original concrete and the determination of applicable properties of the overlay concrete. Vertical movement, as caused by changes in temperature or moisture, is also discussed. However, there are no design inputs presented which consider either of these factors.

Factors which affect the bonding conditions are presented separately. Shear at the slab interface can be applied by transverse loading. This was found to be very low, ranging from 2 to 10 psi (13.8 to 69.0 kPa). While a temperature gradient would add to this stress, it was not possible to calculate the added stress. Horizontal shear forces may also be applied to the overlay by braking tires or snow removal equipment. These were calculated and, while higher than the stress induced by an applied loading, were still fairly low. Bond strengths were then calculated from cores retrieved from the IH 610 overlay project in Houston. An overall average bond strength of 225 psi (1.6 MPa) was calculated, which gave a factor of safety of 2.4 when divided by the maximum applied stress (not including volume change stresses). A factor of safety of 3.0 was recommended to assure proper bonding. Again, these principles are explained in the report, but are not incorporated into the design procedure.

Drying shrinkage has been shown to be an important factor in the loss of bond at the corner of thin bonded concrete overlays.[31] The effects of drying shrinkage were noted as having an effect on the life of the overlay. However, they were not incorporated into the analysis of debonding.

Federal Aviation Administration (FAA)

The most widely used design procedures for airport pavements are discussed in *Design of Overlays for Rigid Airport Pavements*.[32] The general approach presented is to predict the performance of an overlaid pavement using a mechanistic approach and elastic layer theory. The analysis incorporates the fatigue damage present in the original pavement prior to placement of the overlay, the continued deterioration of the original pavement once the overlay is constructed and loadings continue, and loss of load transfer at the overlay's joints. The design method is an iterative process in which an overlay thickness is selected and the structural condition at the end of the design traffic is projected. If the projected structural condition is either unacceptable or too high, the thickness is modified and the calculation is repeated. The method is based on test data developed by the Corps of Engineers for unbonded and partially bonded slabs. One set of test data are available for a fully bonded overlay.

The method as described is not applicable to the design of thin, bonded concrete overlays. Only one bonded overlay was part of the database used to develop the procedure, and it consisted of an 11-in (279 mm) overlay bonded to a 17-in (432 mm) original concrete pavement. These two slabs had different load transfer mechanisms (dowels and aggregate interlock) and the overlay exhibited cracking and spalling over the dowel bars almost as soon as it was loaded. This pavement failed long before the application of its design loadings. Because an adequate load transfer construction joint cannot physically be built, the Corps of Engineers and most agencies limit fully bonded concrete overlays for airfield pavements to between 2 in and 5 in thick (51 and 127 mm), and their use is restricted to the correction of surficial smoothness or deterioration.

The method does not incorporate pavement damage due to non-structural causes, such as D-cracking or pumping. These must be considered elsewhere. Also, this method is developed for very heavy applied loads, and it is not clear how this would relate to highway loadings, which are typically lower in magnitude, but much higher in frequency.

Construction Technology Laboratories/Portland Cement Association Procedure (CTL/PCA)

The Final Report for Project HR-288 conducted for the Iowa DOT, *Field Evaluation of Bonded Concrete Overlays*, analyzes five bonded overlay pavement sections in Iowa.[33] In addition to a visual condition survey, material properties were obtained, and the pavements were instrumented to obtain load related strain and deflection measurements, and temperature related pavement movements. A finite element computer program, JSLAB, was used to obtain theoretical responses, which were compared to the measured responses.

This method is a modification of the thickness deficiency approach. It was developed using critical tensile stress calculations made with the use of JSLAB. Design charts were then developed to aid in thickness determination.

The design procedure recommends an evaluation and characterization of the existing pavement. A pavement condition survey is recommended to determine the type, quantity, and severity of distress for the entire length of the project. Based on the results of the visual survey, NDT may be performed. If the condition survey indicates the existence of, or potential for, load related distress, then NDT should be performed to determine the existence or extent of the problem. The material properties, such as strengths of the paving layers, should be backcalculated using the NDT data. For bonded overlays, a materials testing program should be implemented to evaluate the engineering properties of the existing materials. Since it is impractical to obtain beam samples from an in-place concrete pavement, it is recommended that cores be obtained. Split tensile tests should be performed on the cores and correlated to the flexural strength (modulus of rupture) and the modulus of elasticity of the material.

This design procedure is based on the assumption that the combination of the base slab and the overlay is structurally equivalent to a new full-depth concrete pavement. The applicable design assumptions are limited to the following conditions:

1. The modulus of elasticity (E) of the full-depth concrete pavement is either 4,000,000 psi or 5,000,000 psi (27,580 or 34,475 MPa).

2. The flexural strength of the full-depth concrete pavement is between 600 psi and 650 psi (4.14 and 4.48 MPa).

3. The value of the constant D is between 6000 and 7000 in the relationship $E_c = Df_r$.

4. The flexural strength of the existing concrete is between 425 psi and 575 psi (2.93 and 3.96 MPa).

The design equation is

$$t_o = t_t - t_e \tag{4}$$

Where:

t_o = Overlay thickness required, in
t_t = Total thickness of the existing slab and bonded overlay from the design charts, in
t_e = Thickness of the existing slab after surface preparation, in.

Figure 13 shows the design nomograph to be used for the design of thin, bonded concrete overlays. The nomograph is based on the thickness of a new slab for the design traffic level and the flexural strength of the material.

There is no correction factor applied to the existing slab thickness, as this is probably compensated for in the difference in flexural strengths between the new and old pavements. Also, it is assumed that the original pavement is a good candidate for a thin-bonded concrete overlay. This means that the pavement is structurally sound and any areas of distress are repaired before placement of the overlay. However, the validity of this approach is not known, especially since the tight restrictions placed on its applicability limit its usefulness.

FHWA Equations

In a study of rehabilitation techniques for concrete pavements sponsored by the FHWA, researchers in Illinois studied bonded concrete overlays. A database was developed using pavement design information, traffic and environmental data, and condition surveys of several overlays. Physical test data such as coring and boring, NDT, and debonding were not collected. With the available data, tentative models were developed to predict transverse joint faulting and reflective cracking. Due to the limitations of the database, no models were developed for the occurrence of pumping, joint deterioration, serviceability, and debonding. It is stated, however, that the performance of bonded concrete overlays in terms of pumping, joint deterioration, and serviceability is thought to be similar to the standard concrete pavement sections modeled in the National Cooperative Highway Research Program (NCHRP) 1-19, *Concrete Pavement Evaluation Study (COPES)*.[35]

The transverse joint faulting and reflective cracking models were developed using nonlinear regression techniques. The faulting and reflective cracking models follow:

$$\begin{aligned} FAULT = {} & 0.0015897*ESAL^{0.233}[\,-10.942 - 30.657*BASE \\ & + 0.0005652*(FI + 1)^{2.299} + 33.322*(DIA + 1)^{-0.8447}] \end{aligned} \tag{5}$$

$R^2 = 0.54$
SEE= 0.02 in
n= 27

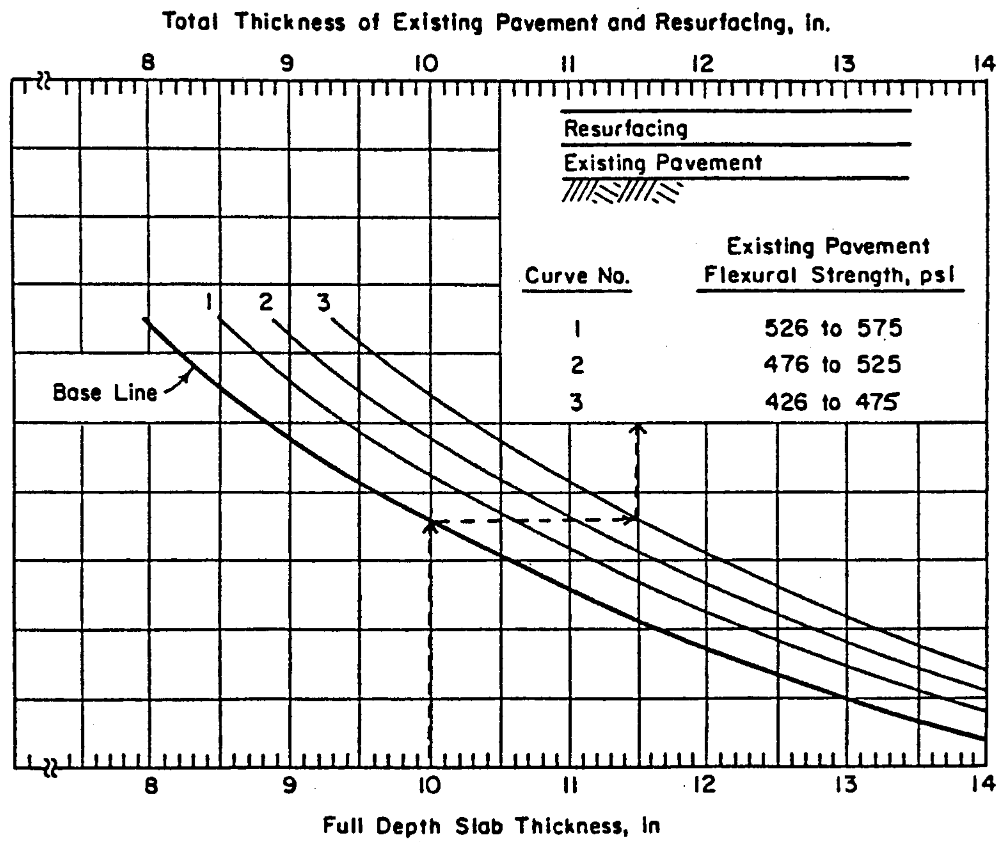


Figure 13. CTL/PCA design chart for bonded resurfacing.(34)

$$CRACK = 11.328*ESAL^{0.07546}[\ 21.426*(AGE*(FI + 1)/1000)^{0.66876}]$$

$$+ ESAL^{0.002}[378.5*INDEXM + 1257.1*INDEXH] \quad (6)$$

$R^2 = 0.75$
SEE = 326 ft/mile
n = 13

Where:

FAULT	=	Mean faulting across transverse joints, in
ESAL	=	Equivalent single-axle loads accumulated on the overlay in the outer lane, millions
BASE	=	0, if granular base type 1, if stabilized base type (cement, asphalt, etc.)
FI	=	Freezing Index, mean degree-days below freezing
DIA	=	Diameter of dowel bars in the original pavement, in (0, if no dowel bars exist in the original pavement) Note: the dowel spacing in all cases was 12 in
CRACK	=	Total length of medium and high severity deteriorated transverse reflection cracks, ft/mi
AGE	=	Time since construction of the overlay, years (Indicator of the number of temperature cycles affecting shrinkage and expansion of the concrete layers.)

INDEXM and INDEXH values:

If the original pavement was JPCP:

Existing Cracking[1]	INDEXM	INDEXH
Low - 0 to 100	0	0
Medium - 101 to 500	1	0
High - > 500	0	1

[1] Total linear feet per mile of medium- and high-severity transverse cracking on existing pavement prior to overlay placement [or 0 to 18.9, 19.0 to 94.7, and > 94.7 total linear meters per kilometer]. According to the authors, this is obtained by determining the amount of cracking in the overlay and assuming that all of the cracks are reflective, as well as an examination of preoverlay repair.

If the original pavement was JRCP:

Existing Cracking[1]	INDEXM	INDEXH
Low - 0 to 200	0	0
Medium - 201 to 1000	1	0
High - > 1000	0	1

All independent variables which were considered to have a meaningful and significant influence on the performance of bonded overlays were included in the equation. However, this determination was made subjectively rather than by statistical methods. Based on the specific data available, unbalanced analysis of variance or possibly an analysis of variance using incomplete blocking techniques could have been used to remove the subjectivity from the determination of the independent variables.(36,37,38)

The INDEXH and INDEXM terms of the reflective cracking equation were estimated based on the cracking exhibited in the overlay as well as examination of preoverlay repair. The INDEXH and INDEXM terms are variables which indicate the condition of the original pavement before the overlay was placed. The report documenting the research states that data on the condition of the existing pavements prior to overlay was not available for sections other than those at Clayton County (which were not used in the development of the reflective crack equation).(1) The total linear feet of transverse reflective cracking in the overlay was assumed to be the total amount of medium- and high-severity cracking present in the original pavement prior to overlay. Implicit in this is the assumption that all of the deteriorated cracks in the original pavement will reflect through the bonded surface within a relatively short period (1 to 2 years in several cases). This is based on the finding that "nearly all cracks in the base slab have reflected through the bonded overlays [at Clayton County] within a few years."(13) Another researcher found that many of the cracks in the original slab had reflected through to the surface within several years of service.(39) While it is generally accepted that the "joints and cracks in an existing concrete slab will eventually reflect through bonded concrete resurfacings, regardless of the type," very little study has actually been given to the rate of progression of reflective cracking of different severity levels.(40)

These models were developed using most of the same projects included in this study. It might therefore be expected that they would be applicable in predicting the performance of these sections. However, these equations were developed for the conditions at one point in time. Traffic is an input in both equations and age is an input in one of them. Since all of the sections have aged and experienced more traffic, use of these equations on these same pavements

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[1] Total linear feet per mile of medium- and high-severity transverse cracking on existing pavement prior to overlay placement [or 0 to 37.9, 38.0 to 189.4, and >198.4 total linear meters per kilometer]. According to the authors, this is obtained by determining the amount of cracking in the overlay and assuming that all of the cracks are reflective, as well as the examination of preoverlay repair.

would extend beyond the inference space of the models. Also, the field surveys did not include any physical testing. This is especially important for the evaluation of layer strengths, shear strength at the interface, and the amount of debonding observed. Since these are thought to have a large effect on the performance of bonded overlays, their absence in the development of the models is significant.

As with this study, the models are only as good as the data that were used. Because the model development is based on a relatively small sample size, the effect of any errors will be quite large. The calculated cumulative ESAL's at the time of the FHWA-sponsored survey differed from those calculated for the same time period in this study. While the ESAL calculations of four of the projects were within 40 percent of each other, the FHWA calculations were about 50 percent lower on two projects and 4 and 21 times lower on two others. This will have a large effect on the usefulness and applicability of the models.

3. COMPARISON OF PREDICTED VERSUS ACTUAL RESULTS

Design Methods

The methodology and basic assumptions for the comparison of the predicted versus actual design thicknesses for the AASHTO, TBCO1, FAA, and CTL/PCA is presented below.

1. The overlay is assumed to have a 20-year design life. The 20-year design traffic is calculated using the outer lane ESAL sustained by the overlay in the first, full year of trafficking. The growth rate is determined through examination of the growth several years prior to the overlay. The traffic information is summarized in table 27.

2. The overlay thickness is designed based on the design, traffic, and performance information available.

3. The actual overlay thickness is compared to the overlay thickness as determined by the design method.

AASHTO

This design procedure provides a number of options throughout the design process. The steps taken in the design procedure as well as the assumptions made in order to design the pavement are outlined below:

1. The thickness of a new pavement which will sustain the 20-year design traffic, SC_{yn}, was designed using the AASHTO new pavement design procedure.

2. The effective thickness, SC_{xeff}, was determined using the remaining life approach because many of the other methods require very specific information about the original pavement prior to overlay placement (such

Table 27. Traffic calculations to determine the 20-year design traffic.

SECTION ID	FIRST YEAR ESAL (millions)	GROWTH RATE	DESIGN ESAL (millions)
NY 6	0.23	5%	9.60
IA 1	1.32	4%	39.31
IA 2	0.62	4%	18.46
IA 3	0.075	2%	1.83
IA 4	0.11	5%	3.64
IA 5	0.10	7%	4.10
CA 13	0.66	4%	19.65
SD 1	0.18	9%	9.23
WY 1	0.91	7%	12.71
LA 1	0.29	10%	28.52

Note: Compound growth was used to determine $ESAL_{year\ 20}$.

as NDT, size of PCC fragments, or condition survey). In using this approach, the remaining life of the existing pavement, R_{Lx}, must be determined. One of the required inputs is serviceability of the original pavement at the time of overlay placement ($p_{t\ ORIG}$). The serviceability was estimated based on existing distress in the overlay, documentation provided in reports, and engineering judgement. Figure 10 was used to determine the reduction factor, C_x. This factor is multiplied by the existing slab thickness to determine SC_{xeff}.

3. The remaining life factor, F_{RL}, was determined based on R_{Lx} (determined in step 2) and the remaining life of the overlay after the terminal serviceability has been reached, R_{Ly}. The determination of R_{Ly} requires a serviceability at the time of failure of the overlay ($p_{t\ OL}$). This variable was assumed based on the functional classification of the pavement section following the recommendations presented in the Guide.

4. The overlay thickness was determined using equation 2.

The results of this analysis are presented in table 28. The thicknesses which were designed using the AASHTO procedure are substantially higher than the actual as-built thicknesses. The reason for this discrepancy may be found in an examination of several of the input variables used by the AASHTO procedure.

Reliability

The AASHTO procedure is quite sensitive to the selection of the level of reliability. Increasing or decreasing the reliability level by as little as 5 percent can increase or decrease the thickness by several inches. For this analysis, the reliability factor was chosen based on functional classification of the pavement section following the recommendations given in the *Guide*. For Interstate pavements, a reliability factor of 95 percent was used for the analysis. For rural and urban pavements, a reliability factor of 80-90 percent was used, based on traffic volume and functional classification.

Overall Standard Deviation

The overall standard deviation is an input parameter which accounts for the variability of the traffic estimation. The *Guide* recommends a value of 0.25 for traffic estimates without traffic prediction error and 0.35 for traffic estimates with traffic prediction error. For this analysis, a value of 0.35 was used for all pavement sections.

Traffic

The procedure is highly sensitive to the projected traffic. The design traffic was estimated for all sections as previously outlined and the results are shown in table 27.

Table 28. Comparison of the actual overlay design thickness with the thickness designed using the AASHTO design method.

SECTION ID	AS-BUILT THICKNESS (in)	DESIGN THICKNESS[1] (in)	DESIGN THICKNESS[2] (in)
NY 6	3.0	9.0	6.1
IA 1	4.0	9.2	6.8
IA 2	3.0	6.8	4.8
IA 3	2.0 - 5.0	8.4	6.2
IA 4	3.0	7.1	4.3
IA 5	2.0	6.6	4.2
CA 13	3.0	11.1	8.4
SD 1	3.0	9.9	7.3
WY 1	3.0	8.0	6.4
LA 1[3]	4.0	11.0	7.9

Note: The serviceability at the time of data collection was estimated for IA2, IA3, and IA4 based on the distress information and engineering judgment.

[1] Design thickness using the F_{RL} term.

[2] Design thickness excluding the F_{RL} term.

[3] The 1987 survey was performed at the wrong site and therefore did not include the bonded overlay.

$p_{t\ ORIG}$

The serviceability of the original pavement at the time of overlay is an important quantity in determining R_{Lx} which, in turn, has a large effect on the determination of the effective structural capacity of the original slab and F_{RL}. Since the serviceability of the original pavement prior to construction of the overlay was unknown, it was estimated from the available information.

$p_{t\ OL}$

The terminal serviceability of the overlay was chosen based on the functional classification of the roadway and the current condition of the pavement section. For Interstate pavement sections, a $p_{t\ OL}$ of 2.5 was assumed. For urban and rural roads, a terminal serviceability of 2.0 was assumed. This variable has a large effect on the determination of the R_{Ly} value.

F_{RL}

The F_{RL} value is a function of the R_{Lx} and the R_{Ly} values. It has a large impact on the overlay thickness. For this analysis, a number of the pavement sections fell in the range of R_{Ly} between 0 and 30 and R_{Lx} between 0 and 40. This is the portion of the graph that has been questioned by researchers.[19] The determination of the F_{RL} value in this range is questionable. In this analysis, the curves for R_{Lx} were extended at the minimum value to determine the F_{RL} value used for analysis.

It is believed that all of these factors contribute to the difference between the in-place thickness and the thickness as designed with the AASHTO procedure. It should be noted that since the use of this procedure in this instance requires that many assumptions about the inputs be made, agreement between the predicted and actual values is very unlikely.

According to the AASHTO procedure, all of the pavements should have been built significantly thicker. The last column in table 28 shows the overlay thickness determined without using the F_{RL} term. The results appear more realistic without this term. This is effectively what is recommended in reference 19: use $F_{RL} = 1.0$.

The design traffic, traffic sustained by the pavement at the time of the data collection, and the serviceability at the time of data collection are presented in table 29, as is the percent of the design ESAL's consumed and the percent of the serviceable life consumed. The only pavements which were close to their terminal serviceabilities were IA 4 and IA 5. Although serviceability ratings were not obtained on the IA 3 sections, these sections are near or beyond their terminal serviceability. These three projects are among the oldest bonded overlays in the country and the preoverlay repair was minimal. The need for thicker overlays in these cases may be warranted. However, the thicknesses actually recommended through use of the AASHTO design method appear excessive.

Table 29. Current levels of serviceability and traffic.

SECTION ID	DESIGN ESAL (millions)	1987-88 ESAL (millions)	1987-88 PSR	% DESIGN ESAL CONSUMED	%PSR CONSUMED
NY 6	9.60	2.54	3.2	26.4	65.0
IA 1	39.31	6.31	4.2	16.1	15.0
IA 2	18.46	7.93	3.7[1]	43.0	40.0
IA 3	1.83	1.03	---	56.3	---
IA 4	3.64	1.31	2.4	36.3	84.0
IA 5	4.10	1.32	2.4	32.2	84.0
CA 13	19.65	3.09	4.2	15.7	15.0
SD 1	9.23	0.71	4.2	7.7	12.0
WY 1	12.71	1.50	4.2	11.8	15.0
LA 1[2]	28.52	2.09	---	7.3	---

Note: The initial serviceability of the overlay was assumed to be 4.5 and the terminal serviceability was assumed to be 2.5 for Interstate pavements and 2.0 for urban and rural pavements.

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[1] Present Serviceability Rating of the inner lane.

[2] The 1987 survey was performed at the wrong site and therefore did not include the bonded overlay.

TBCO1

Several attempts were made to execute the TBCO1 program. The FHWA-supplied, FORTRAN source code was developed and compiled on a mainframe computer. In the conversion of the program for use on a microcomputer, several problems were encountered. While the program did compile, in trying to execute the program, several errors appeared. For example, the sample input file would not execute. Attempts were made to discover the source of the errors; however, due to the complexity of the program, the problem was not uncovered. The mainframe computer required to run a working version of this program was not available and the conversion of the program to a PC was beyond the scope of this project.

FAA

The design procedure documented in reference 32 is for unbonded and partially bonded overlays only. The document states the following concerning fully bonded overlays:

A fully bonded overlay and base slab are essentially a monolithic structure. The layered elastic analysis can account for differences in modulus values and flexural strengths between the overlay and the base pavement as well as for previous traffic fatigue damage on the base pavement. However, the value of this analysis ability for a fully bonded overlay is moot since an adequate load transfer construction joint cannot physically be built. For this reason, the Corps of Engineers and most other agencies require fully bonded airfield overlays to be between two and five inches thick, and their use is restricted to correction of surface smoothness or deterioration.

CTL/PCA

The first step in this design procedure is to determine the new concrete pavement thickness required to support the design traffic. The PCAPAV program was executed using the design traffic shown in table 27 and assuming a 20-year pavement design life.[41] The overlay thickness required to support the design traffic for each section as well as the actual in-place thicknesses are shown in table 30.

None of the sections fell within the required flexural strength range of 425 psi to 575 psi (2.9 to 4.0 MPa). All of the sections, except for those at Clayton County, Iowa showed substantially higher flexural strengths. The highest flexural strength range on the curve was used in all of the cases where the flexural strength exceeded the range. This will lead to a more conservative, thicker design than may actually be required. The lowest flexural strength range was used for the Clayton County sections, in which case the overlay will be underdesigned.

The overlay thicknesses presented in table 30 appear very reasonable. All of the overlay thicknesses fall within 1 to 2 in (25 to 51 mm) of the actual in-place overlay thickness. It is important to point out a very important basic assumption

Table 30. Comparison of the actual overlay design thickness with the thickness designed using the CTL/PCA design procedure.

SECTION ID	DESIGN OVERLAY THICKNESS (in)	AS-BUILT OVERLAY THICKNESS (in)
NY 6	2.5	3.0
IA 1	2.0	4.0
IA 2	2.0	3.0
IA 3	4.5	2.0 - 5.0
IA 4	3.0	3.0
IA 5	1.0	2.0
CA 13	4.0	3.0
SD 1	4.0	3.0
WY 1	4.0	3.0
LA 1[1]	2.5	4.0

[1] The 1987 survey was performed at the wrong site and therefore did not include the bonded overlay.

of the CTL/PCA procedure: the procedure assumes that a bonded overlay will be placed on structurally sound concrete pavements. Several of the experimental sections (WY 1, IA 3, IA 4, and IA 5) which are included in this study do not meet this fundamental assumption. Interestingly, these are the pavements that exhibit the most distress. Overall, it appears that the CTL/PCA method provides a good indication of the required overlay thickness, for pavements that meet the structural soundness requirement.

It is difficult to draw conclusions about the accuracy of the procedure at this time. All of the pavements which meet the structural soundness requirement are relatively new pavements. From the amount of distress exhibited by those pavements that meet the structural requirement, the procedure appears to be reasonable.

Predictive Models

In order to determine the accuracy of the predictive models developed for the FHWA under a previous project, the following steps were taken:

1. The faulting and reflective cracking as predicted by the FHWA equations were calculated for each of the sections.

2. The predicted faulting and reflective cracking were compared to the actual, field-measured distresses though the use of a paired *t*-test and though scattergrams.

Faulting Model

The inputs to the faulting equation are ESAL, BASE, FI, and DIA. The exact inputs for each section were retrieved from the database and are illustrated in table 31. The Statistical Analysis System (SAS) was used to compare the actual, field-measured faulting to the faulting as predicted by the FHWA equation.[42] The paired *t*-test was conducted to examine the statistical similarity of the data sets. The paired *t*-test assumes the following methodology:

1. For each section, the absolute value of the difference between the measured faulting (field data = FAULT) and the predicted faulting (FHWA Equation = UIFAULT) is calculated as shown below:

   $$\text{DIFFAULT} = \text{ABS}[\text{FAULT} - \text{UIFAULT}] \tag{7}$$

   Note: If the FHWA equations exactly predict the measured faulting, then DIFFAULT will equal 0.0 for every section.

   Table 32 shows the values of FAULT, UIFAULT, and DIFFAULT.

2. The mean of the DIFFAULT (d = ΣDIFFAULT/Number of observations) values for all sections is calculated. The hypothesis to be tested is: d = 0.0. This hypothesis assumes that the mean difference of the faulting

Table 31. Data Required for the FHWA thin bonded overlay equations.

SECTION ID	FI	DIA	AGE	ESAL	INDEXM	INDEXH	BASE
NY 6	990	1.25	6	2.54	1	0	0
IA 1	625	1.25	4	6.31	0	0	0
IA 2	688	1.25	9	7.93	0	0	0
IA 3-1	875	0.00	11	1.03	0	1	0
IA 3-2	875	0.00	11	1.03	0	1	0
IA 3-3	875	0.00	11	1.03	0	1	0
IA 3-4	875	0.00	11	1.03	0	1	0
IA 3-5	875	0.00	11	1.03	0	1	0
IA 3-6	875	0.00	11	1.03	0	1	0
IA 3-7	875	0.00	11	1.03	0	1	0
IA 4	875	0.00	10	1.31	0	1	0
IA 5	868	0.00	12	1.32	1	0	0
CA 13	1000	0.00	3	3.09	0	0	1
SD 1	1000	0.00	3	0.71	0	0	0
WY 1	750	0.00	4	1.50	1	0	0
LA 1[1]							

[1] The 1987 survey was performed at the wrong site and therefore did not include the bonded overlay.

values is 0.0 or, in other words, that the field-collected faulting data set is statistically the same data set as the faulting data set generated by the FHWA equation.

3. The one-sample t-statistic is calculated using the following:

$$t = d/SE_d \tag{8}$$

Where:

t = t-statistic calculated from data
d = Mean of DIFFAULT values
SE_d = Standard error of the mean = $s_d/(n)^{0.5}$
n = Number of observations
s_d = Standard deviation of DIFFAULT values
= $[\Sigma(\text{DIFFAULT} - d)/(n - 1)]^{0.5}$

4. The calculated t-statistic (t_{calc}) is compared to a tabulated t-statistic (t_{table}) for a given confidence level. If $t_{calc} > t_{table}$, then the two samples are statistically not the same sample.

Table 32 presents the results of this analysis for the faulting equation. The value of t_{calc} is calculated as 4.760 and the value of t_{table} for a 90 percent confidence level and 15 observations is 2.602. Since 4.760 > 2.602, the two samples are not statistically the same sample. Therefore, the FHWA equations do not accurately predict the field measured faulting for the sections in this project.

The data were also plotted on a scattergram, as shown in figure 14. This figure graphically illustrates the actual, field-measured data (FAULT) versus the predicted faulting (UIFAULT). If the FHWA equations exactly predicted the measured faulting, all of the data would fall on the line of equality (45°). It appears that the faulting model predicts significantly less faulting than the pavement experiences when subjected to the calculated traffic level.

Reflective Cracking Model

The results of the paired t-test for the reflection cracking model are shown in table 33. The value of t_{calc} is 5.005 and the value of t_{table} for a 90 percent confidence level and 8 observations is 2.896. Since 5.005 > 2.896, the two sample data sets are not statistically the same data set. Furthermore, the FHWA equations do not accurately predict the amount of reflection cracking observed for the data collected under this study.

The data are plotted on a scattergram, shown in figure 15. This graph illustrates the inability of the model to predict the reflective cracking observed in this study. From the data available, the reflective cracking equation over-predicts the amount of reflective cracking observed for the calculated traffic level. The cracks at IA 1 were routed and sealed between the time of the FHWA survey and the survey conducted for this study. Many of the cracks that were counted as

Table 32. Measured faulting versus predicted faulting using FHWA equations.

SECTION ID	ACTUAL FAULTING, IN	PREDICTED FAULTING, IN	DIFFAULT, IN
NY 6	0.07	0.09	0.02
IA 1	0.02	0.04	0.02
IA 2	0.10	0.05	0.05
IA 3-1	0.07	0.05	0.02
IA 3-2	0.08	0.05	0.03
IA 3-3	0.16	0.05	0.11
IA 3-4	0.11	0.05	0.06
IA 3-5	0.17	0.05	0.12
IA 3-6	0.11	0.05	0.06
IA 3-7	0.06	0.05	0.01
IA 4	0.07	0.06	0.01
IA 5	0.12	0.06	0.06
CA 13	0.00	0.09	0.09
SD 1	0.03	0.07	0.04
WY 1	0.04	0.04	0.00
LA 1[1]			
		ΣDIFFAULT =	0.68000

Analysis Variable : DIFFAULT

No. Observations	Mean	Std. Error.	t_{calc}	prob>|T|
15	0.06	0.0095	4.76000	0.0003

[1] The 1987 survey was performed at the wrong site and therefore did not include the bonded overlay.

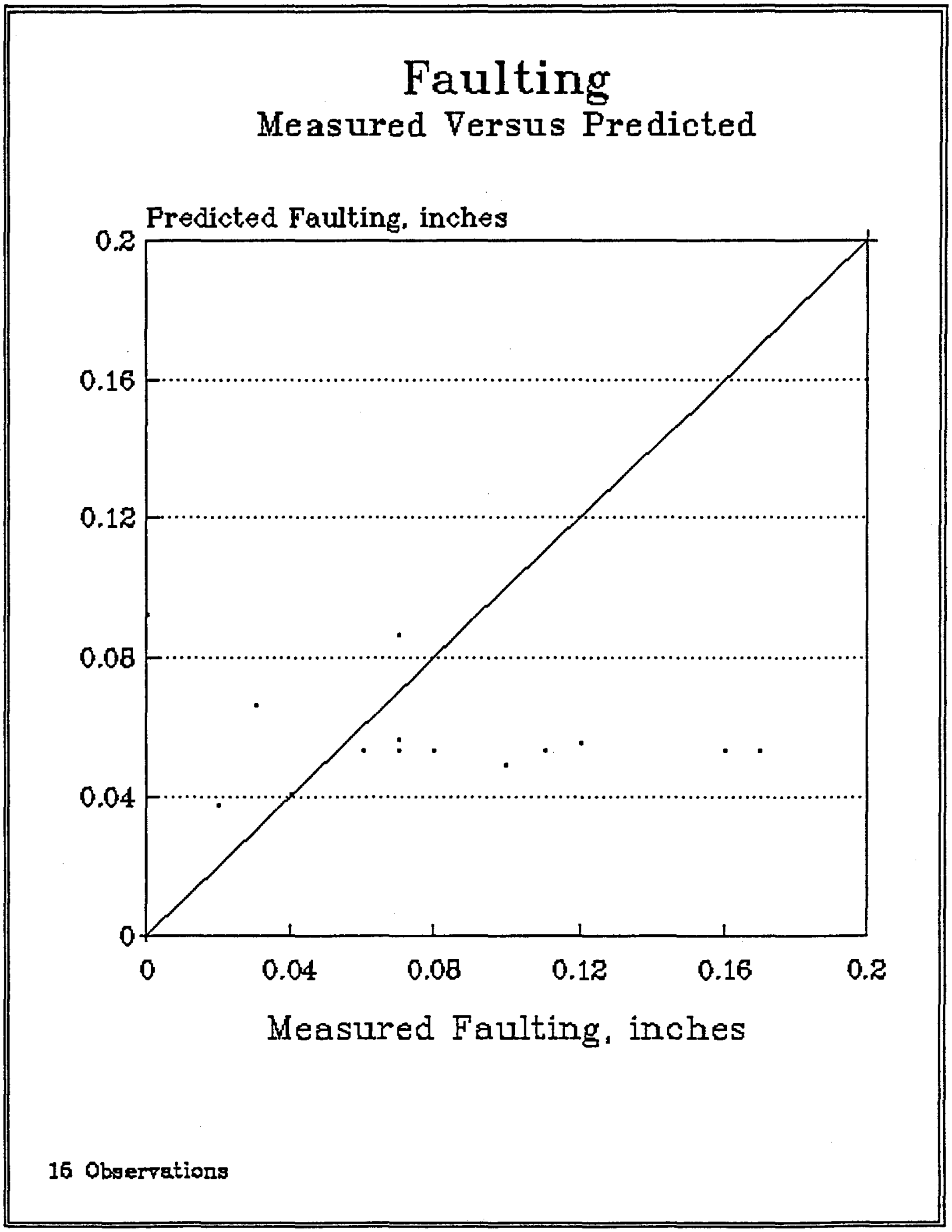

Figure 14. Actual field measured faulting versus faulting as predicted by the FHWA faulting model.

Table 33. Measured cracking versus predicted cracking using FHWA equations.

SECTION ID	ACTUAL CRACKING, FT/MI	PREDICTED CRACKING, FT/MI	DIFCRACK, FT/MI
NY 6	1237	240	997
IA 1	515	0	515
IA 2	961	72	889
IA 4	2315	720	1595
IA 5	2447	2952	505
CA 13	551	0	551
SD 1	493	0	493
WY 1	1780	504	1276
LA 1[1]			

ΣDIFCRACK =6826

Analysis Variable : DIFCRACK

No. Observations	Mean	Std. Error.	t_{calc}	Prob>|T|
8	952	147	5.8089	0.0007

[1] The 1987 survey was performed at the wrong site and therefore did not include the bonded overlay.

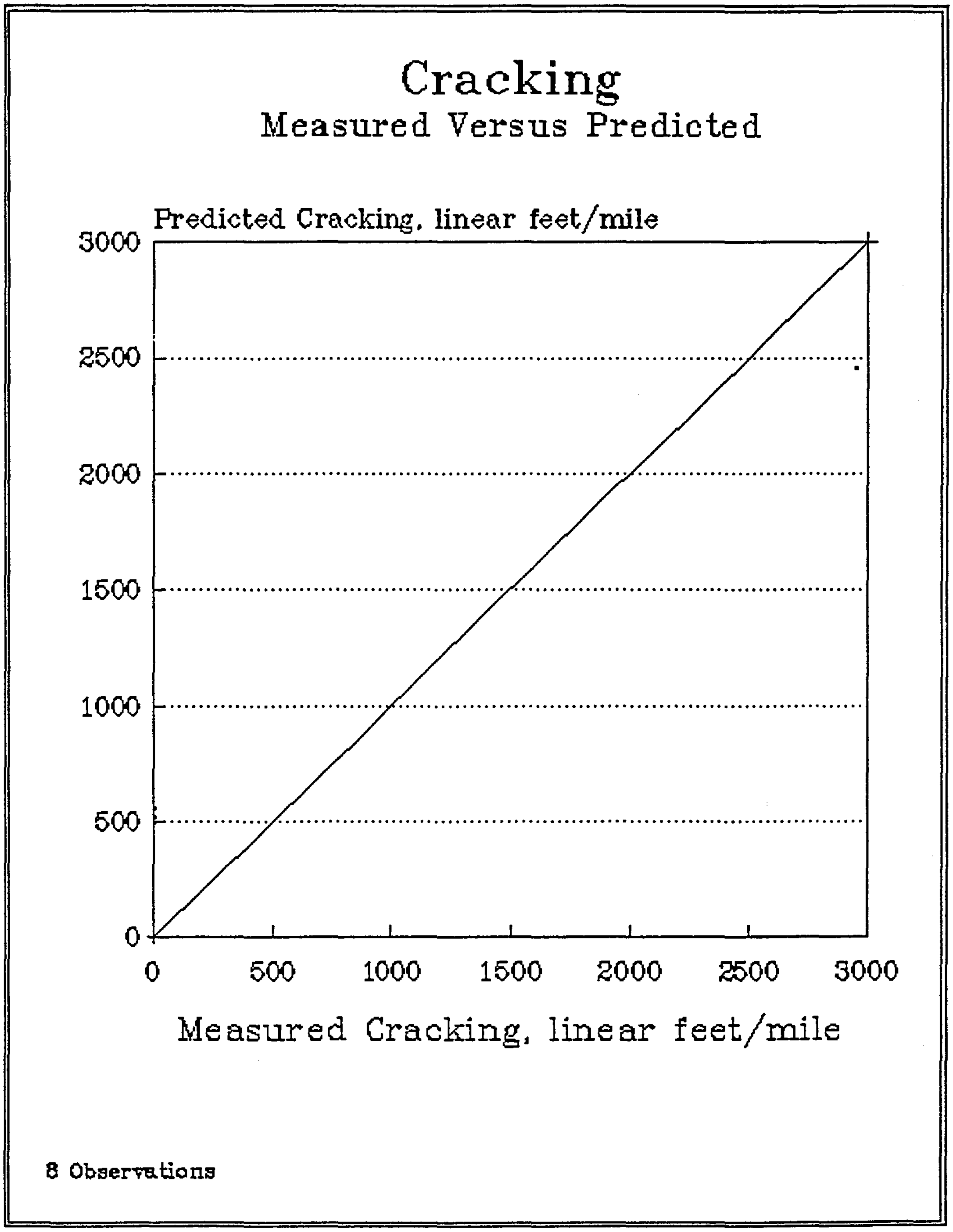

Figure 15. Actual field measured cracking versus cracking as predicted by the FHWA cracking model.

medium- and high-severity in 1985 were reduced to low-severity due to this maintenance, when surveyed in 1988. This may explain the over-prediction in the case of IA 1. The difference in the ESAL calculations may be the reason for the general trend of overprediction.

4. CONCLUSIONS

Of the design methods evaluated, the CTL/PCA method was the most theoretically sound. The thicknesses obtained from the procedure were reasonable, based on the amount of distress exhibited in the overlays. The method requires that the existing slab be structurally sound and show no signs of load-related distress. It appears that for pavements that meet these basic requirements, the CTL/PCA procedure yields thicknesses close to the design thickness of the overlay. The accuracy of the procedure cannot be determined at this time because most of the pavements which meet the structural requirements are relatively new pavements.

The AASHTO procedure yielded extremely thick overlays for all of the sections under study. The excessive thicknesses may have resulted from many of the assumptions built in to the procedure and made in order to use the procedure for this analysis. Care was taken to follow the values recommended in the *Guide*. However, it is important to point out that very different overlay thicknesses could result from the procedure if different values for several of the variables were chosen.

The FAA procedure was developed for the design of partially bonded and unbonded overlays only. The design of bonded overlays was discussed only briefly and as a surficial treatment.

Results were not obtained from the TBCO1 procedure due to difficulties in the execution of the program. The theoretical basis of the program is questionable, as a fatigue equation which was developed for asphalt overlays of CRCP is used to determine the fatigue in thin bonded overlays of original CRCP, JRCP, and JPCP. The entire procedure is derived from a single CRCP overlay of an original CRC pavement. Extrapolation to overlays of JRCP and JPCP is questionable. Also, a large portion of the development of the procedure is based on elastic layer theory for the analysis of rigid pavement sections.

The predictive equations developed under the earlier FHWA study do not accurately predict the reflective cracking and faulting of the thin bonded overlays in this study. The equations were developed from a database which consisted of many of the same pavement sections in this study. The inability of the equations to predict performance of the sections under study may be due to the fact that the ESAL's calculated by the researchers were significantly lower than the ESAL's calculated under this study.

CHAPTER 5 EVALUATION OF FACTORS AFFECTING OVERLAY PERFORMANCE

1. INTRODUCTION

A number of different factors are known to have an effect on the performance of bonded concrete overlays. This study evaluated 16 different designs at 10 locations in 6 States. A wide range of variables was included through these projects. While only the project in Clayton County, Iowa allows for a direct comparison of the effect of different variables, observations can be made across projects, as long as it is remembered that any conclusions made are confounded by the effects of the other factors. As an example, the original pavement's condition is probably a critical factor in assessing the performance of the overlay; however, no means is available to accurately and objectively compare the preoverlay conditions of the different sections. A subsequent task of this study will provide updated performance models based on a statistical analysis of the variables.

2. FACTORS AFFECTING PERFORMANCE

Age of Original Pavement

Typically, the bonded overlays in this study have been constructed on pavements that are fairly old and have carried a significant percentage of their design traffic. The age of the original pavement at the time of overlay construction of projects included in this survey ranged from 9 to 25 years. While consideration of age incorporates environmental effects, it does not take into consideration the varying amounts of traffic that these projects have carried.

Another way of evaluating the effect of the age of the original pavement on overlay performance is to consider the number of loadings applied to it. Highway pavements are designed to carry a certain number of applied 18-kip (80 kN) equivalent single axle loads (ESAL's) during their design lives. If the design ESAL's were known, the pavement's "traffic age" could be calculated as a ratio of the number of applied ESAL's to the intended design ESAL's. The pavements in this study have carried from 0.35 million ESAL's to 11.8 million ESAL's prior to construction of the overlays. However, design ESAL's were not known for most sections. Given the most common trend in underestimation of traffic loadings, it is probably safe to say that most of these projects had already carried their design traffic prior to construction of the overlay.

Table 34 summarizes some of the key data from the summary tables presented in appendix B. As can be seen, there is no trend that develops for either age or traffic of the original pavement that affects the performance of the overlay. It is interesting to note that 6 out of 10 projects as originally constructed did reach a typical design life of 20 years. As noted above, however, those that did not most likely had already carried their design traffic.

Table 34. Key summary data for bonded overlay projects (outer lane).

Project	Age at Overlay, yrs	ESAL's at Overlay, 10^6	Age of Overlay, yrs	ESAL's on Overlay, 10^6	L, M, & H Transverse Cracks on Overlay/mi	L, M, & H Longitudinal Cracks on Overlay, lin ft/mi	PSR
IA 3	9	0.35	11	1.03	---[1]	---[2]	N/A
IA 2[3]	13	0.81	9	1.39	162	0	3.7
WY 1	16	2.02	4	1.50	201	2693	4.2
IA 5	18	1.19	12	1.32	444	5935	2.4
IA 1	20	11.8	4	6.31	210	0	4.2
CA 13	20	5.90	3	3.09	245	1002	4.2
LA 1	22	2.69	6	2.09	N/A	N/A	N/A
IA 4[3]	24	0.18	10	0.15	70	5260	2.4
NY 6	24	3.55	6	2.54	172	20	3.2
SD 1	25	1.13	3	0.71	0	142	4.2

[1] 7 sections ranged from 36 to 726 cracks/mi. The average was 350 cracks/mi.

[2] 7 sections ranged from 309 to 10,560 lin ft/mi. The average was 6844 lin ft/mi.

[3] Data from inner lane.

Condition of Original Pavement

The condition of the original pavement, no matter how old it is or how much traffic it has carried, is a factor which has a major effect on the performance of the overlay. Three major distresses of a bonded concrete overlay, reflective cracking, joint spalling, and debonding, can all be affected by the extent and severity of the deterioration of the original pavement. Unfortunately, this is extremely difficult to quantify without detailed descriptions of the original pavements, including photographs and summaries of all of the work performed on the pavement prior to construction of the overlay. Available descriptions of the pavements suggest that they were all experiencing distresses requiring rehabilitation. However, no acceptable means of quantifying those distresses or differentiating among the distresses on the different sections is available. An indirect means of taking into account the condition of the original slab is to evaluate the extent of preoverlay repairs, of which somewhat more is known.

Preoverlay Repairs

In bonded overlay design, the combination of the original pavement and the bonded overlay are intended to act as a single, monolithic layer. Because of the monolithic nature of the bonded overlay pavements, the original pavement must be in very good condition. If it is not, many of the distresses, and particularly working transverse and longitudinal cracking, will rapidly reflect through to the overlay slab. The presence of these distresses will then accelerate failure of the pavement. Implicit in this theory is the concept that if bonded concrete overlays were placed on pavements exhibiting no distress, there is no reason to believe that the composite pavement, if properly constructed, would not last reasonably long. However, all of these overlays were constructed over pavements in need of various levels of rehabilitation.

The most common types of repair needed were partial-depth repairs (PDR's) and full-depth repairs (FDR's). Some of these repairs were performed immediately prior to the construction of the overlay. Others were part of an ongoing rehabilitation process that eventually culminated in the decision to place the overlay. Partial-depth repairs were placed at 7 of the 10 projects and full-depth repairs were present on 6 of the 10 projects.

Another common repair found on these pavements was the construction of pressure relief joints (PRJ's). These were found at 6 of the 10 different project locations. Pressure relief joints were constructed where the pavement was experiencing blowups. Many of the projects with PRJ's had additional PRJ's added when the overlay was constructed.

Several of these pavements displayed significant transverse or longitudinal cracking. At 5 of the project locations, some type of metal reinforcement was placed over the cracks of the existing pavement prior to placement of the overlay. The purpose of this reinforcement is to reduce movement at the crack and to keep the crack from reflecting through the overlay, or from reflecting through with the same severity.

The project in Wyoming reported subsealing with a cement-pozzolan mixture to fill voids beneath the pavement prior to construction of the overlay. The presence of asphalt cement beneath the corner core on I-80 in Avoca suggests that this project also had some subsealing performed.

It should be clear from the performance data that placement of an overlay does not solve problems in the existing pavement. The onset of reflective cracking may be slow, but the bonding surveys show a disturbingly large amount of apparent debonding associated with the cracks, once they do reflect through. Also, despite the great extent of pre-overlay repair on some of these projects, it is clear that all of the existing distresses can never be economically removed or repaired. An examination of the core log in appendix C reveals that cores recovered from several of the projects with extensive preoverlay repairs showed further deterioration. Specifically, the corner cores from I-81 in Syracuse, I-80 at Grinnell and Avoca, U.S. 20 in Waterloo, and U.S. 61 in Baton Rouge all showed that the deteriorated portions of the original pavement were not entirely removed. It is most likely that these distresses were present at the time of construction of the overlay, and that they have contributed to the deterioration of the overlay.

Age/ESAL's of Overlay

As in the discussion of the age and traffic carried by the original pavement, it is also interesting to consider the age and number of ESAL's carried by the overlays. The overlay projects included in this study ranged in age from 3 to 12 years old and had carried from 0.7 million ESAL's to almost 8 million ESAL's at the time of their surveys. Key indicators of their performance are summarized by age in table 35.

The Present Serviceability Rating (PSR) is an overall measure of pavement serviceability and thus may be a more useful tool to evaluate the effect of the overlay's age and performance. As table 35 shows, there is a decline in the PSR as the age of the overlay increases. The oldest overlay was 12 years old at the time of its survey and its overall serviceability is approaching a failed condition. Four of the other projects for which data are available had carried more traffic and had higher PSR's. Unfortunately, PSR's were not obtainable on several of the older projects, which makes drawing any conclusion more difficult. It must be remembered that other factors will have an impact on the performance besides time and the traffic placed on the overlay. These include the condition of the original pavement, the presence of materials-related distresses, and the degree of bond achieved.

Climatic Zone

Projects in this study were located in three environmental zones; the only area not represented was the Dry Non-Freeze zone. Examining the performance results by climatic zone is misleading, as there are many more projects in the Wet Freeze zone than in the other two. From the limited data available, the projects in the Wet Freeze zone also exhibited more distresses. The Dry Freeze zone had the best overlay performance, although there is only one project in this zone.

Table 35. Performance of bonded overlays relative to overlay age (outer lane).

Project	Age of Overlay, yrs	ESAL's on Overlay, 10^6	Transverse Cracks on Overlay/mi	Longitudinal Cracks on Overlay, lin ft/mi	PSR
CA 13	3	3.09	245	1002	4.2
SD 1	3	0.71	0	142	4.2
WY 1	4	1.50	201	2693	4.2
IA 1	4	6.31	210	0	4.2
NY 6	6	2.54	172	20	3.2
LA 1	6	2.09	N/A	N/A	N/A
IA 2[1]	9	1.39	162	0	3.7
IA 4	10	0.15	70	5260	2.4
IA 3	11	1.03	---[2]	---[3]	N/A
IA 5	12	1.32	444	5935	2.4

[1] Data is from the inner lane.

[2] 7 sections ranged from 36 to 725 cracks/mi. The average was 350 cracks/mi.

[3] 7 sections ranged from 309 to 10,560 lin ft/mi. The average was 6844 lin ft/mi.

Most of the original pavements exhibited D-cracking and other joint-related distresses requiring extensive repair work. These distresses can ultimately be traced back to climate and moisture problems. The distresses for which bonded overlays have typically been used as a rehabilitative measure do not occur in the milder, Dry Non-Freeze climate. Pavements located in a Dry Non-Freeze climate, however, would probably be ideal candidates for the placement of bonded overlays, due to the absence of moisture and freezing conditions and because of the less severe deterioration usually found on those pavements.

Temperature Range at the Time of Paving

It is a fairly widespread belief that the ambient temperature at the time of paving and during the curing period has an effect on the performance of the overlay. High temperatures, low humidity, high winds, and dry conditions all have an adverse effect on both the curing of the overlay and the drying of the bonding agent after it is applied and before the overlay is placed. The ability to achieve a good bond is probably inhibited if the overlay is placed on a drying grout, as that drying layer will act as a bond-breaker.

The ambient temperature during the curing of the overlay has a large effect on the stresses that the overlay experiences. Higher temperatures will induce tensile stresses on the top of the overlay as it expands, while the bottom of the overlay is cooler and may experience compressive stresses. Cooler temperatures will cause the opposite effect. These stresses are coupled with drying shrinkage stresses that are induced in the concrete as it cures.

The available data concerning the ambient temperature at the time of paving of the overlay are summarized in appendix B. These show a range from 41 °F to 86 °F (5 °C to 30 °C). A more thorough analysis of the effect of climatic conditions at the time of paving is necessary in order to draw any conclusions from this data.

Overlay Thickness

Bonded concrete overlays are typically constructed thinner than either unbonded or partially bonded concrete overlays. This study included projects ranging in thickness from 2 in to 5 in (51 mm to 127 mm). Construction of an overlay any thinner than 2 in (51 mm) is impractical and raises the question as to whether there does not exist a more appropriate rehabilitation alternative. While there is no limit to the thickness to which a bonded overlay can be constructed, when very thick bonded overlays are deemed necessary it is possible that the pavement is a candidate for an unbonded overlay or some other type of rehabilitation.

It is not possible to compare the performance of overlays of different thickness, as the performance is closely tied to the preexisting distresses. With the exception of the Clayton County (IA 3) and the Waterloo (IA 5) projects, the overlays were either 3 or 4 in thick (76 or 102 mm). It is not known how the thickness was designed for these sections, nor how the condition of the existing pavement was taken into account in the design process. As is noted earlier, if just the results from Clayton County are examined, it would appear that the thinner overlays performed better than the thicker overlays. However, the thinner overlays were placed in areas that did not exhibit as widespread or severe distresses as those with thicker overlays.

Surface Preparation

The development of a good bond between the original slab and the overlay concrete is dependent on the surface preparation of the existing pavement. Good surface preparation consists of removing all paint, oil, debris, laitance, and loose material. The goal is to leave a clean and rough surface to which the overlay can easily adhere. The process of surface preparation is typically divided into two steps, initial and final surface preparation. During the initial surface process, most of the pavement surface material is removed. This is typically less than 0.25 in (6 mm), although greater quantities were reported. This removal has been accomplished by many different means, including sandblasting, air blasting, water blasting, shot blasting, acid etching, and cold milling (also referred to as scarifying). The most commonly used methods of initial surface preparation found in this study were shot blasting and milling. A direct comparison of these two methods is not possible due to the confounding of these factors with many others.

After the initial preparation is completed, the pavement is ready to be overlaid. However, for construction or other reasons, there is usually a period of time which elapses before the overlay is placed. Therefore, a final surface preparation is needed in order to remove any accumulated contaminant or dust which might interfere with bonding. This final surface preparation is most often accomplished by sandblasting or air blasting, and is proceeded as rapidly as possible by placement of the overlay. Care is taken during the actual placement of the overlay to keep all construction vehicles and other possible sources of recontamination off of the prepared surface.

Type of Bonding Agent

One of the major concerns with bonded overlays is the loss of bond between the overlay and the original pavement. Bond is usually achieved through the use of a bonding agent applied to the prepared surface of the original pavement just prior to placement of the overlay. There are three major types of bonding agents used in the construction of bonded concrete overlays. They are water-cement grouts (or neat cement), sand-cement grouts, and epoxy adhesives. All of these were used on at least one project included in this study.

A bonding survey was carried out at each of the projects at approximately 20 percent of the joints and slabs. The area which was apparently debonded was mapped on the survey sheets and then debonded "areas" were manually calculated. Any attempt to group together the results from the projects constructed in different locations and under widely varying conditions will be misleading. Also, the results from the shorter sections are bound to be less accurate than those from the longer sections, for statistical reasons alone. Therefore, the grouping together of these results is for purposes of discussion only.

As table 36 shows, the cement grout performed the best of the three. While there were a high number of debonded joints with all three types, the cement grout appeared to provide better adhesion at the joint and cracks. There was also much less of the total area debonded on these projects.

Debonding in the wheelpaths was only a problem at Truckee (CA 13), where the epoxy grout was used. However, the damage to the original pavement which necessitated the overlay was brought about by excessive wear in the wheelpaths from tire traffic with chains. It is likely that the existence of the wheelpath debonding had more to do with this wear pattern than with the grout used.

The most prevalent bonding problem on all of the sections was at the joint corners. There are several possible reasons for this. Most of the pavements experienced joint-related distress prior to construction of the overlay. All of the deteriorated joints may not have been repaired. Also, deflections are higher at joints, which might contribute to the development of debonding. Some States reported difficulties applying the grout at the edges of the pavement, since the application was from the center of the pavement out toward either edge. Once the concrete is cured, debonding at the corners may occur through differential curling or warping of the overlay and the existing slab. Finally, it is possible that the debonding that was detected could be delamination in the underlying slab since this could not be differentiated during the debonding survey. It is not possible to determine how much of the debonding differential between types of bonding agents is due to the effect of some or all of these other factors.

Original Pavement Type

Three of the projects, the one in New York and the two on I-80 in Iowa, were constructed over jointed reinforced concrete pavement (JRCP). The rest were constructed over jointed plain concrete pavement (JPCP). The three JRCP pavements all had long-jointed original slabs and dowels, and all had required extensive joint rehabilitation prior to construction of the overlay, including construction of PRJ's. An examination of the summary tables shows that there is no significant difference in performance between these sections and those constructed on JPCP.

Overlay Pavement Type

There were also three sections constructed with reinforcement in the overlay on C-17 in Clayton County, Iowa. The outer lane on I-80 near Truckee, California also was reinforced. The performance results from these sections are inconclusive, as in the eastbound lane on C-17, the sections with reinforcement generally performed better than the sections of the same thickness without reinforcement, whereas in the westbound lane the opposite was true. Reinforcement in the overlay may help to slow down the rate at which cracks reflect through to the surface. However, the limited results available do not support this.

Table 36. Performance summary of bonding agents.

Type of Bonding Agent	Number of Projects	Percent Debonded Joint Corners	Percent[1] Debonded Crack Corners	Percent Debonded Wheelpath Area	Percent Total Slab Area Debonded
Cement-Sand	8	75	82 (7)	0	43
Cement	5	41	19 (2)	0	4
Epoxy	1	75	N/A	19	57

Note: Some of the apparent debonding shown in the table may be due to delamination in the underlying slab since the source of the actual debonding could not be identified.

[1] The number in parentheses is the number of projects with debonded crack corners.

Overlay Joints

There are several factors relative to the joints in the overlay that have an effect on the overlay's performance. First, the transverse joints in the overlay were matched with those of the original pavement on all of the projects. Because of the movement associated with transverse joints, this is considered essential. In most cases, this matching also included the joints introduced by the construction of full depth repairs. However, verification of the extent of match-up was beyond the scope of this study.

The transverse joints on all of the projects were sawed the full depth of the overlay, with the exception of IA 2 and IA 5. The decision to saw the transverse joints less than full-depth on those two projects may have resulted in the large amount of transverse cracking found on them.

Longitudinal joints were sawed on all of the projects where the two lanes were paved together, with the exception of IA 2, IA 3, and IA 5. None of the longitudinal joints, however, were sawed the full depth of the overlay. The two lanes of CA 13 were paved separately. Generally, it was found that the longitudinal joint of the original pavement reflected through the overlay on all of these sections. These sections also had more additional longitudinal cracking than the others. The depth of the longitudinal sawcut varied from 33 percent to 67 percent of the thickness of the overlay, but this did not appear to have as much of an effect on the longitudinal cracking as did whether or not there was a sawcut.

CHAPTER 6 SUMMARY AND CONCLUSIONS

1. INTRODUCTION

This study considered 16 different bonded concrete overlay projects at 10 locations throughout the country. A comprehensive field survey was conducted to obtain objective and comparable measures of the performance of these different projects. In addition, a variety of field testing was carried out, including FWD testing, coring and boring followed by material testing, and a bonding survey. With traffic data provided by the individual States, estimations were made of the number of ESAL's carried by these pavements to date. Background information about the original pavement's design, condition, and overlay construction was available from published reports and from additional data provided by the State highway agencies. Basic environmental data were also collected for each project for use in the analysis.

Several methods available for design and analysis of bonded concrete overlays were evaluated. The applicability of these methods was analyzed in terms of the conceptual validity of their approach and the types of values that were obtained from employing these design and analysis models. Where appropriate, these values were compared to those actually measured.

In light of the volume of data collected during this study, and also considering previous research, an attempt was made to analyze some of the factors that affect the performance of bonded concrete overlays. This attempt is somewhat impeded by the very limited number of projects available for analysis and the uniqueness of each project. Comparisons of the effect of one design variable among the different projects was usually confounded by the presence of several other unique variables.

The following conclusions are made based on the data obtained and the observations of the researchers. Suggestions are also made concerning the need for future research, especially where it is felt that more information is needed to draw conclusions.

2. CONCLUSIONS AND RECOMMENDATIONS

Condition of the Original Pavement

The condition of the original pavement plays a large role in the performance of bonded overlays. This has been fairly well documented in previous research.[13,39,43] There are several distresses in the original pavement which, if present and unrepaired, will inhibit good performance of the overlay. For example, joint spalls are a problem because they constitute areas of weakness in the original slab. When not totally removed or repaired, the overlay in an area of spalling will not have full support. Movement from loading or thermal or moisture gradients will result in deterioration of the overlay in the affected areas.

Most working cracks and joints will reflect through the overlay. This is because slab movement is associated with these discontinuities; placement of a bonded concrete overlay does little to change the effect of slab movement. On the I-25 project in Wyoming, one of the preoverlay repairs consisted of cleaning and resealing cracks in the original slab. Given the extent of cracking on the overlay, the repair apparently did not prevent those cracks from reflecting through to the overlay. While joints in the overlay can be sawed directly over those in the original slab in order to avoid the differential movement, no means exists of accurately matching random cracks. This step would be necessary, however, if it were desired to accommodate the anticipated movement in the overlay from cracks in the original slab.

Several of the projects in this study had experienced materials-related distresses in the original slab. This includes I-81 in Syracuse and all of the Iowa projects with the exception of Clayton County. The distresses manifested themselves as a deterioration of the concrete at the slab joints, where the pavement is exposed to the combination of moisture and cyclic freeze-thaw action. In addition to the spalling of the joints, this weakening of the joints may also contribute to the development of slab blow-ups. Where material-related distress is present, it typically occurs on all of the joints and must be removed.

The extent of the distresses dictates what amount of preoverlay repair is needed and, indirectly, whether the pavement is a viable candidate for a bonded overlay. A widespread materials problem at each joint can only be treated by full-depth repairs performed at every joint. Because problems such as D-cracking progress from the bottom of the slab upward, it is possible that a joint is deteriorated that does not manifest distress on the surface. It is probably not cost-effective to perform such extensive repairs. As the descriptions of the retrieved cores in appendix C show, continued deterioration was found in the corner cores retrieved from I-81 at Syracuse, I-80 at both Avoca and Grinnell, U.S. 20 at Waterloo, and possibly U.S. 61 at Baton Rouge, despite full-depth repairs made prior to construction of the overlay. This suggests that it is very difficult to entirely remove material-related distress from the original pavement and supports the conclusion that pavements with these types of problems should not be overlaid with bonded concrete.

Insufficient information exists about the condition of the original pavements of the projects included in this study. In most cases, the type and quantity of pre-overlay repair is known, but the condition of the pavement following that repair is not, especially as far as what distresses were <u>not</u> repaired. From a description of the repairs, it is clear that all of the pavements had experienced materials distresses, structural distresses, or a combination of both. All of the projects show a certain amount of cracking in the overlay. It would be highly desirable to know more about the existing condition of the slab prior to construction of the overlay. A detailed mapping of all distresses existing in the original pavement could be compared to maps made during regular post-overlay surveys to determine both the extent of reflection cracking and the rate at which it occurs. Deflection data from the pavement before it is overlaid would also provide a wealth of information for future use. It would help to identify sub-slab voids, determine load transfer efficiencies, and help to illustrate how the placement of the overlay reduces the effects of these conditions.

Preoverlay Repairs

Unless the pavement is in excellent condition prior to construction of the overlay, preoverlay repairs are essential for ensuring good performance of the bonded overlay. Preoverlay repairs performed for this project included partial-depth spall repair, full-depth repair to replace corner breaks or shattered slabs, reinforcement of cracks, deep milling of deteriorated areas, and the provision of subdrainage while the overlay is being placed. Cleaning and sealing joints was also noted. In some cases, pressure relief joints were constructed to accommodate excessive slab movement.

As is discussed above, if there exists deterioration in the original pavement, it must be repaired prior to construction of the overlay. While this improves the surface condition of the slab, it is not an improvement in the structural capacity of the original slab. Generally speaking, as long as all deterioration is removed, there would be no reason not to expect good performance from the overlay. The major problem arises with extensive working cracks in the existing slab and subsurface conditions such as voids or deterioration that are not identified. Problems associated with deteriorated load transfer devices, such as corrosion or socketing, must also be identified and addressed prior to construction of the overlay.

Some attempts have been made to address working cracks in the existing pavement by placing reinforcement over the crack prior to construction of the overlay. Placing reinforcement over cracks is not expected to prevent them from reflecting through the overlay, only to reduce the severity of the reflection crack in the overlay. It is almost assured that any working cracks in the original slab will reflect through in the overlay. Less is known about the rate of reflection of tight, nonspalled low-severity cracks.

Several studies have been conducted to examine and model the rate of reflection cracking on asphalt concrete overlays of rigid pavements.[44,45,46] The mechanism of cracking has been attributed to horizontal movement caused by temperature variation and vertical movement due to traffic loading. The propagation of cracks through a bonded concrete overlay are believed to be caused by these same mechanisms.

A good guideline to follow for preoverlay repairs is that if the distress is related to movement in the underlying pavement, it must be repaired or it will cause movement in the overlay. Further research is needed about the rate of crack reflection and the identification of the causes of cracks in the overlay that are not reflective.

Surface Preparation

Much attention has been given to the preparation of the surface prior to the placement of the bonding agent and overlay. While five different methods were used on the projects in this study, the most common ones were shot blasting and milling. Any surface preparation method that effectively removes all deterioration and foreign material that will inhibit development of a good bond is probably acceptable. A comparison of bonds developed on pavements prepared by these different methods was not possible, as many other factors confounded such an analysis. The only project which did not show a corner shear strength well above 300 psi (2.07 MPa) was on I-25 in Wyoming, where a retrieved core showed that there was no bond at all. This project was milled, sandblasted, and air blasted before the overlay was placed.

The method of surface preparation eventually selected should depend on the hardness of the aggregate, whether it is desired to remove surface defects, and how much material must be removed to get down to clean, uncontaminated concrete. This can be determined through examination of the pavement and, if necessary, trials with the equipment. Undoubtedly, currently available equipment and technologies will be modified and improved to better perform this job.

Paving Temperature/Climatic Conditions

The ambient conditions at the time of paving have an effect on the curing and bonding of the overlay. Concrete should cure slowly so that shrinkage stresses are minimized. Curing will be accelerated by elevated temperatures, winds, and low humidity. The importance of selecting a period when these effects are minimized must be emphasized, as rapid curing has been associated with cracking and debonding of the overlay, particularly at the corners. Undesirable climatic conditions may also accelerate the drying of the grout, which will then cause it to act as a bondbreaking layer rather than a bonding layer. More work needs to be done on what constitutes optimum climatic conditions for the placement of bonded concrete overlays. As an upper limit, it is recommended that placement of thin bonded overlays be prohibited if the surface temperature of the existing pavement exceeds 90 °F (32 °C). Paving should also be restricted if large temperature changes are likely to occur. These sudden temperature changes will greatly increase the chances of bonding failures.

Type of Bonding Agent

Three types of bonding agents were used on the projects in this study. They were a sand-cement grout, a water-cement grout, and an epoxy resin. The cement grout appeared to perform better than the sand-cement grout in terms of bonding. There was only one project which used the epoxy, on I-80 at Truckee. This project had severe debonding problems in the wheelpaths, where the original pavement had been worn by chains. Because of the increased cost of this type of bonding agent and its poor performance in this instance, there is no compelling reason to recommend its use. This result has been supported in at least one other case, where it was found that cement grout performed better than epoxy resin grout.[47] Epoxy has, however, successfully been used by at least one state (Louisiana) to adhere areas of the overlay that were identified as prematurely debonded.

Bonding

Perhaps the single most important consideration in the construction of a bonded concrete overlay is the development of a good bond between the existing slab and the overlay. It is this bond that ensures that the overlay and original slab act monolithically. This is essential because the thin concrete overlay does not provide sufficient structure to carry heavy loadings alone.

Debonding was tested for with a very subjective sounding method that was not capable of differentiating between debonding at the layer interfaces and delamination or deterioration at some greater depth in the existing pavement. Actual debonding, therefore, may be less than that reported in this study.

Debonding, when present and a problem, was most commonly found at joint corners and crack corners. Satisfactory bond, commonly defined as a bond strength greater than 200 psi (1.38 MPa), was developed after construction for all of the projects, with the possible exception of I-25 near Douglas. This can be stated because many of the projects were actually checked for delamination after construction and delaminated areas that resulted from construction problems were corrected. Also, an examination of the shear test results supports the contention that an adequate bond was present at all mid-slab locations and all of the corner locations, with the exception of the shear results from U.S. 20 in Waterloo and I-25 near Douglas.

Therefore, some factor or combination of factors develops which contributes to the progression of debonding. The location of the debonding at joints and cracks suggests that horizontal and vertical movement of the pavement are at least contributing to debonding. Vertical movement can result from heavy loadings and from the development of voids beneath the original slab. Deflection data obtained prior to the placement of the overlay would have helped to establish the presence of voids and high deflection at the corners. In the absence of that data, the presence or absence of voids can only be surmised. The recovery of subsealing material at the bottom of cores on I-80 at Avoca and I-25 at Douglas confirms that voids were a problem on at least two projects. These were also the two projects with the lowest shear strengths.

The debonding at cracks may also be caused by horizontal slab movement. For the crack to reflect through to the surface, there must be some associated movement in the original slab. The horizontal movement associated with the expansion and contraction of the slabs may be causing the debonding problem at the joints and cracks.

If the overlay is debonded at the joint, then the slab and overlay are not acting monolithically. The opening of the joint due to temperature variation will be different for the original pavement and the overlay, thus inducing additional horizontal stresses at the interface. Also, if a working crack exists in the original slab, the effects of drying shrinkage above the crack will be similar to a joint. As the overlay cures, a large horizontal shrinkage stress develops at the corners of the joints and cracks which results in debonding of the overlay.

Performance Expectations of Bonded Overlays

The overlay projects included in this study ranged in age from 3 years to 12 years at the time of their 1987-88 survey. While the serviceabilities were generally good and the surface distresses were not severe in most cases, the extent of debonding is a cause for concern. Some of these projects appear to be nearing failure, based on the accumulation of surficial distresses and the apparent widespread debonding. One State's report on their bonded overlay assigned an average life of 7 years to asphalt overlays and 15 years to concrete overlays.[11] On that basis, their life cycle costs were nearly equal. At this time, however, no agency is on record as stating that these overlays should last 15 or 20 years in order to be cost-effective rehabilitation strategies; it is doubtful that these project can all last that long. However, that is a fairly typical life that would be expected from a high-type concrete resurfacing.

Design Procedures

With the exception of the Clayton County projects, all of the overlays were constructed either a nominal 3 or 4 in (76 or 102 mm) thick. This lack of variation in overlay thickness exists despite the fact that the projects have carried loads ranging from 0.7 million ESAL's to almost 8 million ESAL's. It is not clear what design procedures were used on each of the projects in this study, but it is suspected that because of the uniformity of as-built overlay thicknesses, given the wide range of conditions and traffic, no formal procedure was followed.

An evaluation of several design procedures in chapter 4 showed technical or procedural problems with each one. Further analysis of designed overlays is highly desirable, to directly compare whether the design procedures produce a thickness that can provide the required service for the intended period. Also, design procedures are needed to clearly identify which projects are and are not candidates for this type of rehabilitation.

Design and Construction Guidelines

Based on the experience gained through the evaluation of the 16 projects in this study, the guidelines proposed in the earlier FHWA study for the design and construction of bonded concrete overlays have been modified.[1] These modified guidelines are presented in appendix A.

General Conclusions

Overall, bonded concrete overlays have enjoyed mixed success. Some of the projects have been able to carry high volumes of ESAL's and maintain good serviceability. These pavements exhibit a large amount of deterioration, as did the original pavements. In chapter 1 it was suggested that bonded overlays are intended either to improve the structural capacity of a pavement or to repair surficial, nonstructural defects. However, it does not seem possible that any of the overlay projects studied were constructed for either of these reasons. Previous researchers found that overlaying a pavement with higher remaining life gives better performance of the overlay.[47] There has not been an extensive amount of research into the cost-effectiveness of this type of rehabilitation, especially in terms of how much life is required for this to be a viable rehabilitation technique compared to the next alternative. It was also stated that construction of a bonded concrete overlay holds the promise of extended service life, increased structural capacity, and lower life cycle costs compared to other overlay techniques. This remains to be demonstrated by future projects designed and constructed to increase structural capacity or repair surficial defects.

Recently, much attention has been paid to the concept of fast-track concrete paving.[48] The use of fast-track concrete paving offers the potential of early-opening times for bonded overlays. This concept will aid in meeting one of the major constraints that has been traditionally been a part of concrete paving operations.

APPENDIX A DESIGN AND CONSTRUCTION GUIDELINES FOR BONDED CONCRETE OVERLAYS

1. INTRODUCTION

This section provides a comprehensive summary of recommended design and construction guidelines for bonded concrete overlays. The information presented herein is a synthesis of information obtained from past research studies, from published design and construction procedures, and from field performance results. The guidelines are applicable for jointed concrete overlays of existing jointed plain or jointed reinforced concrete pavements.

2. NEED FOR BONDED CONCRETE OVERLAYS

When properly used, bonded concrete overlays can offer increased performance and service life over conventional overlays. It is recommended that they be used to rehabilitate existing concrete pavements that, while in generally good overall condition, are in need of structural and/or rideability improvements. Bonded concrete overlays can be an effective rehabilitation strategy if the following conditions are met:

- Truck traffic loadings have or are expected to greatly increase, resulting in the structural deficiency of the existing pavement.

- The existing concrete slab does not suffer from serious durability problems, such as D-cracking or reactive aggregates.

- The pavement does not include an extensive amount of working transverse or longitudinal cracks or joint spalling. If any of these distresses do exist, preoverlay repair, in the form of full- or partial-depth concrete repairs, must be performed. However, bonded overlays may not be cost effective if more than 5 percent of the surface area must be repaired to correct these deficiencies.

- Thick asphalt concrete overlays are not feasible because limitations exist on the thickness of the proposed overlay, for such reasons as overhead clearances, bridges, restricted widening, etc.

- Traffic control to construct the bonded concrete overlay is possible. However, fast-track bonded overlays are successfully addressing this issue.

It should be reiterated that bonded concrete overlays may be particularly cost effective for pavements which need a major structural improvement. They are an appropriate response to the need of increased structural capacity <u>prior</u> to the development of major structural distresses. This situation is becoming more commonly encountered as truck traffic volumes and weights are increasing at a rapid rate on many highways. If not considered in some way, truck traffic in excess of the design traffic will cause significant fatigue damage and crack deterioration in the existing pavement.

Corner and edge loading conditions are critical elements in the design of JPCP and JRCP. Excessive stresses produced by loading at these locations will cause premature failure of the concrete slab. The placement of a bonded concrete overlay will reduce the magnitude of the stresses and thus prolong the life of the pavement. For example, figure 16 shows the reduction in edge stress and corner deflection under an 18-kip (80 kN) axle load for varying thicknesses of bonded concrete overlays. Figure 17 compares the reduction in edge stresses provided by bonded concrete overlays and equivalent thicknesses of asphalt concrete. Edge load stresses are approximately 35 percent lower for equivalent thicknesses of bonded concrete overlay.

Structural distress indicators which may denote that structural improvements are needed include corner breaks, transverse cracking for JPCP, deteriorated transverse cracks for JRCP, and shattered slabs. The presence of any of these distresses may be an indication that the structural capacity of the pavement is being exceeded. However, a pavement exhibiting a substantial amount of these distresses is probably beyond the point where bonded overlays would be a feasible rehabilitation alternative.

Bonded concrete overlays can also be placed to provide improved pavement rideability. This is considered an important factor from the aspects of both comfort and safety. Major indicators of the need for rideability improvements include severe concrete scaling from concrete mix deficiencies or poor surface finishing, wheelpath polishing, and "rutting" from studded tire or chain wear.

3. EFFECTIVENESS OF BONDED CONCRETE OVERLAYS

In a concrete pavement, the critical tensile stress is typically located along the outside edge at the bottom of the existing slab under an edge wheel load. If the tensile stresses become excessive or are frequently produced, slab cracking develops. The addition of a few inches of portland cement concrete bonded to the existing slab thickness forms a thicker monolithic slab that results in a large reduction in the tensile stresses. Reduced tensile stresses mean reduced fatigue damage per load application, which consequently prolongs pavement life. The end result is an increase in the number of load applications that the monolithic slab can carry before the onset of slab cracking.

The thicker monolithic slab also results in reduced deflections at slab corners and cracks, if debonding can be prevented. This may reduce the potential for faulting, pumping, and crack deterioration.

The projected life of bonded concrete overlays is an item of major concern. Among the factors that influence the effectiveness of bonded concrete overlays are:

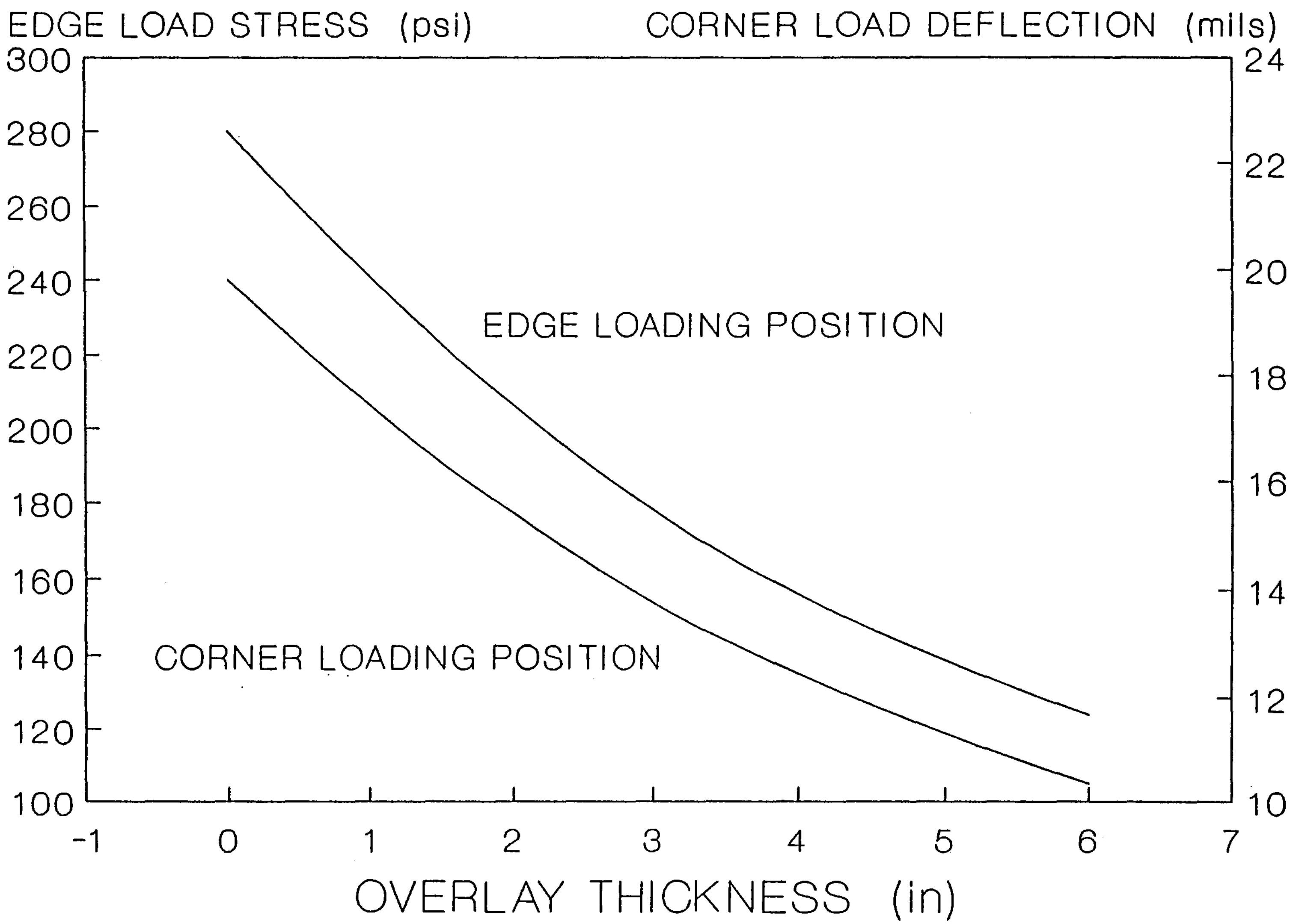

Figure 16. Edge load stress and corner deflection versus overlay thickness on a standard 9 in concrete pavement. (Developed by ILLI-SLAB modelling of a 9 kip wheel load [tire pressure = 75 psi] positioned at free edge and corner.) (1)

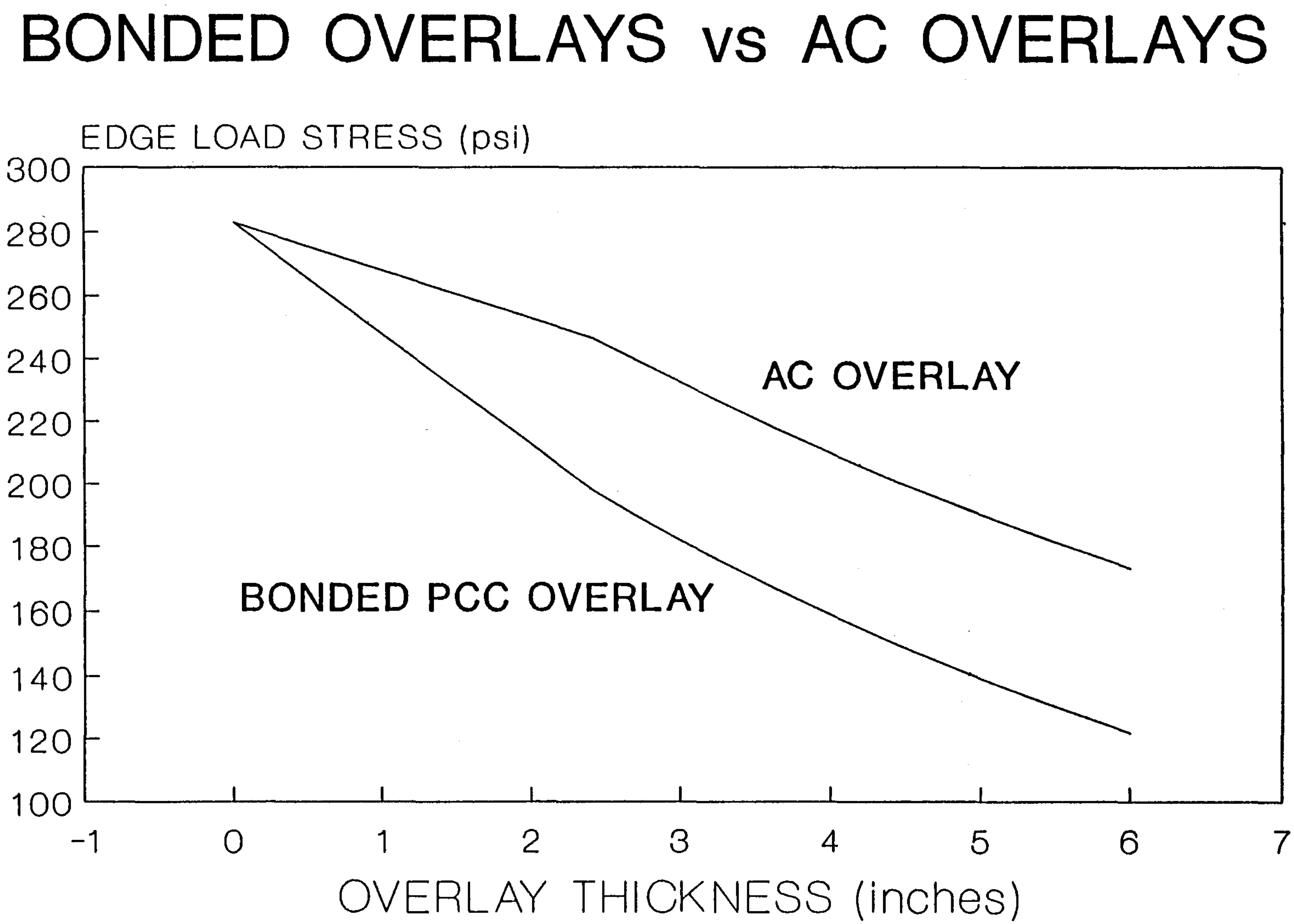

Figure 17. Comparison of free edge stress for bonded concrete overlay, to equivalent thickness of asphalt concrete overlay (Computed by ILLI-SLAB finite element program for 12x20 ft. slab with 18-kip single axle load at edge.) (1)

- The soundness of the existing concrete slab (absence of materials durability distress, such as D-cracking).

- The amount of working cracks in the existing concrete slab (and the extent of full-depth repair of these cracks).

- The amount of joint spalling, both at the top and bottom of the slab joints (and the extent of full-depth repair of these areas).

- The degree of bond achieved between the overlay and the existing slab at construction and its variation over time.

- Thickness of the bonded concrete overlay.

- The project truck traffic.

- The drainage characteristics of the pavement.

When these criteria are properly considered and addressed in the design and construction of a bonded concrete overlay, the bonded concrete overlay should meet its projected design life.

4. LIMITATIONS OF BONDED CONCRETE OVERLAYS

The past performance of bonded concrete overlays has ranged from poor to very good. Their poor performance can generally be linked to a failure to adequately consider one or more of the guidelines presented in section 8. Several of the reasons for the poor performance of bonded concrete overlays are described below.

The performance of bonded concrete overlays has been poor when applied to existing pavements with significant amounts of working transverse and longitudinal cracks. This is because it is expected that all working cracks in the existing pavement will reflect through the overlay, usually within the first 2 years of service. When a bonded overlay is used on an existing pavement where there are many working cracks and they are not repaired, the overlay will deteriorate at an accelerated rate. Accelerated development of debonding is also associated with the presence of these cracks. Full-depth dowelled repairs of the cracks (where the joint edges become joints in the overlay) will minimize this problem.

As mentioned in the preceding paragraph, debonding of the concrete overlay from the existing slab at joint corners and working cracks is another serious problem. This phenomenon has been observed on many projects. The causes of debonding have not entirely been determined. It is suspected that one cause is a high range of thermal changes during construction (e.g., hot days and cool nights), or very hot weather construction that results in shrinkage of the overlay at the edges and corners. This causes large horizontal shear stresses at the interface of the concrete overlay and the existing pavement.

Another mechanism suspected of causing debonding of concrete overlays is the constantly repeated horizontal shear stress at the interface at corners of joints and cracks caused by thermal gradients and drying shrinkage. If this stress is larger than the bond strength achieved between the two layers, a progressive debonding occurs. Finally, the presence of D-cracking or other weakened joint conditions in the existing slab may also be a cause of debonding.

It is essential that a good bond is achieved between the concrete overlay and the exiting slab. Improved bonding techniques or materials are clearly needed to ensure long-term bonding.

Other deterioration problems frequently observed on bonded concrete overlays are secondary joint cracking and shrinkage cracking. These problems are the result of poor construction techniques and can be prevented by following recommended procedures in this guide.

Bonded concrete overlays should not be placed on existing pavements with extensive D-cracking or reactive aggregate distress. These distresses may hamper the ability to achieve a permanent bond, and are also very likely to reflect through the bonded overlay.

5. CONCURRENT WORK

When applying a bonded concrete overlay, a variety of preoverlay repairs and other needed rehabilitation is normally required. This is particularly true for pavements exhibiting a fair amount of transverse or longitudinal working cracks. For the overlay to reach its design life, it is imperative that all major structural distresses be repaired prior to overlay.

6. PREOVERLAY REPAIRS

Preoverlay repairs are necessary to bring the existing pavement to an acceptable condition. The final pavement condition must be adequate for a bonded overlay to perform satisfactorily over its design life. Major work items to consider include:

- Full-depth repairs or slab replacements of deteriorated joints and cracks. The use of reinforcement across working cracks does not appear to be very effective in preventing the reflection of the crack, although it <u>may</u> reduce the rate of deterioration of the crack. An alternative to full-depth repair of working cracks is retrofitting dowel bars across working transverse cracks and cross-stitching (with deformed rebars) at working longitudinal cracks.

- Load transfer restoration at faulting joints.

- Partial-depth spall repairs.

- Addition of subdrainage to reduce erosion beneath the slab and subsequent faulting.

- Resealing of existing joints, if they are open and significant incompressibles from the cleaning, grouting, or paving operation could infiltrate.

- Slab undersealing to fill voids and slab jacking of high severity settlements.

7. OTHER CONCURRENT REHABILITATION WORK

In addition to the preoverlay repairs mentioned above, it is also necessary to consider the shoulders of the proposed concrete overlay. Specifically, the major item of concern is that the shoulders match the geometrics of the new concrete overlay surface. If the existing asphalt concrete (AC) shoulders are in good condition, a simple AC overlay to match the thickness of the bonded concrete overlay is cost effective. If the existing shoulders are deteriorated, it may be cheaper to remove some of the deteriorated material, rework and compact the surface, and place either an asphalt concrete shoulder or a concrete shoulder that is tied into the mainline pavement.

8. DESIGN AND CONSTRUCTION OF BONDED CONCRETE OVERLAYS

This section deals with general guidelines pertaining to the actual design and construction of the concrete overlay. Each step in the design process is outlined and described.

Design Procedures

The design procedures employed in determining the thickness of the bonded concrete overlay should include the following components.

- Visual condition survey of the existing pavement to identify all deterioration. Those areas that require full- and partial-depth repairs should be clearly identified and marked for later repair.

- Structural evaluation, using deflection testing, to determine:

  (a) Elastic modulus of the existing slab and an effective *k*-value of the base and subgrade.

  (b) Transverse joint load transfer.

- Erodibility analysis, using deflection testing, to detect loss of support beneath the slab corners.

- Destructive coring through the slab centers and through the joints of the existing pavement to estimate strength of the concrete and to determine the extent of underlying deterioration, respectively.

- Analysis of past fatigue damage from past traffic to provide an estimate of the remaining life of the existing slab.

- Prediction of realistic future traffic loadings for the future overlay design period.

- An evaluation of the extent and severity of any concrete durability problems, such as D-cracking or reactive aggregate, that may exist on the pavement.

The use of heavy load deflection testing, such as a Falling Weight Deflectometer, is recommended to accurately determine the structural integrity of the pavement. The resulting slab and foundation modulus values and joint load transfer efficiencies are extremely valuable to the structural evaluation of the pavement and the design of the bonded concrete overlay.

For overall constructability, a minimum thickness of 3 in (76 mm) is recommended for most bonded concrete overlay applications. Bonded overlays have been successfully constructed as thick as 5 in (127 mm) on highway pavements and 10 in (254 mm) on airfield pavements.

It is recommended that the bonded concrete overlay thickness needed to meet future expected traffic be determined through a comprehensive design procedure. One example of a design procedure is that developed by the Construction Technology Laboratories.[34] This procedure utilizes the structural deficiency approach:

$$\text{Bonded Concrete Overlay Thickness} = H_{\text{New Slab}} - \text{Condition}*H_{\text{Existing Slab}}$$

The new slab thickness can be determined using any procedure that allows the direct consideration of future traffic loadings. The condition factor used in the procedure is usually assigned as 1.0, because it is strongly recommended that only structurally sound pavements, or pavements with repaired working cracks, be considered as candidates for bonded concrete overlays. However, there may be situations where it is desirable to use a value of less than 1.0. Examples of the application of the procedure to design are given in table 37. A detailed evaluation of available overlay design procedures is provided in chapter 4 of this report.

After the overlay thickness has been determined using a given design procedure, it is very important that the results be checked using another procedure. This added step is to ensure the "reasonableness" of the overlay thickness.

Table 37. Example bonded overlay thickness design.

EXISTING PAVEMENT:IA 1 I-80 Grinnell, Iowa

Existing Pavement:	Thickness -	10.0 in
Original Design:	k of Subgrade -	105 psi
Parameters:	Modulus of Rupture -	670 psi
	Joint Spacing -	76.5 ft
	Life -	20 years
	Traffic -	39,000,000 18-kip ESAL
	Reliability -	95 percent

Existing Condition: Age is 20 years.

Full depth repairs will be performed at all joints to correct the "D" cracking. Pressure relief joints will be installed.

Remaining Life Factor = 0.74

Current Traffic Level: 1.32 million ESAL's per year

Actual Growth Factor: 4 percent per year

BONDED OVERLAY DESIGN:

T_{new} for extension of Life 15 years: 9.85 in (Using AASHTO Design Guide for ESAL = 26.43 million - 95% reliability)

$$T_{OL} = T_{new} - T_{exist\ condition} = 14.5 - 9.2 * 0.77 = 7.7 \text{ in}$$

To Extend Pavement Life 15 Years: T_{OL} = 7.7 in

T_{new} for extension of Life 20 years: 10.50 in (Using AASHTO Design Guide for ESAL = 39 million - 95% reliability)

$$T_{OL} = T_{new} - T_{existing\ condition} = 16.0 - 9.2 * 0.74 = 9.2 \text{ in}$$

To Extend Pavement Life 20 Years: T_{OL} = 9.2 in

Materials for Construction

The portland cement concrete used for a bonded concrete overlay should ideally have a low water-cement ratio, a low slump, and still be workable enough for bonded overlay thickness design placement. The specifications for the portland cement concrete material will be governed by the specifying agency. Both Type I and Type III (high early strength for early opening to traffic) cement can be used. Aggregate materials should meet the gradation and durability requirements of the specifying agency. It is generally recommended that the maximum size of the coarse aggregate be limited to less than one-half of the overlay thickness for these thin surfacings. For thicker overlays, the gradation limits for conventional mixes may be appropriate.

The mix design specifications that are used for new construction should be used in specifying the concrete mix for the overlay. Where Type III cement is employed, adjustments in the mix design and precautions in the curing process are mandatory.

When workability of the concrete is a concern, a water reducing admixture meeting the requirements of ASTM can be specified. If this alternative is chosen, the mix design should be adjusted according to the requirements of the specifying agency. Admixtures such as calcium chloride that will increase the strength gain of a Type I cement concrete mix can be used where particular concern exists for reopening the lanes to traffic in shorter periods of time. However, attention must be paid to achieve highly efficient curing to prevent or at least reduce debonding and shrinkage cracking.

A portland cement-based grout material to bond the overlay to the existing prepared slab surface has proven to be capable of generating the required bond strength between the original slab and the overlay. Past data has indicated that a direct shear strength of 200 psi (1.4 MPa) is sufficient to withstand expected shearing forces. Satisfactory results to meet this criteria have been obtained with grouts using sand-cement-water or simply cement-water. In table 38 below is direct shear test data from Iowa that indicates expected bond strengths for various application techniques and grout materials.

The tests were performed on cores that were prepared with a simulated scarification and cured for 23 days under moist conditions. Results are averaged for three cores tested at each interface condition.

Based on these results and past experience, the water/cement ratio should not exceed 0.62 (7 gallons [26.5 l] of water per bag of cement). The grout should have a creamy consistency. Care should be used with the grout material. The grout should be stir-agitated from the time of mixing to the time of application. Grout should not be applied to the pavement and left uncovered long enough for it to start to set. This time is dependent on the ambient conditions at the time of paving. If possible, all grout material should be used within 90 minutes from the time of mixing. Most recent projects have utilized only cement-water bonding material and it has apparently worked fairly well.

Table 38. Iowa shear strength data.

TREATMENT AT INTERFACE	BONDING STRENGTH, psi
1:1 Sand-Cement-Water	450 [3.1 MPa]
Water-Cement at 0.70 lb/lb Brushed on	640 [4.4 MPa]
Water-Cement at 0.70 lb/lb Sprayed on	490 [3.4 MPa]
Water-Cement at 0.62 lb/lb Brushed on	390 [2.7 MPa]
Water-Cement at 0.62 lb/lb Sprayed on	610 [4.2 MPa]

Recent coring of several projects in Iowa after several years of service showed the interface bond strength at mid-slab to range from 664 to 1000 psi (4.6 to 6.9 MPa). However, cores taken from slab corners have often shown that no bond or a reduced bond exists. Soundings taken from practically all projects, ranging in age from 3 to 12 years, has indicated that anywhere from 1 to nearly 100 percent of joint corners or crack corners debonded to some extent (about 2 ft^2 [0.19 m^2] at a typical corner). Therefore, as progressive debonding occurs, there must be a debonding mechanism acting at the corners of joints and cracks.

Liquid epoxy resins have recently been used for the bonding material. Research at Caltrans laboratories has shown much better bonding than achieved with grout.(49) Caltrans experience has been with pressure spraying application of low viscosity epoxy material over the entire surface of the existing slabs. However, one project that is 4 years old has shown significant debonding at both the joint and crack corners.

The best results have been obtained when the liquid epoxy resin is applied at a high coverage rate, about 40 ft^2 per gallon (0.98 m^2/l). This will ensure a relatively thick (0.1-in [3 mm]) layer, which helps promote a uniform bond. The material used by Caltrans had a pot life of about 1 hour and retained its adhesive properties for 10 to 15 minutes after application.

It is recommended that lab testing of the compatibility of the bonding material and existing pavement be performed for verification of the bonding procedures and properties. Testing should be performed on a portion of the existing slab that was removed from the field, prepared as closely to the field procedures as possible, and overlaid with similar material and under climatic conditions as expected in the field. The same material and surface removal/cleaning methods should be used, as well as curing at conditions equivalent to the weather conditions expected at the time of placement. After curing, cores should be cut and tested for shear strength across the bonding interface. Special attention should be paid to the amount of time allowed before the grout loses its adhesive qualities and starts to act as a bondbreaker.

Some recent results from three States have indicated good initial bond strengths without the use of a bonding agent.[2] However, the long-term effect of using no bonding material has not been established.

9. REPAIR OF PAVEMENT DISTRESS PRIOR TO OVERLAY

The performance of a bonded concrete overlay depends largely on the condition of the pavement on which it is placed. If a bonded overlay is to perform well, the existing pavement must be brought to an adequate condition. This involves preoverlay repair to upgrade the existing pavement. However, bonded overlays are not a cost effective rehabilitation alternative if a substantial amount of preoverlay repair is required. The following guidelines describe the conditions for which preoverlay repairs are required.

Pumping and Loss of Support

Loss of support typically exists when visible pumping occurs. Loss of support can be verified by deflection testing at joint corners. Joints having loss of support must be identified and subsealed to fill the voids created by the eroded subbase or subgrade materials. This will help to ensure full support beneath the existing slabs and overlay. A subdrainage evaluation must also be conducted to determine if edge drains would be effective in reducing the pumping activity.

Faulting

Faulted joints will be somewhat smoothed during the surface preparation and paving operations. Where faulting is a major problem on an existing pavement to be overlaid, the pavement should be evaluated for pumping and loss of support beneath the slab corners. The bonded concrete overlay will fill in the faulted areas and grinding is not required. However, progressive faulting, if not checked, will also be exhibited in the overlay.

Cracked Slabs

Shattered slabs in the existing pavement must be replaced full depth with portland cement concrete. Full-depth repairs should also be employed to repair working transverse or longitudinal cracks. The repair joints will become joints in the bonded overlay and they therefore should be dowelled if it is expected that there will be greater than 100 heavy trucks in the outer (truck) lane. Nonrepaired working cracks in the existing slab will rapidly reflect through the overlay and begin to deteriorate.

Another means of repairing transverse cracks is the retrofitting of dowel bars across the crack. This provides good load transfer across the crack and reduces faulting and deflections. A joint must be sawed full-depth in the overlay above the newly-dowelled crack.

Other than full-depth repairs, a possibility for repairing longitudinal cracks could be "cross-stitching" with deformed rebars to help hold the crack tight. This technique has been used on newer pavements with some success.

Joint Deterioration

Serious joint deterioration problems, such as corner breaks, major spalling, or blowups, should be repaired with full-depth concrete repairs. If partial-depth joint spalling problems exists, these areas should be either sawed or milled partial-depth to sound concrete. The shallow depressed areas left by the removed concrete can be filled during the paving operation. However, it is recommended that if removal is made to a depth exceeding 2 in (51 mm), the repair should be filled and cured prior to paving to ensure a uniform surface on which to place the overlay. The joints must be maintained with fillers in the partial-depth patch areas to prevent fresh concrete from entering the joint. Project-wide deterioration at the joints should be a signal to consider other rehabilitation alternatives.

Nonsealed Joints

Existing pavement joints must be sealed to keep the bonding agent, the overlay concrete, and incompressibles from the surface preparation operations from infiltrating the joint reservoirs. This will help to reduce the build up of compressive stresses upon thermal expansion.

Pressure Relief Joints

Pressure relief joints are often placed in pavements to address pressure build-up problems. If pressure relief joints are present in the existing pavement, they must also be placed in the overlay precisely above the existing pressure relief joints. Pressure relief joints pose many potential problems in a bonded concrete overlay due to the discontinuities and the large movements associated with their use. This can result in a large amount of overlay debonding in the vicinity of the pressure relief joint.

10. SURFACE PREPARATION

The pavement must be thoroughly cleaned so that all foreign matter and surface contaminants are removed. The procedures employed must use equipment which is capable of removing paint, oil, rubber, rust, and loose concrete while not severely damaging the surface or creating environmental hazards. The procedure generally consists of initial surface preparation, to remove the contaminants and achieve desired geometry, and final surface preparation, to prepare the surface immediately before the application of the bonding agent and the overlay.

Shot blasting and cold milling have both been successfully used in the initial surface preparation. Shot blasting may be more economical in some instances and does not damage the joints as much as cold milling. However, cold milling is more effective in removing extensive surface contaminants and provides a rougher texture to which the overlay more easily adheres. As previously indicated, a concrete surface removal procedure should be followed by a secondary cleaning operation immediately prior to overlay.

Shot Blasting

Shot blasting is performed by a self-contained mechanical unit that does not cause dust or particulate problems. The machine is capable of removing all surface contaminants with the exception of asphalt concrete or asphalt cement. The machine throws abrasive metal shot at the surface in a contained cleaning head. The shot are reused as they are picked up by magnetic action and recycled to the blast wheel. The particulate matter and dust created by the operation are also picked up and discharged. The average depth of removal for this equipment is about 0.125 in (3 mm).

Care should be employed when using this equipment, as the shot can penetrate the joint reservoirs where they will lodge and not be recycled. It is recommended that a backer rod be installed in all open transverse joint reservoirs prior to the shot blasting operation. Depending on the efficiency of the vacuum attachment on the equipment, secondary cleaning may not be necessary after this procedure, but is highly recommended.

Cold Milling (Scarifying)

Cold-milling of the existing surface should remove all contaminants and loose material. The equipment should be capable of removing the old surface to depths necessary to provide a uniform profile, cross-slope, and surface texture. A depth of approximately 0.25 in (6 mm) has proven to be adequate for bonded overlay projects. Milling machines provide protection from particulate matter created by the cutting action and are usually equipped with dust abatement.

When cold-milling is used, a secondary cleaning must follow to ensure the removal of dust and particulate material from the milling operation. Secondary cleaning can be accomplished through sandblasting or water blasting, which should be followed by a pass with a mechanical sweeper or air blowing operation immediately prior to application of the bonding grout.

Secondary cleaning of the shot blasted or milled surface is accomplished using one of the following techniques.

Sandblasting

Sandblasting is strongly recommended as a secondary cleaning operation only. This method will normally remove any additional deteriorated material about 0.03 to 0.06 in (1 to 2 mm) from the surface. When proper procedures are employed, the concretes aggregates should be exposed to the extent that the colors are easily detected. As there may be some dust problems from a sandblasting operation, the determination of whether dust abatement is necessary is left to the prudence of the engineer.

Water Blasting

If water blasting is chosen as the secondary cleaning method, the equipment used should generate enough pressure to remove all remnants from the surface preparation operation. When water blasting is used, extra time must be scheduled to allow for the complete drying of the prepared surface before paving can begin.

Air Blasting

Air blasting is employed to thoroughly blow the exposed concrete free of debris caused by the milling or sandblasting operation. The debris should be blown to the nearest shoulder to avoid resettling of the debris on adjacent lanes. Air blasting equipment should be equipped with filters to prevent the spraying of compressor oil on the freshly cleaned surface.

11. PLACEMENT OF BONDING AGENT

Care must be taken to avoid contamination of the pavement once surface preparations are complete. This includes covering the existing prepared slab with protective sheeting to avoid oil and other drippings from equipment. Under no circumstances should the bonding agent be applied to a wet, moist, or unprepared surface. Specifications should require a completely dry, clean prepared surface before paving operations can begin.

Sand-cement bonding grout can be applied with a stiff brush or broom in a thin, even coating. When this procedure is used, it is emphasized that the grout should not run or puddle in depressions or low spots. The coating should be about 0.06 to 0.25 in (2 to 6 mm) thick. The grout should be placed just far enough ahead of the paving operation to avoid delaying the paver. If there are problems with the grout drying too quickly, a thin fogging of water should be applied ahead of the grout application at a distance that will allow the pavement surface to be dry prior to applying the grout. Paving should not be continued over dried grout; should the surface of the grout appear whitish-gray, additional grout should be applied before continuation of the overlay placement. Removal and replacement of thoroughly dried grout is mandatory. The determination of the need and equipment used is subject to the approval of the engineer. Equipment used in the operation should meet the standards required for the surface preparation procedures.

Applying neat-cement grout through a mechanical spraying device can give an excellent uniform coat. Mechanical spraying devices can apply the grout at a minimal distance ahead of the paving operation. An 8-ft (2.4 m) margin between the grout application and the slip-form paver should be sufficient. When using this equipment, neat cement (cement-water) must be specified. By using this application technique, the drying problems encountered when applying the grout with a brush may be reduced. If problems do exist, the same procedures for removal or application of additional grout apply.

Epoxy resin materials have had limited use on bonded overlays. Therefore it is recommended that thorough testing procedures be employed to determine the compatibility of the epoxy with the existing and overlay concrete. The specimens should be prepared in a manner closely representing the field environment. Direct shear tests or slant shear tests can be used to evaluate bond strengths.

The epoxy specified should have a liquid (low viscosity) consistency that is capable of being applied through pressure spraying equipment. Laboratory bond strength values for liquid epoxy materials have been rated at over 5000 psi [34.5 MPa] from slant shear tests. This strength value indicates failure of the concrete before failure along the bond interface. These values are more than adequate for bonded overlay applications.

12. PLACING AND FINISHING CONCRETE

The placing and finishing of the bonded concrete overlay consists of all operations to place, finish, texture, and cure the overlay. Placing the overlay concrete should be to the thickness as shown on the standard plans. Any deviation from this thickness, for filling partial-depth repairs from milling or otherwise, should be to thicknesses greater than those on the plans and included in the placing of the overlay concrete. It is extremely important that every available precaution be taken to ensure a smooth riding surface of the overlay. This requires good coordination between the contractor and the engineer on the construction activities and inspection.

Texturing the pavement should be performed to produce the desired frictional characteristics. The normal procedures employed by the specifying agency on new concrete construction should govern.

Curing of the bonded concrete overlay is a most critical aspect of the work. Shrinkage of the overlay concrete during the early curing stage is very critical. High shear stresses at the interface can result in failure of the bond. Shrinkage can be reduced to acceptable levels through a low water-cement ratio and highly efficient curing procedures.

Bonded resurfacing is best done during the cooler portions of the year. Paving should not be performed during periods when large temperature changes are expected to occur, as these temperature changes may greatly decrease the chances of obtaining a good bond. When temperatures are cool, good results have been obtained with a curing compound applied at rates of between 1.5 to 2.0 times the normal rate. It may be necessary to apply this in two passes to avoid running and puddling of the compound.

Placement of bonded overlays is not recommended during extremely hot weather (greater than 90 °F [32 °C]) due to accelerated moisture loss. If hot drying conditions such as high ambient temperatures, low humidity, drying winds, and direct sunlight exist, it is necessary to require more effective curing procedures such as wet burlap, placed on the overlay immediately after texturing is completed. The burlap must be kept wet for at least 72 hours to prevent excessive moisture loss. After removal, the surface should still be sprayed with a curing compound. Failure to cure the bonded concrete overlay adequately will result in the formation of shrinkage cracks and debonding of the overlay.

If high early strength concrete is used, a thermal curing blanket should be placed immediately after the joints are sawed, approximately 3 to 4 hours after paving. The thermal blanket is used to keep the concrete temperatures higher to promote hydration of the cement. The blanket is not placed immediately after texturing because it is believed that the high temperatures in the overlay would cause the existing slab to expand at the joints, resulting in the debonding of the overlay.

13. JOINT FORMING PROCEDURES

Joints must be cut or formed in the overlay as soon as it is feasible to do so. This will prevent the underlying joints from reflecting through the overlay in the form of a crack. Prior to applying the bonding agent, the contractor should carefully mark <u>all</u> existing pavement joints, including those formed by preoverlay full-depth repairs and repairs from rehabilitation at other times during the life of the pavement. A tolerance of 1 in (25 mm) to either side of the existing joint is acceptable. This tolerance is provided to allow for the adjustment to a crack that has initiated prior to joint forming. This will help avoid the development of secondary joint cracking and eventual delamination of the overlay at the joint. Sawing operations provide for the smoothest and cleanest joint faces.

It is essential that for overlay thicknesses of 4 in (102 mm) or less, joints in the overlay be sawed completely through the thickness of the overlay <u>plus</u> another 0.5 in (13 mm). This is to ensure that the full thickness of the overlay is cut. For overlay thicknesses in excess of 4 in (102 mm), a minimum of 3-in (76 mm) sawcut depth is recommended.

A standard joint sealant reservoir should be provided at transverse joints, and the joints sealed with a high quality sealant.

Pressure relief or expansion joints in the existing pavement should be specially marked and reformed as such in the overlay. Any particularly wide joint in the existing slab should also be cut wider in the overlay to avoid any potential for point to point contact in the overlay.

Longitudinal joint sawing should also be performed. Failure to provide a sawed longitudinal joint above an existing centerline joint will result in the rapid development of a reflected crack that will require routing and sealing. The depth of saw cut used and location should be governed by the same recommendations given for transverse joints. All longitudinal joints should be sealed with a high quality sealant.

The centerline joint requires a special marking method, especially where inserts were used in the original construction. This is because, in many cases, insert longitudinal joints do not follow a straight line, nor do they evenly split the two lanes. If the centerline joint in the overlay does not match the existing joint location, a reflective crack will form along the new joint, which can lead to delamination. To mark the location of the existing longitudinal joint in the overlay, the existing joint must be referenced from the survey baseline (paver stringline) using horizontal offsets. The offset measurements should be taken prior to the commencement of paving. After texturing and curing operations, the sawing crew can mark the location of the longitudinal centerline joint using an offset string and chalkline.

14. PREPARATION OF PLANS AND SPECIFICATIONS

It is recommended that a complete list and location of all preoverlay repair areas be included on the plans. Reference should be made to pavement stationing or other markings to define the size and type of repair warranted at each location. As-built changes in these plans should be noted. As joints must be cut over the full-depth repairs as well, they too must be carefully marked.

Existing profile grades and the necessary changes of this grade to be completed during a milling or other surface removal operation or during the paving operation should be clearly noted on the plans.

Specifications have been developed by several agencies for the construction of bonded concrete overlays.[50,51]. These specifications should be consulted prior to the actual construction of a bonded concrete overlay.

15. SUMMARY

Bonded concrete overlays will provide good performance when they are properly designed and constructed. Key aspects that must be considered are presented in the preceding sections.

Cracking in the existing pavement is expected to reflect through the overlay during the very early stages of the life of the overlay. Working cracks will reflect through to about the same severity as that of the crack in the existing pavement. However, the thicker the overlay, the less severe the reflected crack will tend to be in comparison to that of the existing crack. Therefore, bonded concrete overlays should not be placed on pavements with a significant amount of working cracks unless the cracks are repaired with full-depth repairs and joints are formed in the overlay at the repair joints. However, if a substantial amount of preoverlay repair is required, the construction of the bonded overlay is probably not a cost-effective rehabilitation alternative.

Secondary joint cracking is a significant distress on bonded concrete overlays. Because there were many projects that had no secondary joint cracking, it is obvious that good construction techniques are available to eliminate this problem. The critical item is sawing the overlay joint as soon as possible before a crack forms from contraction of the base slab. It is recommended that joints be sawed completely through the overlay as soon as possible after placement.

Faulting of the overlay transverse joints has not generally been observed to be a significant problem, although several sections in this study had faulting greater than 0.12 in (3 mm). Similar to the development of faulting on new pavements, faulting on bonded overlays is believed to be rapid at first and then level out. It is further believed that the development of faulting in bonded concrete overlays will be similar to the development of faulting of the existing pavement when it was opened to traffic. This is because, other than a reduction in deflection due to a thicker slab, the overlay provides no preventive measures against faulting. Thus, subdrainage and/or reducing water infiltration are needed if the existing pavement has faulted considerably.

Visible pumping has not been observed to be a problem on bonded overlays for the traffic levels applied. The decreased deflections from thick monolithic slabs has some effect on reducing pumping. A drainage evaluation should always be conducted to determine if the installation of edge drains would benefit the performance of the pavement.

While very little shrinkage occurs on conventional concrete pavements, a substantial amount of shrinkage cracking was observed on bonded concrete overlays. This indicates that either curing of the overlay concrete was not adequate, or that the mix design was inadequate. However, less than ideal weather conditions, (e.g., high temperatures [above 90 °F (32 °C)] and wind velocity) at the time of construction of the overlay may require upgraded curing techniques to prevent shrinkage cracking and debonding. The need to prevent this problem may warrant the application of wet burlap and curing compound at twice the normal rate for most projects. Paving should not be attempted if large temperature changes are expected to occur.

Many occurrences of debonding of the overlay at corners indicates the need for improved techniques to achieve bond. Analytical results show that horizontal shear stresses are greatest at the edges and can become large enough to cause debonding if efficient curing methods and low water/cement ratios are not used. The use of liquid low viscosity epoxy resin material has shown promise of providing improved bonding in California. Based upon the amount of apparent debonding observed during the 1988 field tests, further research into obtaining a more permanent and reliable bond is greatly needed.

The concept of "fast tracking" a bonded concrete overlay, where the project is opened to traffic in 24 hours, has been recently developed.[52] This methodology is expected to increase the attractiveness and the applicability of bonded concrete overlays. However, the effect of the early opening on the bond formed between the overlay and existing slab needs to be evaluated.

Table 39. General and design data for projects included in study.

PROJECT LOCATION	Number of Sects. Eval.	Env. Zone	Project Average Section Length, FT	Thornthwaite Moisture Index	Corps of Engr. Freez. Index	No. of Freeze/ Thaw Cycles/ Year	Highest Average Daily Maximum Temp, F	Lowest Average Daily Minimum Temp, F	Average Maximum-Average Minimum Temp, F	Average Number of Days Precip/ Year	Annual Average Precip, IN
I-81 Syracuse, NY	1	WF	1075	45	990	90	82	15	87	171	39.0
I-80 Grinnell, IA	1	WF	1055	30	625	100	85	7	78	105	35.1
I-80 Avoca, IA	1	WF	850	18	688	90	87	8	79	97	31.9
CR C17 Clayton Cty, IA	7	WF	343	18	875	90	84	8	76	112	31.5
SR 12 Sioux City, IA	1	WF	1054	2	875	90	86	6	80	96	25.2
US 20 Waterloo, IA	1	WF	879	20	868	90	83	5	78	108	33.2
I-80 Truckee, CA	1	WF	1012	80	07	180	82	13	69	90	31.2
SR 38A Sioux Falls, SD	1	DF	1082	-4	1000	90	86	2	84	98	25.2
I-25 Douglas, WY	1	DF	780	-19	750	140	90	14	78	92	13.2
US 61 Baton Rouge, LA	1	WNF		37	0	<20	91	41	50	109	55.8
TOTAL	17										

Table 40. Bonded overlay surface design data.

Project Location (Orig. Const. Date/ Overlay Const. Date)	Project Section ID	BONDED OVERLAY SURFACE THICKNESS, IN Design	Core	Joint Spacing, FT	% Steel JRCP	Skewed Joints Y/N	E, KSI from FWD*	Initial Surface Prep	Final Surf. Prep	Type of Bonding Grout	Pre-Overlay Repair	Trans. Joints Matched
I-81 Syracuse, NY NB (1957/1981)	NY 6	3.0	5.0	43.0	0.00	N	3690	MILL/SB	AB	CEM/SND	PDR/PRJ/ REINF	Y
I-80 Grinnell, IA WB (1964/1984)	IA 1	4.0	4.8	76.5	0.00	N	4490	SHB	AB	CEMENT	FDR/PRJ REINF	Y
I-80 Avoca, IA EB (1966/1979)	IA 2	3.0	3.2	76.5	0.00	N	3920	MILL	SB	CEMENT	FDR/PDR PRJ	Y
C 17 Clayton Cty, IA E/W (1968/1977)	IA 3-1	3.0	3.7	40.0	0.00	N	3100	SB	AB	CEM/SND	----	Y
	IA 3-2	3.0	N/A	40.0	0.22	N	3300	SB	AB	CEM/SND	----	Y
	IA 3-3	5.0	5.9	40.0	0.00	N	3200	MILL	AB	CEM/SND	----	Y
	IA 3-4	5.0	N/A	40.0	0.13	N	2100	MILL	AB	CEM/SND	PDR/FDR/	Y
	IA 3-5	4.0	3.9	40.0	0.00	N	3780	SB	AB	CEM/SND	REINF	Y
	IA 3-6	4.0	4.6	40.0	0.16	N	3600	MILL	AB	CEM/SND	----	Y
	IA 3-7	2.0	2.8	40.0	0.00	N	6010	SB	AB	CEM/SND	----	Y
SR 12 Sioux City, IA NB (1954/1978)	IA 4	3.0	4.7	20.0	0.00	N	6500	MILL	SB	CEM/SND	PDR/PRJ	Y
US 20 Waterloo, IA WB (1958/1976)	IA 5	2.0	2.7	20.0	0.00	N	4550	MILL/SB	AB	CEM/SND	PDR/FDR	Y*
I-80 Truckee, CA EB (1964/1984)	CA 13	3.0	3.0	12-13-19-18	0.00	Y	9450	SHB	AB	EPOXY	REINF/ MILL	Y
SR 38A Sioux Falls, SD E/W (1950/1985)	SD 1	3.0	3.5	15.0	0.00	N	4930	SHB	SHB	CEMENT	FDR/PDR PRJ/ REINF	Y
I-25 Douglas, WY SB (1968/1984)	WY 1-1	3.0	2.8	20.0	0.00	Y	5010	MILL/SB	AB	CEMENT	FDR/PDR SEALING SUBSEAL	Y
US 61 Baton Rouge, LA (1959/1981)	LA 1	4.0	4.5	20.0	0.00	N	4860	SHB	AB	CEMENT	PRJ/PDR	Y

* E surface is a composite E for overlay and original pavement.

* Only 20% were sawed.

Table 41. Original PCC surface design data.

Project Location (Orig. Const. Date/ Overlay Const. Date)	Project Section ID	ORIGINAL PCC SURFACE THICKNESS, IN Design	Core	Joint Spacing, FT	% Steel JRCP	Skewed Joints Y/N	DOWELS Dia., IN	Coating	Mr,PSI (from cores)
I-81 Syracuse, NY NB (1957/1981)	NY 6	9.0	8.3	43	0.12	N	1.25	STEEL	776
I-80 Grinnell, IA WB (1964/1984)	IA 1	10.0	10.2	76.5	0.19	N	1.25	GREASE	667
I-80 Avoca, IA EB (1966/1979)	IA 2	10.0	8.9	76.5	0.16	N	1.25	GREASE	854
C 17 Clayton Cty, IA E/W (1968/1977)	IA 3-1	6.0	6.2	40	0.00	N	0.00	---	623
	IA 3-2	6.0	N/A	40	0.00	N	0.00	---	623
	IA 3-3	6.0	8.4	40	0.00	N	0.00	---	623
	IA 3-4	6.0	N/A	40	0.00	N	0.00	---	623
	IA 3-5	6.0	5.8	40	0.00	N	0.00	---	623
	IA 3-6	6.0	6.0	40	0.00	N	0.00	---	623
	IA 3-7	6.0	6.2	40	0.00	N	0.00	---	623
SR 12 Sioux City, IA NB (1954/1978)	IA 4	9.0	9.0	20	0.00	N	0.00	---	833
US 20 Waterloo, IA WB (1958/1976)	IA 5	10.0	9.5	20	0.00	N	0.00	---	676
I-80 Truckee, CA EB (1964/1984)	CA 13	8.0	7.0	12-13-19-18	0.00	Y	0.00	---	900
SR 38A Sioux Falls, SD E/W (1950/1985)	SD 1	8.0	7.9	20	0.00	N	0.00	---	705
I-25 Douglas, WY SB (1968/1984)	WY 1-1	8.0	7.7	20	0.00	Y	0.00	---	805
US 61 Baton Rouge, LA (1959/1981)	LA 1	9.0	9.4	20	0.00	N	1.13		929

Table 42. Design data for supporting pavement layers and outer shoulder.

Project Location (Orig. Const. Date/ Overlay Const. Date)	Project Section ID	BASE						SUBBASE				SUBGRADE		OUTER SHOULDER		
		Type	THICKNESS, IN Design	THICKNESS, IN Core	Estmtd. Perm., FT/HR	Keff, (Dyn.) PCI		Type	THICKNESS, IN Design	THICKNESS, IN Core	Estmtd. Perm., FT/HR	AASHTO Soil Type	Estmtd. Perm., FT/HR	Type	THICKNESS, IN Surface	THICKNESS, IN Base
I-81 Syracuse, NY NB (1957/1981)	NY 6	AGG	12.0	6.0	0.16	331		NONE	---	---	---	A-4	0.11	AC	2.0	5.0
I-80 Grinnell, IA WB (1964/1984)	IA 1	AGG	4.0	*22.8	0.21	215		AGG	4.0	N/A	0.21	A-7	0.11	AC	2.0	10.0
I-80 Avoca, IA EB (1966/1979)	IA 2	AGG	4.0	*22.5	0.01	161		NONE	---	---	---	A-7	0.11	AC	1.0	6.0
C 17 Clayton Cty, IA E/W (1968/1977)	IA 3-1	AGG	N/A	6.5	0.33	211		NONE	---	---	---	A-4	0.11	AGG*	3.0	6.0
	IA 3-2	AGG	N/A	---	---	156		NONE	---	---	---	A-4	0.11	AGG*	3.0	6.0
	IA 3-3	AGG	N/A	5.5	0.12	62		NONE	---	---	---	A-4	0.11	AGG*	5.0	6.0
	IA 3-4	AGG	N/A	---	---	158		NONE	---	---	---	A-4	0.11	AGG*	5.0	6.0
	IA 3-5	AGG	N/A	7.1	0.04	143		NONE	---	---	---	A-4	0.11	AGG*	4.0	6.0
	IA 3-6	AGG	N/A	5.5	0.63	129		NONE	---	---	---	A-4	0.11	AGG*	4.0	6.0
	IA 3-7	AGG	N/A	---	---	189		NONE	---	---	---	A-4	0.11	AGG*	2.0	6.0
SR 12 Sioux City, IA NB (1954/1978)	IA 4	AGG	N/A	---	---	67		NONE	---	---	---	A-7-6	0.11	N/A	---	---
US 20 Waterloo, IA WB (1958/1976)	IA 5	NONE	---	*13.5	0.20	209		NONE	---	---	---	A-6	0.28	AGG	3.0	10.0
I-80 Truckee, CA EB (1964/1984)	CA 13	CTB	4.0	4.0	---	218		AGG	12.0	10.1	1.33	A-2-4	N/A	AC	4.0	3.0
SR 38A Sioux Falls, SD E/W (1950/1985)	SD 1	AGG	6.0	3.5	0.17	103		NONE	---	---	---	A-7-6	0.28	AGG	3.0	3.0
I-25 Douglas, WY SB (1968/1984)	WY 1-1	AGG	6.0	*20.5	0.08	111		NONE	---	---	---	A-6	N/A	PCC	5.0	6.0
US 61 Baton Rouge, LA (1959/1981)	LA 1	SAND	6.0	*22.0	0.01	206		NONE	---	---	---	A-4	0.07	AC	3.0	7.0

* Probably a combination of base and/or subbase and subgrade.

* 6' non-tied PCC bike lane is adjacent to pvmt.

NOTE: Granular material typical of an aggregate base was found beneath the sections with "NONE" as a base type. The depth of that layer is reported.

Table 43. Pavement joint data.

Project Location (Orig. Const. Date/ Overlay Const. Date)	Project Section ID	Overlay T, IN	OL Pvt. Type	Exist Slab T, IN	Exist Pvt. Type	Joint Spacing FT	OVERLAY TRANSVERSE JOINT: Skewed Joints Y/N	Jt Seal Shape Factor	JOINT SEALANT: Type	Age	Cond.	Depth of Trans. Joint IN	Depth of Long. Joint IN
I-81 Syracuse, NY NB (1957/1981)	NY 6	3.0	JPCP	9.0	JRCP	43.0	N	0.38	PREF	6	EXC	5.0	2.0
I-80 Grinnell, IA WB (1964/1984)	IA 1	4.0	JPCP	10.0	JRCP	76.5	N	0.40	HP	4	GOOD	4.0	1.5
I-80 Avoca, IA EB (1966/1979)	IA 2	3.0	JPCP	10.0	JRCP	76.5	N	1.00	HP	9	POOR	3.0	0.0
C 17 Clayton Cty, IA E/W (1968/1977)	IA 3-1	3.0	JPCP	6.0	JPCP	40.0	N	0.33	HP	11	FAIR	1.5	0.0
	IA 3-2	3.0	JRCP	6.0	JPCP	40.0	N	0.33	HP	11	GOOD	1.5	0.0
	IA 3-3	5.0	JPCP	6.0	JPCP	40.0	N	0.20	HP	11	FAIR	1.5	0.0
	IA 3-4	5.0	JRCP	6.0	JPCP	40.0	N	0.20	HP	11	FAIR	1.5	0.0
	IA 3-5	4.0	JPCP	6.0	JPCP	40.0	N	0.25	HP	11	FAIR	1.5	0.0
	IA 3-6	4.0	JRCP	6.0	JPCP	40.0	N	0.25	HP	11	FAIR	1.5	0.0
	IA 3-7	2.0	JPCP	6.0	JPCP	40.0	N	0.50	HP	11	FAIR	1.0	0.0
SR 12 Sioux City, IA NB (1954/1978)	IA 4	3.0	JPCP	9.0	JPCP	20.0	N	0.40	HP	10	FAIR	3.0	1.0
US 20 Waterloo, IA WB (1958/1976)	IA 5	2.0	JPCP	10.0	JPCP	20.0	N	0.40	HP	12	POOR	1.0	0.0
I-80 Truckee, CA * EB (1964/1984)	CA 13	3.0	JPCP	8.0	JPCP	12-13-19-18	Y	1.60	SIL	3	FAIR	3.0	N/A
SR 38A Sioux Falls, SD E/W (1950/1985)	SD 1	3.0	JPCP	8.0	JPCP	15.0	N	1.00	SIL	3	GOOD	3.0	1.0
I-25 Douglas, WY SB (1968/1984)	WY 1-1	3.0	JPCP	9.0	JPCP	20.0	Y	0.75	SIL	4	FAIR	3.0	2.0
US 61 Baton Rouge, LA (1959/1981)	LA 1	4.0	JPCP	9.0	JPCP	20.0		1.00				4.0	

* Lane # 2 is reinforced.

Table 44. Deflection data for the outer lane.

Project Location (Orig. Const. Date/ Overlay Const. Date)	Project Section ID	Over-lay T, IN	OL Pvt. Type	Exist Slab T, IN	Exist Pvt. Type	Joint Spacing FT	DEFLECTION, mils Mid-Slab High	Low	Ave.	Loaded Corner	Non-loaded Corner	Area of Defl. Basin	Adj. % LTE	Avg. Pvmt Test Temp, F	Percent Corners with Voids
I-81 Syracuse, NY NB (1957/1981)	NY 6	3.0	JPCP	9.0	JRCP	43.0	6.5	2.1	3.8	18.8	4.8	42.1	29	66	55
I-80 Grinnell, IA WB (1964/1984)	IA 1	4.0	JPCP	10.0	JRCP	76.5	2.8	2.1	2.4	7.6	6.1	48.9	85	77	10
I-80 Avoca, IA EB (1966/1979)	IA 2	3.0	JPCP	10.0	JRCP	76.5	3.9	2.9	3.3	15.6	9.4	48.2	64	71	21
C 17 Clayton Cty, IA E/W (1968/1977)	IA 3-1	3.0	JPCP	6.0	JPCP	40.0	5.4	3.7	4.7	24.6	7.4	45.5	32	60	50
	IA 3-2	3.0	JRCP	6.0	JPCP	40.0	6.1	5.6	5.9	22.7	5.3	39.3	30	60	50
	IA 3-3	5.0	JPCP	6.0	JPCP	40.0	6.7	6.7	---	29.6	7.4	49.3	26	60	33
	IA 3-4	5.0	JRCP	6.0	JPCP	40.0	5.4	5.4	---	29.4	3.8	43.4	14	62	100
	IA 3-5	4.0	JPCP	6.0	JPCP	40.0	14.8	4.0	6.6	16.8	8.0	42.4	50	62	23
	IA 3-6	4.0	JRCP	6.0	JPCP	40.0	8.9	8.9	---	17.8	5.5	34.0	38	62	0
	IA 3-7	2.0	JPCP	6.0	JPCP	40.0	8.4	3.4	5.2	15.1	10.3	43.1	74	68	29
SR 12 Sioux City, IA NB (1954/1978)	IA 4	3.0	JPCP	9.0	JPCP	20.0	5.9	3.4	4.5	4.1	4.1	51.2	100	96	0
US 20 Waterloo, IA WB (1958/1976)	IA 5	2.0	JPCP	10.0	JPCP	20.0	3.4	2.5	2.8	13.6	5.5	45.1	45	68	35
I-80 Truckee, CA * EB (1964/1984)	CA 13	3.0	JPCP	8.0	JPCP	12-13-19-18	6.0	3.9	4.7	3.5	3.0	41.7	100	55	0
SR 38A Sioux Falls, SD E/W (1950/1985)	SD 1	3.0	JPCP	8.0	JPCP	15.0	5.9	4.0	4.7	15.0	6.4	47.2	46	69	33
I-25 Douglas, WY SB (1968/1984)	WY 1-1	3.0	JPCP	9.0	JPCP	20.0	5.7	3.7	4.5	11.0	5.4	47.9	52	62	20
US 61 Baton Rouge, LA (1959/1981)	LA 1	4.0	JPCP	9.0	JPCP	20.0	3.3	2.5	2.8	6.4	5.3	48.0	88	69	0

* Lane # 2 is reinforced.

Table 45. Traffic information.

Project Location (Orig. Const. Date/ Overlay Const. Date)	Project Section ID	ORIGINAL DESIGN TRAFFIC ESAL's, (million)	ADT, thous.	% Trucks	Age at Overlay	Age of Overlay at Survey	OUTER LN Estimated ESAL's to Overlay	LANE #2 Estimated ESAL's to Overlay	1988 ESTIMATED ADT, thous.	% Trucks	OUTER LANE (1) 1988 ESAL from ADT, % Trucks	Accum. ESAL on Overlay	LANE # 2 1988 ESAL from ADT, % Trucks	Accum ESAL on Overlay
I-81 Syracuse, NY NB (1957/OCT 1981)	NY 6				24	6	3.55	1.38	45.4	8.0	0.51	2.54	0.254	1.21
I-80 Grinnell, IA WB (1964/AUG 1984)	IA 1				20	4	11.80	2.23	18.6	34.1	1.55	6.31	0.358	1.41
I-80 Avoca, IA EB (1966/SEPT 1979)	IA 2	6.0	9.5	11.0	13	9	5.41	0.81	12.7	33.2	1.07	7.93	0.198	1.39
C 17 Clayton Cty, IA E/W (1968/OCT 1977)	IA 3-1				9	11	0.35	0.15	0.7	43.0	0.13	1.03	0.046	0.44
	IA 3-2				9	11	0.35	0.15	0.7	43.0	0.13	1.03	0.046	0.44
	IA 3-3				9	11	0.35	0.15	0.7	43.0	0.13	1.03	0.046	0.44
	IA 3-4				9	11	0.35	0.15	0.7	43.0	0.13	1.03	0.046	0.44
	IA 3-5				9	11	0.35	0.15	0.7	43.0	0.13	1.03	0.046	0.44
	IA 3-6				9	11	0.35	0.15	0.7	43.0	0.13	1.03	0.046	0.44
	IA 3-7				9	11	0.35	0.15	0.7	43.0	0.13	1.03	0.046	0.44
SR 12 Sioux City, IA NB (1954/APRIL 1978)	IA 4				24	10	1.66	0.18	7.2	8.8	0.15	1.31	0.019	0.15
US 20 Waterloo, IA WB (1958/OCT 1976)	IA 5				18	12	1.19	0.09	3.2	10.8	0.09	1.32	0.004	0.11
I-80 Truckee, CA EB (1964/OCT 1984)	CA 13				20	3	5.90	0.86	11.7	30.0	0.78	3.09	0.137	0.53
SR 38A Sioux Falls, SD E/W (1950/JUNE 1985)	SD 1				35	3	1.13	1.13	5.8	13.1	0.22	0.71	0.22	0.71
I-25 Douglas, WY SB (1968/SEPT 1984)	WY 1				16	4	2.02	0.10	4.8	29.1	0.39	1.50	0.030	0.11
US 61 Baton Rouge, LA (1959/APRIL 1981)	LA 1		6.24	11.0	22	6	2.69	0.28	10.6	14.6	0.32	2.09	0.053	0.32

NOTES: 1) IA 3 and SD 1 are two-lane; Lane #2 is Westbound.
2) All ADT's are two-way.
3) ESAL's are reported in millions.

Table 46. Outer shoulder information.

Project Location (Orig. Const. Date/ Overlay Const. Date)	Project Section ID	Over-lay T, IN	OL Pvt. Type	Exist Slab T, IN	Exist Pvt. Type	Joint Spacing FT	TYPE-Thickness, IN Surface	Base	Overall Shoulder Condition	Shoulder Joint Sealant Condition
I-81 Syracuse, NY NB (1957/1981)	NY 6	3.0	JPCP	9.0	JRCP	43.0	AC- 2	AC-5	FAIR	NONE
I-80 Grinnell, IA WB (1964/1984)	IA 1	4.0	JPCP	10.0	JRCP	76.5	AC- 2	ATB-10	EXC	EXC
I-80 Avoca, IA EB (1966/1979)	IA 2	3.0	JPCP	10.0	JRCP	76.5	AC- 1	ATB- 6	POOR	FAIR
C 17 Clayton Cty, IA E/W (1968/1977)	IA 3-1	3.0	JPCP	6.0	JPCP	40.0	*AGG-3	SG-6	N/A	N/A
	IA 3-2	3.0	JRCP	6.0	JPCP	40.0	*AGG-3	SG-6	N/A	N/A
	IA 3-3	5.0	JPCP	6.0	JPCP	40.0	*AGG-5	SG-6	N/A	N/A
	IA 3-4	5.0	JRCP	6.0	JPCP	40.0	*AGG-5	SG-6	N/A	N/A
	IA 3-5	4.0	JPCP	6.0	JPCP	40.0	*AGG-4	SG-6	N/A	N/A
	IA 3-6	4.0	JRCP	6.0	JPCP	40.0	*AGG-4	SG-6	N/A	N/A
	IA 3-7	2.0	JPCP	6.0	JPCP	40.0	*AGG-2	SG-6	N/A	N/A
SR 12 Sioux City, IA NB (1954/1978)	IA 4	3.0	JPCP	9.0	JPCP	20.0	N/A	N/A	N/A	N/A
US 20 Waterloo, IA WB (1958/1976)	IA 5	2.0	JPCP	10.0	JPCP	20.0	AGG-3	SG-10	POOR	N/A
I-80 Truckee, CA * EB (1964/1984)	CA 13	3.0	JPCP	8.0	JPCP	12-13-19-18	AC- 4	AC- 3	FAIR	N/A
SR 38A Sioux Falls, SD E/W (1950/1985)	SD 1	3.0	JPCP	8.0	JPCP	15.0	AGG-3	AGG-3	FAIR	N/A
I-25 Douglas, WY SB (1968/1984)	WY 1-1	3.0	JPCP	9.0	JPCP	20.0	PC- 5	AGG-6	GOOD	FAIR
US 61 Baton Rouge, LA (1959/1981)	LA 1	4.0	JPCP	9.0	JPCP	20.0	AC-			

* Lane # 2 is reinforced.

* 6' non-tied PCC bike lane is adjacent to pavement.

Table 47. Drainage information.

Project Location (Orig. Const. Date/ Overlay Const. Date)	Project Section ID	Over-lay T, IN	OL Pvt. Type	Exist Slab T, IN	Exist Pvt. Type	Joint Spacing FT	Permeability, FT/HR Base	Sub-base	Sub-grade	Sub-Drainage Type	Depth to Ditch FT	Average Trans. Slope, %%	Average Longit. Grade, % %
I-81 Syracuse, NY NB (1957/1981)	NY 6	3.0	JPCP	9.0	JRCP	43.0	0.16	0.22	0.11	NONE	.8.0	1.39	0.35
I-80 Grinnell, IA WB (1964/1984)	IA 1	4.0	JPCP	10.0	JRCP	76.5	0.21	0.21	0.11	EDG DRN*	10.0	-2.08	-1.56
I-80 Avoca, IA EB (1966/1979)	IA 2	3.0	JPCP	10.0	JRCP	76.5	0.01	---	0.11	EDG DRN*	115.0	-1.56	0.35
C 17 Clayton Cty, IA E/W (1968/1977)	IA 3-1	3.0	JPCP	6.0	JPCP	40.0	0.33	---	0.11	N NONE	15.0	-2.32	0.52
	IA 3-2	3.0	JRCP	6.0	JPCP	40.0	N/A	N/A	0.11	NONE	15.0	-2.32	0.52
	IA 3-3	5.0	JPCP	6.0	JPCP	40.0	0.12	---	0.11	NONE	15.0	-2.08	0.00
	IA 3-4	5.0	JRCP	6.0	JPCP	40.0	N/A	N/A	0.11	NONE	15.0	-2.08	0.00
	IA 3-5	4.0	JPCP	6.0	JPCP	40.0	0.04	---	0.11	NONE	15.0	-1.56	0.26
	IA 3-6	4.0	JRCP	6.0	JPCP	40.0	0.63	---	0.11	NONE	15.0	-1.04	0.52
	IA 3-7	2.0	JPCP	6.0	JPCP	40.0	---	---	0.11	NONE	40.0	-1.39	-4.35
SR 12 Sioux City, IA NB (1954/1978)	IA 4	3.0	JPCP	9.0	JPCP	20.0	---	---	0.11	NONE E	1 1.0	-3.13	-2.08
US 20 Waterloo, IA WB (1958/1976)	IA 5	2.0	JPCP	10.0	JPCP	20.0	0.20	---	0.28	NONE E	1 10.0	-1.39	0.00
I-80 Truckee, CA * EB (1964/1984)	CA 13	3.0	JPCP	8.0	JPCP	12-13-19-18	---	1.33	N/A	NONE	>50	-4.34	-6.08
SR 38A Sioux Falls, SD E/W (1950/1985)	SD 1	3.0	JPCP	8.0	JPCP	15.0	0.17	---	0.28	N NONE	5.0	-1.04	0.00
I-25 Douglas, WY SB (1968/1984)	WY 1-1	3.0	JPCP	9.0	JPCP	20.0	0.08	---	N/A	NONE	10.0	4.86	2.08
US 61 Baton Rouge, LA (1959/1981)	LA 1	4.0	JPCP	9.0	JPCP	20.0	0.01	---	0.07	EDG DRN*			

* Lane # 2 is reinforced.

* Added at time of overlay construction.

Table 48. Bonding data for the outer lane.

Project Location (Orig. Const. Date/ Overlay Const. Date)	Project Section ID	Over-lay T, IN	OL Pvt. Type	Exist Slab T, IN	Exist Pvt. Type	Joint Spacing FT	DEBONDING % Slab Area	% of Joint Corners	% of Crack Corner	% of Wheel-Path	Type of Bonding Grout	Ambient Temp at Time of Paving, F	Shear Str., psi	Percent Corners with Voids	Max. Corn. Defl, mils
I-81 Syracuse, NY NB (1957/1981)	NY 6	3.0	JPCP	9.0	JRCP	43.0	3.0	95.0	N/A	0.0	CEM/SND	50 - 80	N/A	54.6	27
I-80 Grinnell, IA WB (1964/1984)	IA 1	4.0	JPCP	10.0	JRCP	78.5	5.8	22.2	27.1	0.0	CEMENT	51 - 78	714*	10.0	15.9
I-80 Avoca, IA EB (1966/1979)	IA 2	3.0	JPCP	10.0	JRCP	76.5	1.7	30.0	11.4	0.0	CEMENT	51 - 79	756	20.8	37.2
C 17 Clayton Cty, IA E/W (1968/1977)	IA 3-1	3.0	JPCP	6.0	JPCP	40.0	69.6	100.0	100.0	0.0	CEM/SND	49 - 73	347*	50.0	49.3
	IA 3-2	3.0	JRCP	6.0	JPCP	40.0	72.3	50.0	100.0	0.0	CEM/SND	49 - 73	N/A	50.0	24.1
	IA 3-3	5.0	JPCP	6.0	JPCP	40.0	77.9	100.0	100.0	0.0	CEM/SND	49 - 73	586*	33.3	32.2
	IA 3-4	5.0	JRCP	6.0	JPCP	40.0	N/A	N/A	N/A	N/A	CEM/SND	49 - 73	N/A	100.0	31
	IA 3-5	4.0	JPCP	6.0	JPCP	40.0	46.5	77.8	100.0	0.0	CEM/SND	49 - 73	509*	23.1	21.6
	IA 3-6	4.0	JRCP	6.0	JPCP	40.0	N/A	N/A	N/A	N/A	CEM/SND	49 - 73	378*	0.0	18.1
	IA 3-7	2.0	JPCP	6.0	JPCP	40.0	26.2	92.9	91.7	0.0	CEM/SND	49 - 73	310*	29.0	28.4
SR 12 Sioux City, IA NB (1954/1978)	IA 4	3.0	JPCP	9.0	JPCP	20.0	5.5	3.3	8.3	0.0	CEM/SND	41 - 50	537*	0.0	4.7
US 20 Waterloo, IA WB (1958/1976)	IA 5	2.0	JPCP	10.0	JPCP	20.0	45.7	83.3	76.9	0.0	CEM/SND	74	160*	35.0	26.5
I-80 Truckee, CA * EB (1964/1984)	CA 13	3.0	JPCP	8.0	JPCP	12-13-19-18	56.8	75.0	0.0	19.0	EPOXY	46 - 86	N/A	0.0	6.1
SR 38A Sioux Falls, SD E/W (1950/1985)	SD 1	3.0	JPCP	8.0	JPCP	15.0	0.1	9.2	N/A	0.0	CEMENT	57 - 80	675*	33.3	20.5
I-25 Douglas, WY SB (1968/1984)	WY 1-1	3.0	JPCP	9.0	JPCP	20.0	3.6	52.9	N/A	1.0	CEMENT	65 - 86	0*	20.0	15.8
US 61 Baton Rouge, LA (1959/1981)	LA 1	4.0	JPCP	9.0	JPCP	20.0	10.3	93.1	N/A	0.0	CEMENT		N/A	0.0	8.1

* Lane # 2 is reinforced.

* From corner cores.

Table 49. Performance data for lane 1.

Project Location (Orig. Const. Date/ Overlay Const. Date)	Project Section ID	Over-lay T, IN	OL Pvt. Type	Exist Slab T, IN	Exist Pvt. Type	Joint Spacing FT	Ave. PSR	Mays Rough., IN/MI	Avg Trans. Fault, IN.	Transverse Cracks/MILE L	M	H	Longit. Cracks, LIN FT/ MILE	Pumping N/L/M/H	Percent Joints Spalled	Mtls Dur. Dist
I-81 Syracuse, NY NB (1957/1981)	NY 6	3.0	JPCP	9.0	JRCP	43.0	3.2	135	0.07	152	20	0	20	N	0.0	NONE
I-80 Grinnell, IA WB (1964/1984)	IA 1	4.0	JPCP	10.0	JRCP	76.5	4.2	69	0.02	210	0	0	0	L	4.8	NONE
I-80 Avoca, IA EB (1966/1979)	IA 2	3.0	JPCP	10.0	JRCP	76.5	N/A	N/A	0.10	211	6	0	5280	N	25.0	NONE
C 17 Clayton Cty, IA E/W (1968/1977)	IA 3-1	3.0	JPCP	6.0	JPCP	40.0	N/A	N/A	0.07	14	284	14	5948	N	67.0	DCRK
	IA 3-2	3.0	JRCP	6.0	JPCP	40.0	N/A	N/A	0.08	0	264	0	5597	N	0.0	NONE
	IA 3-3	5.0	JPCP	6.0	JPCP	40.0	N/A	N/A	0.16	132	330	0	10560	N	0.0	NONE
	IA 3-4	5.0	JRCP	6.0	JPCP	40.0	N/A	N/A	0.11	0	330	66	9834	N	0.0	NONE
	IA 3-5	4.0	JPCP	6.0	JPCP	40.0	N/A	N/A	0.17	0	236	15	7080	N	54.6	DCRK
	IA 3-6	4.0	JRCP	6.0	JPCP	40.0	N/A	N/A	0.11	0	462	264	8580	N	33.3	NONE
	IA 3-7	2.0	JPCP	6.0	JPCP	40.0	N/A	N/A	0.06	16	20	0	309	N	44.5	DCRK
SR 12 Sioux City, IA NB (1954/1978)	IA 4	3.0	JPCP	9.0	JPCP	20.0	N/A	N/A	0.07	10	55	5	100	N	4.4	NONE
US 20 Waterloo, IA WB (1958/1976)	IA 5	2.0	JPCP	10.0	JPCP	20.0	2.4	174	0.12	198	228	18	5935	N	40.0	NONE
I-80 Truckee, CA * EB (1964/1984)	CA 13	3.0	JPCP	8.0	JPCP	12-13-19-18	4.2	134	0.00	245	0	0	1002	N	3.0	NONE
SR 38A Sioux Falls, SD E/W (1950/1985)	SD 1	3.0	JPCP	8.0	JPCP	15.0	4.2	59	0.03	0	0	0	136	N	1.3	NONE
I-25 Douglas, WY SB (1968/1984)	WY 1-1	3.0	JPCP	9.0	JPCP	20.0	4.2	82	0.04	158	42	0	2693	N	23.0	NONE
US 61 Baton Rouge, LA (1959/1981)	LA 1	4.0	JPCP	9.0	JPCP	20.0										

* Lane # 2 is reinforced.

C 17 Clayton County, IA and SD 38A Sioux Falls, SD data are from Eastbound lane.

Table 50. Performance data for lane 2.

Project Location (Orig. Const. Date/ Overlay Const. Date)	Project Section ID	Over-lay T, IN	OL Pvt. Type	Exist Slab T, IN	Exist Pvt. Type	Joint Spacing FT	Ave. PSR	Mays Rough., IN/MI	Avg. Trans. Fault, IN.	Transverse Cracks/MILE L	M	H	Long. Crk. LIN FT/ MILE	Pumping N/L/M/H	Percent Joints Spalled	Mtls Dur. Dist
I-81 Syracuse, NY NB (1957/1981)	NY 6	3.0	JPCP	9.0	JRCP	43.0	N/A	N/A	N/A	300	5	0	0	N	0.0	NONE
I-80 Grinnell, IA WB (1964/1984)	IA 1	4.0	JPCP	10.0	JRCP	76.5	4.2	44	N/A	225	0	0	5	L	6.7	NONE
I-80 Avoca, IA EB (1966/1979)	IA 2	3.0	JPCP	10.0	JRCP	76.5	3.7	173	N/A	162	0	0	0	N	23.1	NONE
C 17 Clayton Cty, IA E/W (1968/1977)	IA 3-1	3.0	JPCP	6.0	JPCP	40.0	N/A	N/A	N/A	14	185	28	0	N	22.2	DCRK
	IA 3-2	3.0	JRCP	6.0	JPCP	40.0	N/A	N/A	N/A	106	0	0	0	N	0.0	NONE
	IA 3-3	5.0	JPCP	6.0	JPCP	40.0	N/A	N/A	N/A	132	0	0	5280	N	33.3	NONE
	IA 3-4	5.0	JRCP	6.0	JPCP	40.0	N/A	N/A	N/A	66	132	66	2574	N	33.3	NONE
	IA 3-5	4.0	JPCP	6.0	JPCP	40.0	N/A	N/A	N/A	177	0	0	1770	N	27.3	DRCK
	IA 3-6	4.0	JRCP	6.0	JPCP	40.0	N/A	N/A	N/A	0	396	0	4554	N	0.0	NONE
	IA 3-7	2.0	JPCP	6.0	JPCP	40.0	N/A	N/A	N/A	4	24	0	214	N	27.8	DCRK
SR 12 Sioux City, IA NB (1954/1978)	IA 4	3.0	JPCP	9.0	JPCP	20.0	2.4	163	N/A	5	50	15	5260	N	29.8	NONE
US 20 Waterloo, IA WB (1958/1976)	IA 5	2.0	JPCP	10.0	JPCP	20.0	2.6	201	N/A	168	128	6	222	N	50.0	DCRK
I-80 Truckee, CA * EB (1964/1984)	CA 13	3.0	JPCP	8.0	JPCP	12-13-19-18	N/A	N/A	N/A	136	0	0	543	N	1.5	NONE
SR 38A Sioux Falls, SD E/W (1950/1985)	SD 1	3.0	JPCP	8.0	JPCP	15.0	4.0	72	N/A	0	0	0	259	N	6.3	NONE
I-25 Douglas, WY SB (1968/1984)	WY 1-1	3.0	JPCP	9.0	JPCP	20.0	4.0	113	N/A	0	0	0	539	N	3.8	NONE
US 61 Baton Rouge, LA (1959/1981)	LA 1	4.0	JPCP	9.0	JPCP	20.0										

* Lane # 2 is reinforced.

C 17 Clayton County, IA and SD 38A Sioux Falls, SD data are from Westbound lane.

APPENDIX C CORE LOG FOR THIN BONDED OVERLAY SECTIONS

Appendix C contains the core log for the thin bonded overlay sections. The core location and core thickness are reported as well as a description of the core upon retrieval and testing. The Iowa Department of Transportation method for the shear testing of bonded concrete was followed. The procedure is stated in table 51.

SECT. LOC.	OL THICK/ ORIG THICK	DESCRIPTION
NY 6 COR	5.0 in/8.25 in [127 mm/210 mm]	The core was not recovered intact. The pavement was disintegrated in the area of the reinforcement steel.
IA 1 CEN	5.0 in/10.2 in [127 mm/259mm]	Steel 3.4 in [86.4 mm] from top of existing pavement 0.32 in/0.39 in [8 mm/10 mm] in diameter. Excellent bond.
IA 1 COR	4.6 in/7.7 in [117 mm/196 mm]	(Core not recovered intact) 1.25 in [31.8 mm] diameter steel dowel, corroded, located 4.5 in [114 mm] from original surface. Extensive cracking in aggregate and mortar, starting about 3 in [76 mm] below original surface. May be some microcracking in overlay aggregate.
IA 2 CEN	2.9 in/8.7 in [99 mm/221 mm]	Excellent bond. No other distresses noted.
IA 2 COR	3.5 in/9.0 in [89 mm/229 mm]	Good bond. However, existing pavement is totally deteriorated with extensive cracking and disintegration. Visible mesh is 0.35 in x 0.24 in [9 mm x 6 mm]. Pavement appears to have had AC subsealing, as bituminous material is evident in deteriorated portion at bottom of slab.
IA 3-1CEN	4.1 in/6.2 in [104 mm/157 mm]	Nothing noted.
IA 3-1COR	3.3 in/5.9 in [84 in/150 mm]	Nothing noted.
IA 3-1COR	3.8 in/5.8 in [97 mm/147 mm]	Nothing noted.
IA 3-3COR	5.9 in/8.4 in [150 mm/213 mm]	Nothing noted.

SECT. LOC.	OL THICK/ ORIG THICK	DESCRIPTION
IA 3-5COR	3.8 in/5.7 in [97 mm/145 mm]	Nothing noted.
IA 3-5CEN	4.0 in/5.8 in [102 mm/147 mm]	Nothing noted.
IA 3-6COR	4.6 in/6.0 in [117 mm/152 mm]	Nothing noted.
IA 3-7CEN	2.4 in/6.0 in [61 mm/152 mm]	Nothing noted.
IA 3-7CEN	3.0 in/6.5 in [76 mm/165 mm]	Nothing noted.
IA 3-7COR	3.0 in/6.2 in [76 mm/157 mm]	Nothing noted.
IA 4 CEN	4.5 in/9.3 in [114 mm/236 mm]	Excellent bond, no distresses noted.
IA 4 COR	4.8 in/8.4 in [122 mm/213 mm]	Excellent bond, no distresses noted.
IA 5 COR	3.0 in/9.5 in [76 mm/241mm]	The center of a 1.25 in [31.8 mm] diameter dowel is located 3 in [76 mm] below original surface. There is extensive cracking through the aggregate and mortar, to within 0.25 in [6.4 mm] of the original surface. The slab was sitting on polyethylene, which was recovered at the bottom of the core.
IA 5 CEN	2.4 in/9.4 in [64 mm/239 mm]	Nothing noted.
SD 1 COR	3.5 in/7.8 in [89 mm/198 mm]	Red aggregate. No distress noted.
SD 1 CEN	3.5 in/8.0 in [89 mm/200 mm]	Same as above.
WY 1 COR	2.8 in/7.7 in [71 mm/196 mm]	No bond. Pieces of cement about 0.5 in [12.7 mm] thick recovered with core. Possibly from subsealing.

SECT.	LOC.	OL THICK/ ORIG THICK	DESCRIPTION
LA 1	CEN	4.4 in/9.5 in [112 mm/241 mm]	Core retrieved in good condition. Nothing noted.
LA 1	TJT	4.5 in/9.4 in [114 mm/239 mm]	Core was retrieved from transverse joint. Sealant present in both overlay and original pavement. Complete horizontal break present at about mid-depth of original pavement.

Table 51. Iowa Department of Transportation test method for shear strength of bonded concrete.

Test Method No. Iowa 406
June 1978

IOWA DEPARTMENT OF TRANSPORTATION HIGHWAY DIVISION

Office of Materials

TEST METHOD FOR DETERMINING THE SHEARING STRENGTH OF BONDED CONCRETE

Scope

This method covers the procedure used in determining the shearing strength at the bonded interface between new and old concrete. The test is normally conducted on cores drilled from completed structures or pavements.

Procedure

A. Apparatus

1. Testing jig to accommodate a 4" diameter specimen. The jig is designed to provide a direct shearing force at bonded interface.

2. Hydraulic testing machine capable of applying a smooth and uniform tensile load. The accuracy of the reading shall be with $\pm$ 1.0% of the indicated load.

B. Test Specimens

Four-inch-diameter cores are the normal test specimen. Unless otherwise specified the cores are tested in an "as received" condition.

C. Test Procedure

1. Placing the specimen

(a) Place the specimen in the testing jig in such a manner that the bonded interface is placed in the space between the main halves of the jig.

(b) In the event that the interface is irregular and cannot entirely be placed as close as practical and a special notation made.

(c) Carefully align the testing jig in the testing machine with the central axis of the jig in the center of the testing machine.

2. Rate of Loading

(a) Apply the tensile load continuously and without shock. Apply the load at a constant rate with-in the range of 400 to 500 psi per minute.

(b) Continue the loading until the specimen fails, and record the maximum load carried by the specimen during the test.

D. Calculations

Calculate the shear bond strength of the specimen by dividing the maximum load carried by the specimen during the test by the cross-sectional area and express the result to the nearest psi.

APPENDIX D BIBLIOGRAPHY

CALIFORNIA

Neal, B. F., "California's Thin Bonded PCC Overlay," Report No. FHWA/CA/TL-83/04, California Department of Transportation, June 1983, 33 pp + appendixes.

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.

IOWA

Bergren, J. V., "Bonded Portland Cement Concrete Resurfacing," Iowa Department of Transportation, Division of Highways, 24 pp.

The objective of this paper is to describe the experiences of the State of Iowa in developing and refining the process of resurfacing concrete pavements with portland cement concrete. 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, the methods of bonding, 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.

Bergren, J. V., "Bonded Portland Cement Concrete Resurfacing," Transportation Research Record 814, Transportation Research Board, 1981, pp. 66-70.

The experience of Iowa in developing and refining the procedures involved in bonded concrete overlay construction are presented in this paper. It is a condensed version of the report referenced above.

Johnson, M. L., "Bonded, Thin-Lift, Non-Reinforced Portland Cement Concrete Resurfacing," Final Report for Iowa Highway Research Board, Project HR-191, June 1980, 22 pp + Appendixes.

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.

Schroeder, C. J., R. A. Britson, and J. V. Bergren, "Bonded, Thin-Lift Non-Reinforced Portland Cement Concrete Resurfacing," Iowa Department of Transportation, Division of Highways, May 1977, 55 pp.

This report describes the construction of a 2-in (51 mm) bonded PCC overlay on a section of U.S. 20 near Waterloo, Iowa in 1976. The purpose of the project was to determine the constructability of the thin lift, to determine if adequate bond could be achieved, and to observe the long-term performance and economics of this type of construction. Surface preparation included surface repair, partial and full depth patching, sandblasting, and air blasting. It was found that all of the objectives of the project could be successfully met.

Knutson, M. J., "An Evaluation of Bonded, Thin-Lift, Non-Reinforced Portland Cement Concrete Resurfacing and Patching," Prepared for Presentation at the 62nd Annual Meeting of the American Association of State Highway and Transportation Officials, November 15, 1976, 20 pp.

This paper describes the experimental projects constructed near or in Waterloo, Iowa in 1976, including the section on U.S. 20. Several variables were included in this project, including pavement condition, surface preparation, joint sawing, concrete mixes, and curing.

Marks, V. J., "Thin Bonded Portland Cement Concrete Overlay," Progress Report for Iowa Department of Transportation Project HR-520, May 1987, 24 pp.

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 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 Delamtect 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).

Kaler, M. K., J. Lane, and M. L. Johnson, "Performance of Nongrouted Thin Bonded PCC Overlays," Construction Report, Iowa Highway Research Board Project HR-291, August 1986, 12 pp.

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.

Lane, O. J., "Thin Bonded Concrete Overlay with Fast Track Concrete: Condition and Performance Report for IDOT Project HR-531," Iowa Department of Transportation, July 1987, 15 pp.

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.

Tayabji, S. D., and C. G. Ball, "Field Evaluation of Bonded Concrete Resurfacings," Final Report, Iowa Highway Research Board Project HR-288, November 1986, 57 pp.

A field program of strain and deflection measurements was conducted to obtain information on bonded concrete resurfaced pavements that can be used as a database for verifying thickness design procedures. Data gathered included a visual condition survey, engineering properties of the original and overlay concrete, load related strain and deflection measurements, and temperature related curl measurements. The study results indicated that the bonded overlay sections were behaving monolithically. A design procedure is presented in this report for determining the thickness of the bonded overlays to strengthen the pavement and to extend the service life of the existing concrete pavement.

LOUISIANA

Temple, W. H., and S. L. Cumbaa, "Thin Bonded PCC Resurfacing," FHWA/LA-85/181, Final Report, Louisiana Department of Transportation and Development, July 1985, 63 pp.

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 of the grout.

MISSOURI

"Summary of Survey of Bonded and Unbonded Reinforced Portland Cement Concrete Overlays: 1981 Condition Survey," Investigation 80-3, Missouri Highway and Transportation Department, October 1981, 33 pp.

Bonded and unbonded wire fabric reinforced PCC overlays over existing PCC pavements on the Missouri Highway System were surveyed to determine the condition of the overlays. The minimum thickness of these overlays was 4 in (102 mm). The most recent project was built in 1970 and the oldest was constructed in 1939. The general performance of these overlays was good. Longitudinal and transverse cracking had occurred due to a variety of problems, including nondowelled joints, restraint, excessively long slabs, and thin overlays. It is possible that the bonded overlays are actually partially bonded overlays, as no mention of the use of grout is found.

NEW YORK

Obuchowski, R. H., "Construction of a Thin, Bonded Concrete Overlay," Technical Report 82-4, Materials Bureau, New York State Department of Transportation, November 1982, 26 pp.

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 shrinkage cracking and reflective cracks had not posed a problem.

Obuchowski, R. H., "Construction of Thin Bonded Concrete Overlay," Transportation Research Record 924, TRB, 1983, pp. 10-15.

The thin, bonded concrete overlay constructed on I-81 in New York is described. This paper is essentially the same as the NYSDOT report referenced above.

Vyce, J. M., "Concrete Overlays: Current Use and Applicability in New York," Special Report 62, New York State Department of Transportation, April 1979, 17 pp.

This report summarizes the results of other States' experiences with concrete overlays, and evaluates the findings with respect to New York's designs, traffic, and climate. For bonded concrete overlays, it was recommended that a minimum of 0.25 in (6 mm) of existing pavement be removed by milling or grinding. The resulting surface should be sandblasted and then cleaned. The grout should be spread just ahead of the paver.

SOUTH DAKOTA

"Construction of a Thin-Bonded Portland Cement Concrete Overlay in South Dakota," Experimental Project SD 85-03, South Dakota Department of Transportation, December 1985, 17 pp.

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.

Johnston, D., "Performance of a Thin-Bonded Portland Cement Concrete Overlay in South Dakota," First Annual Report, Experimental Project SD 85-03, South Dakota Department of Transportation, July 1986, 5 pp.

This brief report documents the performance of the bonded concrete overlay on Highway 38A after 1 year of service. In general, the pavement was in very good condition. There were some reflected transverse and longitudinal cracks, although only 15 ft (4.6 m) were from cracks that had been treated prior to placement of the overlay. The importance of stabilizing cracks with reinforcing steel, especially those of high severity, is emphasized.

<u>TEXAS</u>

Golding, S., "Experimental Thin-Bonded Concrete Overlay Pavement in Houston, Texas," Construction Report, Experimental Project TX-84-01, Texas State Department of Highways and Public Transportation, November 1985, 22 pp.

Five bonded concrete overlay test sections 200 ft (61 m) long were constructed on Interstate 610 in Houston in 1983 over a continuously reinforced concrete pavement constructed in 1970. The existing pavement displayed spalled transverse cracks, longitudinal cracks, and patches. The existing pavement was selectively repaired using polymer concrete. The repaired surface was scarified by milling and sandblasting. Prior to placing of the water-cement grout, the pavement was air blasted. The overlays constructed consisted of 2- and 3-in (51 and 76 mm) layers, with and without different types of reinforcement. At the time of the report, all of the overlays were performing well.

Golding, S., "Experimental Thin-Bonded Concrete Overlay Pavement in Houston, Texas," Interim Construction Report, Experimental Project TX-84-01, Texas State Department of Highways and Public Transportation, March 1986, 35 pp.

The before and after cracking patterns are presented in this brief report of the overlays placed on I-610 in Houston. There was essentially no spalling, but a lot of reflective cracks.

Golding, S., "Experimental Thin-Bonded Concrete Overlay Pavement in Houston, Texas," Annual Report, Experimental Project TX-84-01, Texas State Department of Highways and Public Transportation, February 1987, 28 pp.

This report presents the results of the 1986 testing of the bonded PCC overlay sections on I-610 in Houston. The condition surveys indicate that the overlay has maintained considerably reduced cracking compared with the original CRCP pavement. The fiber-reinforced sections exhibit better performance with respect to cracking than the steel-reinforced sections. Deflections were lower than prior to placement of the overlay.

Muchaw, D. B., "Thin Bonded Concrete Overlays," Experimental Project TX-84-01, Construction Report, Texas State Department of Highways and Public Transportation, October 1983, 13 pp.

Brief descriptions of the existing roadway, work done prior to placement of the thin bonded concrete, and construction of the test sections on the main lanes of I-610 in Houston are described in this report. Special Specification Item 3337 for thin bonded concrete overlays is also included in this report.

Bagate, M., B. F. McCullough, and D. Fowler, "Construction and Performance Report of an Experimental Thin Bonded Concrete Overlay Pavement in Houston," Prepared for presentation at the 64th Annual Meeting of the Transportation Research Board, January 1985, 40 pp.

This paper discusses several aspects of the experimental thin bonded overlay constructed on IH-610 in Houston. These include the initial design, the actual construction, and initial testing results. A 6-month performance report is also included. The report concluded that the construction of these pavements is feasible and that the concrete mix should be designed to minimize shrinkage cracking. It was also recommended that construction during hot and windy weather be restricted.

<u>WYOMING</u>

Horan, R.D., "Evaluation of Thin Bonded Concrete Overlay in the State of Wyoming," Unpublished Report, Wyoming State Highway Department, 1984, 5 pp. + Appendices.

The bonded overlay project on Interstate 25 in Wyoming is described in this report. Details of the preoverlay rehabilitation and construction of the overlay are provided. The report includes the results of an evaluation after the first year of service, which showed some debonding. The appendices provide construction specifications for both the rehabilitation of the existing pavement and the placement of the bonded overlay.

Horan, R.D., "Second Report on Evaluation of Thin Bonded Concrete Overlay in the State of Wyoming, Unpublished Report, Wyoming State Highway Department, May 30, 1985, 3 pp.

The results of a survey performed in January 1985 are presented here. It was concluded that the rideability of the pavement had been improved, as evidenced by the Mays roughness data. The friction numbers were also satisfactory.

GENERAL

Bagate, M., B. F. McCullough, and D. W. Fowler, "A Mechanistic Design for Thin-Bonded Concrete Overlay Pavements," FHWA/TX-88+457-3, Research Report 457-3, Texas State Department of Highways and Public Transportation, September 1987, 70 pp.

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 data from the bonded overlay project on I-610 in Houston.

Boyer, R. E., and P. T. Foxworthy, "Performance of Thin Bonded Portland Cement Concrete Overlays on Military Airfields," Proceedings, Second International Conference on Concrete Pavement Design, Purdue University, April 1981, pp. 255-264.

The construction history and performance of 14 features involving thin bonded overlays at 4 Air Force bases are discussed in this paper. The pavements range in age from 13 to 21 years and have experienced a range of traffic loadings. The construction procedure for the different pavements was similar, consisting of a scarification of not less than 0.25 in (6 mm) and, if necessary, etching with muriatic acid. The overlay was placed on top of a thin layer of grout. In general, after 17 years of service the overlays are in either VERY GOOD or GOOD condition. Problems have been associated with allowing the grout to dry before placing the overlay and failure to match the joints.

Darter, M. I. and E. J. Barenberg, "Bonded Concrete Overlays: Construction and Performance," Report No. GL-80-11, Prepared for U.S. Army Engineer Waterways Experiment Station, September 1980, 126 pp.

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 1900's. 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.

"Design of Concrete Overlays for Pavements," ACI 325.1R-67

This report updates a 1958 report and presents further information on bonded concrete resurfacing. Design procedures are presented with the caveat that they are subject to further refinement.

Domenichini, Lorenzo, "Factors Affecting Adhesion of Thin Bonded Concrete Overlays (Abridgement)," Department of Hydraulics, Transportation, and Road Construction, University of Rome.

This paper examines the external forces acting on a thin bonded concrete overlay and their effect on shear stresses at the pavement interface. The stresses are induced by shrinkage of the overlay concrete, temperature differences, and traffic. The traffic induced stresses appear to have the least effect. The shrinkage forces and the possible large temperature differentials that accompany a sudden change in weather during paving can contribute to debonding.

"Draft Guidelines for Bonded Concrete Overlays," Technical Bulletin, American Concrete Pavement Association (ACPA), 1988.

These guidelines present criteria for determining the need for a bonded concrete overlay, and the design procedures which can be followed. It also includes ACPA guide specifications for bonded PCC overlays.

Gausman, R. H., "Status of Thin-Bonded Overlays," FHWA-EP-2-1, Federal Highway Administration, June 1986, 37 pp.

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 data, as well as a brief performance evaluation, are presented for projects in Texas, New York, Louisiana, California, Wyoming, Iowa, Georgia, and South Dakota.

Felt, E. J., "Repair of Concrete Pavement," American Concrete Institute Proceedings, Vol. 57, No. 2, August 1960, pp. 139-153.

This paper discusses methods of patching concrete pavements with thin layers of bonded concrete. It presents the finding of laboratory and field work conducted over a several year period by the Portland Cement Association. It was concluded that bonded overlays can be effectively constructed. The surface preparation is important, as is the use of high quality materials and workmanship in all phases of the work. The ambient weather conditions are also important: the new concrete should not change temperature greatly during the first 24 hours, nor dry out during the first 3 days.

Felt, E. J., "Resurfacing and Patching Concrete Pavements with Bonded Concrete," Proceedings, Highway Research Board, Vol. 35, 1956, pp. 444-469.

This report covers laboratory bond tests, experimental field projects, a survey of projects in use and recommended practices concerning bonded concrete. The two major factors affecting bond are the strength and integrity of the existing concrete and the cleanliness of the old surface. Other factors of importance included the precise placement of new joints over old ones and adequate curing of the fresh concrete.

Halm, H. J., "Bonded Concrete Resurfacing," Proceedings, Second International Conference on Concrete Pavement Design, Purdue University, April 1981, pp. 411-419.

This paper details the history of thin bonded concrete overlays, which date back to 1913. Beginning in the 1970's, 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.

Hutchinson, R. L., "Resurfacing with Portland Cement Concrete," NCHRP Synthesis of Highway Practice 99, Transportation Research Board, December 1982, 90 pp.

Portland cement concrete has been used to resurface existing pavements for over 70 years. Performance data indicate that a relatively low-maintenance service life of 20 years can be expected and that many resurfacings have provided 30 to 40 years of service. There is a wide range of resurfacing methods available. Recent developments in surface cleaning techniques have resulted in a new emphasis on the use of thin, bonded concrete overlays, especially if the need is to improve rideability or surface texture of the existing pavement. There are several empirically developed design procedures available, but no widely used mechanistic-based approach.

Ibukiyama, S., S. Kokubun and K. Ishikawa, "Introduction of Recent Thin Bonded Concrete Overlay Construction and Evaluation of Those Performances in Japan," Proceedings, 4th International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 1989, pp. 193-203.

The use of thin bonded concrete overlays of CRCP and PCCP in Japan is described in this paper. This rehabilitation technique was introduced to correct for the loss of wearing surface from the use of studded tires. Steel fibers were used as reinforcement in the overlay in order to control cracking. This method, which has been in use since 1983, was found to be very successful.

Koesno, S., A. H. Meyer, and D. W. Fowler, "A Study of the Influence of the Temperature of the Substrate on the Construction of Bonded Portland Cement Concrete Overlays," Preliminary Review Copy, Research Report 1124-1F, Center for Transportation Research, University of Texas at Austin, November 1988, 122 pp.

A series of laboratory and field experiments to study the effect of substrate temperatures on bond strength at the interface of bonded concrete overlays is summarized in this report. The results showed that the bonded overlays performed better without grout and placed on a dry surface. No significant results were found concerning the effect of temperature.

Lokken, E. C., "Concrete Overlays for Concrete and Asphalt Pavements," Proceedings, Second International Conference on Concrete Pavement Design, Purdue University, April 1981, pp. 211-220.

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 life-cycle costs that are lower than those for resurfacings requiring multiple applications over the same time period and provide a higher level of serviceability and minimum disruption to traffic during the life of the concrete resurfacing.

Lundy, J. R. and B. F. McCullough, "Delamination in Bonded Concrete Overlays of Continuously Reinforced Concrete Pavement," Proceedings, 4th International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 1989, pp. 221-229.

In 1983 the Texas State Department of Highways and Public Transportation placed a bonded overlay on approximately 3.2 mi (5.2 km) of CRCP. Debonding of some areas was found to occur as soon as 1 year after construction. Several techniques were used to characterize the extent of debonding, including manual sounding, ground penetrating radar, coring, and spectral analysis of surface waves (SASW). It was concluded that at the time of the study, the best available method for the detection of delamination was the combination of manual sounding and automated recording of the survey. The ground penetrating radar was not able to locate areas of delamination and the SASW technique was very successful but too slow. It was also found that there was wide variation in the amount of delamination detected from one group of operators to another, and that the time of year that the survey is conducted has an effect on the amount of delamination detected. It was also found that grout reduced the chance of debonding. The type of aggregate and reinforcement used in the overlay was also found to influence the debonding.

PIARC, Technical Committee on Concrete Roads, Subcommittee on Concrete Overlays, 9 September 1986.

Part of this report deals with the construction of thin bonded concrete overlays. It summarizes the practices and experiences of several countries and points out the factors that appear to have the greatest effect on performance. Of these, bonding and the existing surface condition are mentioned. The most important considerations at the time of construction are the temperature conditions at the time of placement of the overlay and insufficient thickness of the overlay.

Suh, Y-C., J. R. Lundy, B. F. McCullough, and D. W. Fowler, "A Summary of Bonded Concrete Overlays," Preliminary Review Copy, Research Report 457-5F, Center for Transportation Research, University of Texas at Austin, November 1988, 94 pp.

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 were examined.

Tayabji, S. D., and P. A. Okamoto, "Thickness Design of Concrete Resurfacing," Proceedings, Third International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 1985, pp. 367-379.

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.

Verhoeven, K., "Thin Overlays of Steel Fiber Reinforced Concrete and Continuously Reinforced Concrete State of the Art in Belgium," Proceedings, 4th International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 1989, pp. 205-219.

Belgium has been constructing experimental sections of thin concrete overlays since 1982. This paper discusses two technologies, the use of steel fiber reinforced overlays and the use of thin continuously reinforced concrete overlays of concrete pavements. It was found that the steel fibers did not adequately control all types of reflective cracks. On concrete pavements with severe deterioration, an asphalt interlayer was used to reduce the effect of the underlying deterioration on the performance of the overlay.

Voigt, G. F., M. I. Darter, and S. H. Carpenter, "Field Performance of Bonded Concrete Overlays," Prepared for the Annual Meeting of the Transportation Research Board, Washington, D.C., January 1987, 42 pp.

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.

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